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Dynamically Loaded Centrifugal Model Foundations

Modèles Centrifugès des Fondations Subjugués à la Charge Dynamique

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SYNOPSIS The similarity requirements for dynamically loaded foundations are discussed with a brief description of equipment used. Cyclic elastic moduli for sands and clays are reported for offshore gravity platforms and correlated with published field data on machine foundations. Brief examples are given of the use of models in predicting the influence of non-uniform strata on deformation and collapse.

INTRODUCTION

Field observations which dominate development of geotechnical science constitute full scale "model" experiments incorporating all ground details, including those not revealed by a site investigation, together with variations in construction method, and materials. However, observations after construction cannot provide direct predictions nor do they allow a parametric study for design. Physical models can fill an important gap between field observations and analysis and enhance the engineers' feel for the modes of deformation and collapse. In common with analysis they suffer from the problem that the input data may not be representative of the site, although to a lesser degree insomuch as the stress-strain response in complex situations is in certain cases independent of the method and interpretation of element stress-strain tests.

Models may be run at scales 1/100 to 1/10 in the laboratory or 1/10 to 1/3 in the field. Examples of the latter are the Osflag 9 experimental offshore platform at Christchurch Bay, England and the 15m wide caissons subjected to cyclic loading for the Oosterschelde Closure, Holland (De Leeuw, 1976). These field models provide information on the irregularities in nature of the wave forces, Christchurch Bay, or the foundation behaviour, Oosterschelde Caisson, but even here the interpretation of the observations and the projection of the data to prototype scale are aided by model studies. Increase in model size ultimately defeats the objective of parametric studies and the field model does not simulate the distribution of gravitational foundation stress. Decrease in size ultimately mitigates against a reasonable representation of the soil strata with its fabric and irregularities, and leads to problems regarding the construction of the structural model, and the measurements of pore pressure. There is generally an optimum model size which depends on the problem.

The application of centrifugal models to practical problems is now widely established [Pokrovsky and Fyodorov (1933-1969), Mikasa et al.(1969), Avgherinos and Schofield (1969), Rowe (1975a)]. The centrifugal model can provide an ideal stress path test where multi-elements are subjected simultaneously to the correct sequence of stress change.

SCALE FACTORS AND SIMILARITY

Model scale factors are denoted herein as follows, using subscripts f and m for field and model. Linear scale, $D_m/D_f = N_s$ where D is the foundation base dimension. Centrifugal acceleration factor $g_m/g_f = N_g$. Total time scale $t_m/t_f = N_t$. Frequency factor $\Delta t_f/\Delta t_m = N_f$ where Δt is the time per cycle. The cycle number factor $N_n = N_t.\overline{N_f}$ where $\overline{N_f}$ is the average frequency factor applicable to total time.

The ideal requirements for similarity are:

(1) Dimensions of model structure and foundation layers to scale N_s.

(2) The same soil types, properties, and scaled fabric structure.

(3) $\gamma_f/\gamma_m = N_s$ for soil and water.

(4) Identical soil stress history.

(5) Identical boundary stress distribution, implying the ratio of the structural bending stiffness $(EI)_m/(EI)_f = N_s^4$, with soil stiffness $E_m = E_f$ from (3) and (4).

(6) Identical boundary stress paths.

(7) Identical degrees of drainage, namely $T_m = T_f$ where $T = C_v t/D^2$. Given items (1)-(6) and compressibility $m_{vm} = m_{vf}$, then $K_m/\mu_m = (K_f/\mu_f).N_s^2/N_t$ where K is the specific permeability of the soil and μ is the coefficient of viscosity of the

pore fluid.

(8) Identical dynamic response between soil and surface, namely $N_f . N_s = 1$.

(9) Identical strain due to creep, $N_f = N_t = 1$.

The ideal requirements cannot be satisfied simultaneously. Considering item (2), for given soil particle hardness, shape, uniformity coefficient, porosity and interparticle friction and cohesion properties, particle size does not alter the stress-strain properties. But since size and particle surface properties are related, while use of a silty fine sand rather than a coarse sand might have no marked effect, one may not substitute clay for a medium silt. Reduction in sand size reduces permeability and this can be useful in order to satisfy item (7). It is not possible to scale fabric except by forming model fissured or layered beds. However, provided the model is large compared to the fabric spacing the mass properties can be retained. For dynamic loading, to satisfy item (7) it can be advantageous to remove permeable fabric from clays by remoulding, and reconsolidating in order to recover the stress history.

Special consideration has to be given to items (7)-(9). Following the ideal requirements with $K_m = K_f$, $\mu_m = \mu_f$, $N_n = 1$, constant frequencies $N_f = \bar{N}_f$, these items lead to three different time laws, namely

(7) $N_t = N_s^2$

(8) $N_t = \dfrac{N_n}{N_f} = N_s N_f . \dfrac{N_n}{N_f} = N_s$

(9) $N_t = 1$.

These cannot be satisfied simultaneously but not all problems require that they should be. For example, item (7) applies only to cases where drainage during cyclic loading is expected, when in certain cases it can be acceptable to violate item (2) and reduce permeability. Item (7) is inapplicable to undrained or drained conditions. Item (8) can be satisfied to study resonance amplification with $N_s N_f = 1$ for short periods with $\bar{N}_f / N_f \ll 1$. Item (9) is unimportant for sands. With increase in clay fraction the effect of creep on total strain and pore pressure development, and on strain rate and dynamic amplitude magnification becomes important. Hyde and Brown (1976) have shown that the decay rate constant λ in the equation $\log \dot{\varepsilon} = \alpha - \lambda \log t$ (where $\dot{\varepsilon}$ is the strain rate at time t and α is the rate at unit time), is the same for both creep and repeated load tests. Element tests may be run at different frequencies to determine the cycle number factor $N_n > 1$ which for a given $N_f > 1$ produces the same total strain, Fig. 1. This condition defines a special value $N_t = \beta = N_n/N_f$. Thus for model tests with $1/N_s = N_f > 1$ the value $N_n > 1$ is chosen such that $N_t = N_n/N_f = \beta \lessgtr 1$. However any dynamic amplitude magnification near resonance is decreased by the increased damping or lack of creep time per cycle, and a first order correction is then necessary based on element tests. If E_1 is the stiffness modulus during cyclic loading at an element frequency equivalent to $N_f = 1$ and E_{fs} is the modulus at $N_f = 1/N_s$, the model amplitude at $N_f = 1/N_s$ should be multiplied by E_{fs}/E_1. One may apply these larger displacement amplitudes to the model, violating item (5) to compensate for the reduction in time available for creep per cycle, in order to study the effect on the degree of softening, by using cyclic displacement, rather than force, controlled wave forms. Alternatively creep time can be extended using square rather than sinusoidal wave forms.

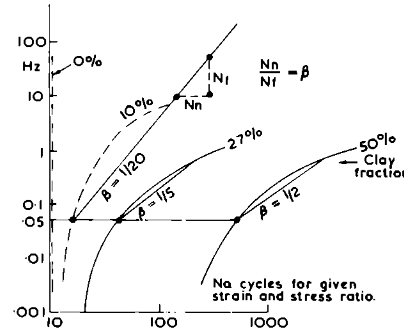

Fig. 1 Frequency-cycle number relations for identical strain

PARTICULAR CASES

Transient impulses from earthquakes at frequencies of 5 Hz require short period model excitations probably exceeding 100 Hz and are of concern solely with sands which would respond. Impulses due to dynamic impact of sands can be modelled correctly by scaling the drop mass as N_s^3 and the height as N_s. With $N_s N_g = 1$ the impulse frequency satisfies $N_s N_f = 1$ and provided relatively large models are used with N_s of the order of 1/10 the induced frequencies should be sufficiently close to field values to avoid damping errors. Hardin and Black (1966) have shown that dry sands are almost unaffected by frequency from essentially zero to several hundred hertz and Peacock and Seed (1968) have found no significant influence in the number of cycles to reach liquefaction when using 4 to 1/6 Hz, but the effect of high model frequencies exceeding 100 Hz on liquefaction has yet to be studied.

The lower range of frequency excitation on machine foundations, below resonance, and on offshore structures, presents a simpler area for dynamic studies in regard to frequency simulation but includes the more complex viscous effects of clays. Following the major work of Barkan (1962) and much published information (Prakash, 1975), the range of resonance frequencies of the order of 9-2 Hz based on observations of machine foundations, size 2-6m, on a variety of soil types may be scaled for offshore structures size 50-150m with inclusion of horizontal and rocking modes of displacement to the range 5-

0.5 Hz. Machine foundations subject to applied frequencies below resonance may be modelled at large scales, $N_s = 1/10$ using up to 90-20 Hz in the case of sands but machines operating at, say, 50 Hz would require continuous model frequencies of 500 Hz, and this would present technical difficulty even at $N_g = 10$. Offshore wave frequencies of 0.1-0.03 Hz are sufficiently below the expected resonance region to restrict the expected dynamic amplitude magnification factor to less than 1.1 so that the range $1 < N_f \leq 1/N_s$ can be used to induce weakening effects in the subsoil without always satisfying item (8).

SANDS

Neglecting item (9), items (8) and (7) are satisfied when $N_t = N_s$ and $K_m/\mu_m = (K_f/\mu_f).N_s$. The viscosity of the model fluid may be increased and/or a finer sand used. Remote from significant dynamic amplitude magnification one can use $1 \leq N_f \leq 1/N_s$, with $N_n = 1$ when item (7) becomes $K_m/\mu_m = (K_f/\mu_f).N_s^2.N_f$ and N_f may be chosen to suit the measured values of K/μ. A silty sand with a slight oil content has achieved $(K_m/\mu_m).(\mu_f/K_f) = 10^{-4}$. Element tests have been run to check similarity in the effective stress-strain relations for similar relative densities indicating that item (2) was not seriously affected.

CLAYS

For permeable clays when $C_{vm}/C_{vf} = N_{cv} < 1$, item (7) becomes $N_s = \sqrt{N_n \cdot N_{cv}/N_f}$ and to satisfy (8) simultaneously with $N_f = \overline{N}_f$, the N_s factor should equal $N_{cv}.N_n$. Alternatively one can model the undrained condition. For foundations diameter D, the maximum allowable time factor $T = C_v t/D^2 \leqq 10^{-4}$ (Rowe et al., 1976), namely $D_m \geqslant 10^2\sqrt{C_{vm}\ t_f\ N_t}$. In order to satisfy item (9), $N_t = \beta \leqq 1$. For storm build up periods of 12 hours and $C_{vm} = 1m^2/yr$, $D_m \geqslant 3.7\sqrt{\beta}$. In the case of $\beta = 1$, models need to be 3.7m diameter, which is larger than any present centrifuge can carry. However, for $\beta = 1/20$, a model diameter 0.8m satisfies the undrained condition with a model time of 12/20 hr = 0.6 hr. For a platform 100m diameter $N_f = 1/N_s = 125$, $N_g = 125$, and $N_n = \beta N_f = 125/20 = 6.25$. The number of cycles in the field over 12 hours at 0.05 Hz would be 2160 and in the model at 6.25 Hz would be 13500. Given equal strain, items (7), (8) and (9) are not violated. The value of β increases with clay fraction and with frequency, Fig. 1, so that for models of a given size there is an upper limit beyond which the creep effect cannot be taken directly into account. But one may then modify displacements and the degree of softening observed in the model using the results of undrained cyclic loading of elements.

MODEL EQUIPMENT

The technical difficulties decrease with increase in model size N_s, and decrease in N_f, N_g. For a given ratio of package dimension to radius of centrifuge the maximum value of N_s^2 is proportional to the g-tonne capacity required to accommodate the model, the loading frame and the strong box container. The Simon Engineering Laboratories centrifuge has a capacity of 700 g-tonne with a working package area of 3.15 m^2. Scales possible are $N_s = 1/10$ for machine foundations and 1/120 for 100m offshore platforms.

The initial cyclic loading equipment, dictated by limited finance, consisted of four 15 kN capacity compressed air actuators coupled in pairs and mounted parallel (A) and perpendicular (B), respectively to the soil surface. More recently actuators of 27 and 42 kN have been used, Fig. 2. The pneumatic circuit is shown in Fig. 3. A 4.5 m^3/min

Fig. 2 Actuators mounted over model offshore platform

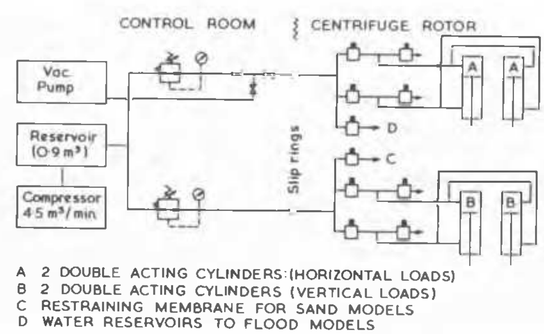

Fig. 3 Pneumatic circuit

capacity air compressor, with a 0.9 m^3 capacity air reservoir supplied air at pressures upto 850 kN/m^2 on the A and B circuits via slip rings to the package. The actuator forces were set by valves at the control console. Compressed air was admitted and exhausted using solenoid valves mounted on the centrifuge rotor close to the axis. The valves were switched automatically in sequence to achieve force cycles on the A and B lines 90° out of phase at frequencies between 0.5 and 0.05 Hz, using an electronic timing mechanism. It was also possible to separate the A actuators into static and cyclic components. The system had no servo control. At a frequency of 0.5 Hz a spiked wave form occurred. At 0.2 Hz an approximate sinusoidal form was achieved. At lower frequencies the loading approximated to a square form. Nevertheless the equipment provided valuable experience and information working generally in the range 0.2 to 0.1 Hz where creep effects for lean clays are less serious.

The present system consists of two servo controlled hydraulic oil actuators mounted in vertical and horizontal modes each having a maximum static thrust of ± 80 kN and a dynamic thrust of ± 54 kN and a total working stroke of 50mm. A 22 KVA, 44 litres/min air cooled hydraulic power pack, supplies oil at 210 bar. A new centrifuge slip ring assembly has been installed having 3 hydraulic slip rings of 210 bar capacity, 2 of 10 bar capacity, with 35 electrical power rings 10 amp, 240 volt rating, and 165 signal rings of 25m. amp, 30 volt rating. The system includes frequencies 0.005 Hz - 100 Hz, sine and square wave forms, with continuously variable phase difference between actuators 0-360°. At high N_g a limitation of 10 Hz is applicable. Chosen circuits can be switched to visual display of peak, mean, trough readings with data acquisition on punched tape at 7 channel /sec. Either load or displacement control is available. A third actuator is used with the system for triaxial element testing.

TEST DATA EXAMPLES

Work to date has been restricted to cyclic loading remote from resonance and with no significant dynamic amplitude magnification, and has been concerned either with uniform beds of sands and clays supporting circular and rectangular structures subjected mainly to rocking moments or to prototype tests of particular offshore structure designs on non uniform strata. Data relating to total and cyclic horizontal and vertical displacements and rotations, total stress and pore pressure mean and swing values have been obtained.

Two types of information illustrate the role of dynamically loaded centrifugal models, namely (a) the equivalent cyclic E_1 values which fit observed oscillations $\pm\psi$ about a vertical axis to elastic solutions for uniform soil, and (b) effects of non-uniform strata on deformation and collapse.

For rocking modes under cyclic moment M the E_1 values are determined using the relation

$$\psi = \frac{2(1 - v^2)M.\alpha_s}{D^3E_1} \quad \text{for bases dimension D}$$

where the coefficient α_s depends on the soil and structure foundation geometry (Poulos and Davis, 1974). Static overall E values were obtained from the observed immediate settlement of the structure under the vertical self weight stress applied to the foundation at constant N_g. Typical static E, and cyclic E_1, values are listed in Table I from model offshore structures on fine sands.

Table I Static and cyclic E values observed on models of large foundations

Test	Relative Density	Shape	Field Structure Dimension	Static Vertical Stress σ_v	Static E kN/m^2	Cyclic E_1 kN/m^2	Ratio E_1/E
1 dry fine sand	1.0	Circle Dia.D	D = 100 m	280 kN/m^2	100 x 10^3	1000 x 10^3	10
2 saturated fine sand	0.7	Rectangle L x B	B = 15	33	17 x 10^3	150 x 10^3	9
3 saturated fine sand	0.7	Rectangle L x B	B = 50 (L/B = 1.8)	193	36 x 10^3	300 x 10^3	8
4 saturated oily silty fine sand	0.5	Rectangle L x B	B = 15	33	3.3 x 10^3	50 x 10^3	15
5 saturated oily silty fine sand	0.5	Rectangle L x B	B = 50	193	9 x 10^3	105 x 10^3	11

For these large bases the load factor against static bearing failure is of the order of 10^3 and the equivalent strain amplitudes are very small, so that one would expect E_1 to vary as $\sqrt{\sigma}$ (Hardin and Black, 1966). In this case σ is the stress partly due to the self weight of the soil, proportional to the scale of the structure, and partly to the imposed structural bearing stress σ_v. Close to the under side of the structure σ_v dominates, whereas at depth the self weight soil stress dominates. Comparing tests 2 and 3, or 4 and 5, the ratio of σ_v values is 5.8 and the self weight values 3.3, so that the ratio of the dominant stress between the tests lies in the region of 4.5. The observed E_1 values vary by a factor of 2 so that the square root law is recovered approximately.

The published information for drained or dry sands shows cyclic E_1 values which vary by no more than a factor of 2 between loose and dense states (Lambe and Whitman, 1969), and Barkan (1962) found that porosity had little effect. In contrast, correcting the values in Table I to a common stress level the E_1 value for test 1 is some 7 times greater than for test 5. A reason could be that in test 1 no pore pressures developed whereas small instantaneous pore pressure fluctuations could have occurred in tests 2 and 3 and to a greater extent in tests 4 and 5, where pressure response was detected while satisfying scale factor item (7), although the exact pore pressure swings could not be recorded at 0.2 Hz. The equilibrium E_1 values were constant with increase in rocking moment although lower E_1 values occurred initially on raising the moment. The ratio of E_1/E of the order of 10 in each case may be of practical interest but with overconsolidated sands E would increase whereas E_1 would be expected to remain roughly the same.

Vibration theory makes use of a coefficient C_u equal to the ratio of change of stress to deflexion and called the coefficient of elastic uniform compression, which will be denoted by $\overline{C}_u$ herein to avoid confusion with the undrained shear strength of clay. Barkan (1962) relates $\overline{C}_u$ to E_1 for a vertically loaded circular rigid surface foundation on an infinite stratum by the equation

$$\overline{C}_u = 1.13\frac{E_1}{(1 - v^2)}\frac{1}{\sqrt{A}}$$

where A is the plate area, and notes that whereas tests at $N_g = 1$ for A upto 4 m^2 confirm the relation it was less satisfactory for field foundations with A = 90 m^2. As a result Barkan quotes $\overline{C}_u = (5\text{-}10) \times 10^4$ kN/m^3 independent of porosity and foundation scale, for practical use where detailed investigations are not undertaken. In the rocking mode the appropriate modulus is C_ϕ and assuming linear pressure distributions, $M/\psi = C_\phi D^4/12$ for circular bases. Equating these expressions to the elastic solutions for an infinite soil $C_\phi/\overline{C}_u = 1.57$ for circular bases and Barkan gives 1.87 for square bases. Noting that $E_1 \propto \sqrt{\sigma}$ and that σ for both self weight and applied stress for a given factor of safety is proportional to scale it follows that $\overline{C}_u \propto A^{-0.25}$ which confirms the near independence of $\overline{C}_u$ on A for machine foundations. However, the centrifuge tests in Table I indicate that for large offshore structures it is not so satisfactory to disregard the effect of scale. For example, in the case of E_1 for test 1 and an infinite foundation using

$$C_\phi = \frac{2\ E_1}{D\ (1 - v^2)} = 2.2 \times 10^4 \text{ kN/m}^3$$

the equivalent value of $\overline{C}_u$ is 1.4×10^4. At the reference scale used by Barkan of 10 m^2, D = 3.57 and the equivalent $\overline{C}_u$ is

$$1.4 \times 10^4\ \frac{100}{3.57} = 7.4 \times 10^4,$$

within the range quoted by Barkan of (5-10) x 10^4, kN/m^3. Similarly, from tests 2, 3, $\overline{C}_u$ = 2.8 and 3.1 x 10^4, close to the lower end of the range. In contrast tests 4, 5 give $\overline{C}_u$ = 0.94 and 1.07 x 10^4. Whereas agreement is reached for drained sands at Barkan's scale, $\overline{C}_u$ and C_ϕ are decreased by a factor of about 5 at 100m diameter and by a further factor depending on development of pore pressure, especially with reduced porosity. This latter process, which ultimately leads to liquefaction and $\overline{C}_u \rightarrow 0$, was foreseen by Barkan.

Barkan quotes $\overline{C}_u$ values for clays for practical use which increase with the permissible static load, namely with shear strength. He notes a large decrease in E_1, with increase in water content, namely with decrease in shear strength C_u. Tests on uniform saturated undrained clay subjected to rocking moments have given E_1/C_u = 1400-3500 at moments halfway to failure by softening, these ratios being about 4 times the immediate static ratios in the case of smooth bases or preloaded bases. In the case of bases having indentations, the initial settlements exceed those to be expected for smooth bases by an amount depending on the degree of penetration of soil into the indentations and therefore on the degree of preloading and back pressure. Penetration continues during cyclic loading.

As with sands care is necessary in using $\overline{C}_u$ independent of scale. In practice the clay stiffness may increase with depth, tending to counteract the scale effect, but for large structures it would seem preferable to make use of E_1 values for each main soil layer.

Some effects of cyclic loading on the softening of clays has been reported (Rowe, 1975b, Rowe et al., 1976). Centrifuge models can also predict deformation and failure modes, in the case of non-uniform strata. For example, sand deposits commonly suffer wide variations in relative density. Under static loading, arching occurs over looser zones and settlements can be predicted on the basis of average mass properties. In the case of low frequency (0.2 Hz) cyclic loading of bases up to 20m width resting on saturated uniform sands, the rate of drainage can prevent rise of appreciable excess pore water pressure. However, where the sand bed contains 5-10% of saturated loose zones a cyclic stress level is reached at which the arches collapse, transferring pressure instantaneously to the pore water, followed by dramatic foundation displacements. Whereas the general modes of deformation and failure can be demonstrated on a model (Rowe and Craig, 1976) it is impossible to obtain a three-dimensional survey of the porosity variations in the ground and to prepare an exact model. However, models demonstrate that unless the minimum relative density is above the critical state or the field sand stratum is densified, dynamic loading should not be applied to large structures on such foundations.

An example of offshore interest is the case of a weaker clay layer confined between stronger layers near the surface of the sea bed. Fig. 4 shows a typical foundation situation. The wave force system may be applied according to a regular or an irregular pattern. Fig. 5 shows the type of progressive shakedown settlement where creep tending to stabilisation under a given wave system is followed by accelerated movements each time larger forces are applied. This diagram includes immediate settlement and progressive penetration of base indentations at the soil surface but excludes consolidation settlement. The strength C_u refers to the weaker layer. The principal shakedown

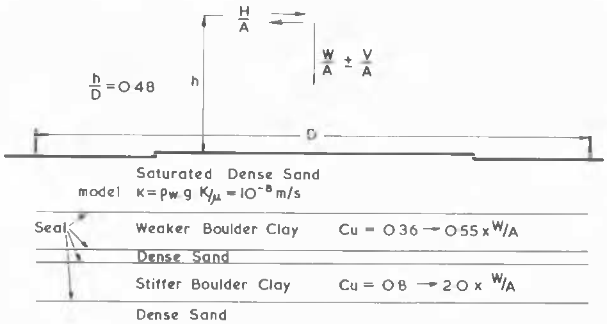

Fig. 4 Platform on layered strata

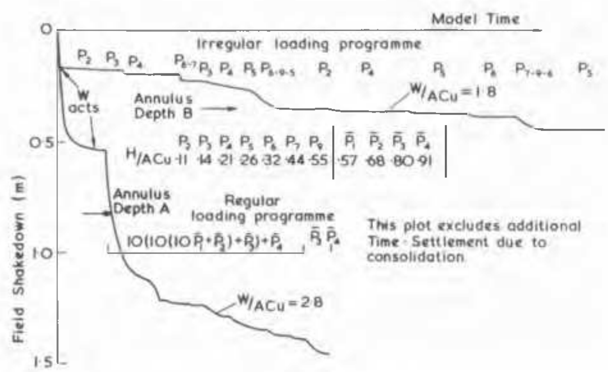

Fig. 5 Settlement-time-force records

due to yield during cyclic loading is shown in Fig. 6 for uniform clay of the same strength as the weaker layer in Fig. 4, for the case of similar although not identical numbers and pattern of wave forces. All

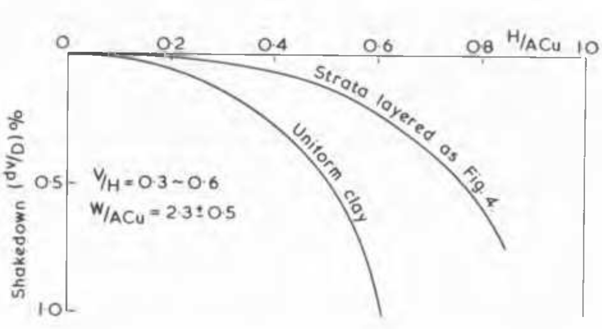

Fig. 6 Effect of layering on shakedown

tests have revealed that under cyclic loading the ultimate region of excessive deformation and softening of clays is close to, or at, the underside of the base. Where a surface stratum has a thickness of 10% the base dimension, and is substantially stronger than an underlying weaker intercalated layer, it is clearly unjustified to analyse the foundation as if the whole consisted of the weaker layer. This may appear to be self-evident but such a procedure has contributed to the rejection of a large gravity platform.

REFERENCES

Avgherinos, P.J. and A.N. Schofield, (1969), "Drawdown failures of centrifuged models," Proc.7th I.C.S.M.F.E. Mexico, Vol.2, 497-505.

Barkan, D.D., (1962), "Dynamics of bases and foundations," Pub. McGraw-Hill.

De Leeuw, E.H., (1976), "Results of large scale liquefaction tests," B.O.S.S. '76, N.I.T., Trondheim, Vol.2.

Hardin, B.O. and W.L. Black, (1966), "Sand stiffness under various triaxial stresses," A.S.C.E., J.S.M.F.D., Vol.92, SM2 : 27-42.

Hyde, A.F.L. and S.F. Brown, (1976), "The plastic deformation of a silty clay under creep and repeated loading," Geotechnique 26 : 173-184.

Lambe, T.W. and R.V. Whitman, (1969), "Soil Mechanics," Pub. J. Wiley & Sons.

Mikasa, M., Takada, N. and K. Yamada, (1969), "Centrifugal model test of a rockfill dam," Proc.7th I.C.S.M.F.E. Mexico, Vol.2, 325-333.

Peacock, W.H. and H.B. Seed, (1968), "Sand liquefaction under cyclic loading simple shear conditions," A.S.C.E., J.S.M.F.D., Vol. 94, SM3, 689-708.

Pokrovsky, G.I. and I.S. Fyodorov, (1969), "Centrifugal model testing in the construction industry," "Niedra" Pub. House, Moscow.

Poulos, H.G. and E.H. Davis, (1974), "Elastic solutions for soil and rock mechanics," J. Wiley & Sons.

Prakash, S., (1975), "Analysis and design of vibrating footings," Soil Mechanics. Recent Developments, Pub. W.H. Sellen Pty. Ltd., Zetland, Australia.

Rowe, P.W., (1972), "The relevance of soil fabric to site investigation practice," Geotechnique 22, No.2, 195-300.

Rowe, P.W., (1975a), "Application of centrifugal models to geotechnical structures," Soil Mechanics. Recent Developments, Pub. W. H. Sellen Pty. Ltd., Zetland, Australia.

Rowe, P.W., (1975b), "Displacement and failure modes of model offshore gravity platforms founded on clay," Conf. Offshore Europe 1975, Aberdeen, Pub. Spearhead Publications Ltd.

Rowe, P.W. and W.H. Craig, (1976), "Model studies of offshore caissons on sand," Proc. 2nd Offshore Structures Conf., I.C.E., London.

Rowe, P.W., Craig, W.H. and D.C. Procter, (1976), "Model studies of offshore gravity structures founded on clay," B.O.S.S. '76, N.I.T., Trondheim, Vol.1.