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Dynamic and Earthquake Geotechnical Centrifuge Modelling

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SYNOPSIS This state of art review discusses scaling principles for dynamic and earthquake geotechnical centrifuge models that have been known for many years, and shows how the new emphasis on modelling of models using reconstituted soil helps to reduce the difficulties of verification of these principles.

It is suggested that models made of reconstituted soil can exhibit a wide range of behaviour that is important in the field. In particular it is suggested that liquefaction and cracking are related phenomena which can both be modelled.

INTRODUCTION

It is not at present possible to conduct controlled experiments with large earthquakes or large explosions. Can those who need to study the mechanical damage caused by such events rely on small scale tests, using models made of reconstituted soil in geotechnical centrifuge tests using existing techniques? Can liquefaction be studied using such tests, and what other sorts of events can be modelled?

Geotechnical centrifuge modelling is among the oldest testing techniques in soil mechanics. If the technique is capable of contributing valuable data from dynamic and earthquake models, why have results of valid tests not been published before now?

SCALING PRINCIPLES UNDER QUASI STATIC LOADING

Fifty years ago Philip B Bucky (1931), an Assistant Professor of Mining at Columbia University, wrote about the use of models for the study of mining problems in the following terms:-

'In the use of models to study the effects of stresses in materials and structures with which the mining engineer is concerned - stresses arising from the weight of the materials themselves - the principles of dynamic similarity reduce to this: to produce at corresponding points in a small scale model the same unit stresses that exist in the full scale structure, the weight of the material of the model must be increased in the same ratio that the scale of the model is decreased with respect to the full scale structure. The effect of an increase in weight may be obtained by the use of centrifugal force, the model being placed in a suitable revolving apparatus.'

The same principle was proposed independently in the U S S R in 1932 by N N Davidenkov and G I Pokrovsky. A brief report of Soviet studies of soil pressures and soil deformations by means of a centrifuge which was contributed to the 1st International Conference on Soil Mechanics and Foundations in Harvard in 1936, was concerned with stability of slopes, pressure distribution under foundations, settlements of foundations and pressures on buried pipes. Two papers in the English language published in Technical Physics of the U S S R by Pokrovsky and Boulytchev (1934) and Pokrovsky and Fedorov (1935) give some details of that pioneering period, and other details can be found in two Russian language books by Pokrovsky and Fyodorov (1968 and 1969)*. The first centrifuge built in 1932-3 in the VODGEO Institute (Moscow) was assembled essentially of Ford's standard automobile parts, produced in 1929. It had a horizontal arm of effective radius one metre, which could rotate at up to 280 rpm. The loading was balanced, with a swinging carriage to contain models at either end. The first carriages had internal dimensions 30 cm high by 20 cm long by 8 cm wide, and were made of welded steel. At a model scale of n = 34.5 these containers represented lengths of sewer trench 0.08 x 34.5 = 2.76 m wide within which models of pipes diameter 1.7 m were tested. Of those tests they write:-

'The experiments were carried out with air dry as well as with moist sand and clay. The soils were moistened through a capillary rise of water, and when subjected to centrifugal action a portion of this water was squeezed out of the soil. Since in the cases where all stresses are exactly reproduced on a model, the size of which is n times smaller than that of its prototype, the gradients are n times as high as those on the prototype and the paths of filtration become as much shorter, the process

*Pokrovsky and Fedorov (1935) and Pokrovsky and Fyodorov (1968 and 1969) are the same people. English translations of the 1968 and 1969 books which were made by D R Crane and edited by A N Schofield were issued in draft form in 1975. They are now being prepared for reissue by Cambridge University Engineering Department's Soil Mechanics Group.

itself takes place on the model with a relative speed which is n^2 times as high as that in the prototype'.

Proceeding next to investigate the stress distribution in ground and the settlement below a loaded foundation, a comparison was made between a field investigation in a trench 6.5 x 3.5 x 3.0 m^3 filled with sand and a model at scale n = 30. On those tests Pokrovsky and Fyodorov write:-

'it must be pointed out that many authors have measured in the laboratory the distribution of stresses under various conditions. But in all these investigations no account was taken of the fact that the ground itself in large layers ought to change its properties under the action of its own weight, and that this condition was absent in the models. It may be assumed that the ground is not a body showing linear deformation, so that the deeper layers of the ground, compressed by the weight of the upper layers, should possess a modulus of elasticity, which is much higher than the modulus of the upper layers. As a result, a thick layer of homogeneous ground cannot be considered as uniform from the point of view of the theory of elasticity!

The paper goes on to note the additional effects caused by the shape and size of the loaded area; it concluded that parallel experiments carried out in the field and in the laboratory agreed very well.

So at that early date the scaling relationships were clear to workers in the U S A and the U S S R. Reduction of length by model factor 1/n meant area and mass reductions, and an increase of acceleration by a model factor n would give identical stresses at homologous points in model and prototype:

Scale factors	length	$1/n$
	area	$1/n^2$
	mass	$1/n^3$
	acceleration	n
	stress	1

These factors will determine the ground's 'at rest' conditions. The soil body must be initially brought into equilibrium under effective stresses σ' before it is then perturbed by a stress increment $\delta\sigma'$, and strain increments observed at homologous points are then found to be identical in model and prototype. This leads directly to consideration of dynamic perturbations.

SCALING PRINCIPLES UNDER DYNAMIC LOADING

Consider a motion (with displacement x small relative to a length dimension l of the soil body) which in the prototype is

$$x = a \sin wt \ : \text{displacement magnitude } a,$$

$$\frac{dx}{dt} = aw \cos wt \ : \text{velocity magnitude } aw,$$

$$\frac{d^2x}{dt^2} = -aw^2 \sin wt \ : \text{acceleration magnitude } -aw^2.$$

The model must experience the prototype strain x/l and since l is reduced by a factor 1/n so also must x be reduced. To experience correct stresses the model acceleration must be increased by a factor n. Hence the motion that the model must experience is:-

$$\bar{x} = \frac{a}{n} \sin nwt \ : \text{displacement magnitude } \frac{a}{n},$$

$$\frac{d\bar{x}}{dt} = aw \cos nwt \ : \text{velocity magnitude } aw,$$

$$\frac{d^2\bar{x}}{dt^2} = -naw^2 \sin nwt \ : \text{acceleration magnitude } -naw^2.$$

In dynamic tests similarity requires

Scale factors	length	$1/n$,
	velocity	1,
	acceleration	n.

Calculating the kinetic energy $\frac{1}{2}mv^2$ of moving masses in model and prototype or the potential energy mgh of falling masses, similarity requires

Scale factor	energy	n^3.

Note that there is a conflict of scale factor for time for dynamic displacements 1/n and for diffusion processes $1/n^2$, which will be discussed later.

In their 1968 book Pokrovsky and Fyodorov report that experiments were conducted at the VIOS Institute in 1940 on the detonation of explosives in sand at a scale n = 65. The cartridge had an operational efficiency of 1.32 gm of TNT, the scaling factor for energy was n^3, so this model was equivalent to an explosion of

$$1.32 \times 65^3 \text{ gm} = 360 \text{ kg of TNT}:$$

Their 1969 book ends with a long section on cratering and includes reference to cratering effects of nuclear weapons.

A small mass moving out of a crater within the rotating centrifuge model container will appear (relative to the rotating reference frame) to have Coriolis acceleration. This acceleration is in the plane perpendicular to the axis of rotation, and is perpendicular to the direction of motion of the mass. The Coriolis acceleration of a mass moving with velocity v in a plane rotating at angular velocity ω has magnitude

$$a_c = 2\omega v.$$

If the effective radius of the model in the centrifuge is R and the model acceleration is

$$a = R\omega^2$$

then the relative error caused by this Coriolis effect is

$$\frac{a_c}{a} = \frac{2\omega v}{R\omega^2} = \frac{2v}{R\omega} ,$$

which is

$$\frac{a_c}{a} = \frac{2 \times (\text{velocity of mass within container})}{(\text{velocity of container in centrifuge flight})} .$$

In a typical centrifuge design the velocity in flight may be 30 m/s and therefore velocities of less than 1.5 m/s within the model (or prototype since the scale factor for velocity is unity) will be modelled with distortions of motions less than those caused by ten per cent variation of accelerations in the prototype. At higher velocities the curving trajectories of moving masses are at first accompanied by significant misrepresentation in the model, but further increase of velocity removes the error. In effect when the masses are ejected from a crater much faster than the velocity of centrifuge flight they will strike the container wall before it has moved very far and the curvature of their path observed relative to fixed axes within the container will be negligible . Pokrovsky and Fyodorov (1968) calculate the radius of curvature R_c of the path as observed within the container using

$$a_c = \frac{v^2}{R_c} ,$$

from which

$$R_c = \frac{v^2}{a_c} = \frac{v^2}{2\omega v} = \frac{v}{2\omega} .$$

A requirement that the curvature R_c should be not less than the curvature R of the walls of the model container leads to

$$\frac{v}{2\omega} = R_c \gtrsim R$$

which occurs when

(velocity of mass within container) =

2 x (velocity of container in centrifuge flight).

So in the case where the container velocity in flight is 30 m/s there is a region of prototype velocities for which modelling errors occur between 1.5 m/s and 60 m/s. Pokrovsky and Fydorov state that in the overwhelming majority of occurences of impact or explosion the velocities are decidedly greater than the upper limit for Coriolis modelling errors.

In the case of seismic vibrations typical ground velocities will be just below the limit at which Coriolis effects cause significant errors because in most centrifuge installations the velocity of the container in its centrifuge flight will exceed 30 m/s. Pokrovsky and Fydorov report seismic loading experiments in 1940 in the VODGEO Institute at a scale of modelling n = 54 with model slopes of height 22 cm corresponding to prototype slopes of 12 m height. A special suspension of the carriage was devised, allowing the carriage suddenly to oscillate in flight. In that 1940 series of tests their slopes of dry sand with angle of internal friction $\phi = 32^\circ$ slumped to an angle of repose of 23° under a horizontal acceleration of 14 per cent gravity, while their clay loam slopes were undamaged. In their 1968 book Pokrovsky and Fyodorov conclude their brief account of these seismic model experiments as follows:-

'The main attention in the work was devoted to the investigation of an earthen embankment during a comparatively long period of vibration (of the order of 1 second). Meanwhile, it has been established in a whole series of investigations that long-period vibrations are by no means always the cause of the collapse of a structure. On the contrary, in a number of cases it is the short-period vibrations with acceleration of the order of g and even greater that turn out to be decisive. It is essential to take into account the fact that the effect of some or other vibrations on structures depends on the natural period of oscillation of the structure. Thus for example bridge supports which have comparatively small natural periods of oscillation are in the main destroyed by the effect of short-period oscillations. At the same time the spans of bridges which have considerably greater natural periods are damaged by vibrations of a greater period.'

'Turning to the material expounded in this section we can conclude that the natural period of the oscillations of the embankment under investigation is fairly large and commensurate with the period of long seismic waves.'

'Thus the devotion of the main attention in this investigation to oscillations of a comparatively significant period is correct and corresponds to the latest practical data concerning the effect of seismic vibrations of different periods on various kinds of structure. It should also be taken into account that the dimensions of the slopes of the embankment under investigation are commensurate with the length of a half-wave of long-period seismic vibrations. This means that the entire slope as a whole can undergo the simultaneous effect of the acceleration of one sign.'

Pokrovsky and Fydorov go on to report an experiment on the seismo-explosive effect of underwater explosions near earth dams. The models were at scale n = 100, with clay slopes of height 14 cm. The explosive charges corresponded to 1.32 gm of TNT and were immersed 10 cm below water at various distances from the slope.

The slumping of the model slopes was measured and compared with calculated predictions.

SCALING FACTORS FOR MODEL TESTS IN GENERAL

In their 1969 book Pokrovsky and Fyodorov introduce principles of thermal similarity, and argue that all known criteria of similarity in the theory of modelling are particular cases of a thermodynamic criterion of similarity. That book is directed to those involved in mining and underground construction, and includes discussion of use of equivalent model materials weaker than rock, to extend the scale of modelling. It further extends the discussion of dynamic events but, as in the 1968 book, pays much more attention to explosions than to seismic events.

With the publication of their 1968 and 1969 books the potential scope of geotechnical centrifuge modelling was demonstrated by Pokrovsky and Fyodorov to be very wide, and the technical difficulties not excessive for an experimental worker with adequate mechanical engineering skill. Wherever it had been possible to compare data of a model and a prototype the results had agreed very well. However, models and prototype can only correspond in some details and will differ in others, and one particular difficulty is made apparent in Pokrovsky and Fyodorov's books. It lies in the difference between the scale factor for time in dynamic events (scale factor $1/n$) and for time for diffusion effects such as pore pressure dissipation (scale factor $1/n^2$). On this question Pokrovsky and Fyodorov (1968) wrote:-

'Summarising the conclusions reached we can assert that in all those cases when the energy of the field of force is completely converted into kinetic energy, as occurs in the case of a falling body and at certain moments of elastic fluctuation, the scale of time coincides with the linear scale. In the case of continuous transmission of the energy of a field of force to surrounding bodies (or the conversion of this energy into heat) the scale of time is equal to the square of the linear scale. The method of modelling should thus be used with great care in all those cases where there is no sharply defined preponderance of elastic or inertial forces or of forces created by viscosity. One should not therefore present a review in general terms of a whole number of problems concerning the application of the method of centrifugal model testing to aero- and hydrodynamics. Each of these problems should be resolved only for some or other precise conditions, and in doing so, in some cases, centrifugal modelling will turn out to be inapplicable in principle.'

MODELLING SETTLEMENT AND MODELLING CONSTRUCTION PROCESSES

One approach to time scale factors in geotechnical model testing is to regard the initial centrifuge flight during which the stresses at every point in a model come into effective equilibrium under the initial boundary conditions as a period of initialisation. In that case no attempt is made to model any prototype process in this initial flight period. Pore pressure dissipation is observed, and not until the model reaches equilibrium is it considered ready for testing. The test may typically be a load increase in a single step without pause in the test flight: for example, Pokrovsky and Fyodorov would pour lead shot into a funnel rotating above the centrifuge axis, from which the lead shot would flow along the rotating arm, and fall out into a container in a loading plate resting on the surface of the model in the high acceleration field. The subsequent process of consolidation and settlement of the model surface would be analysed with a time scale factor $1/n^2$. In the tests reported by Pokrovsky and Fyodorov they did not have transducers avilable to them with which to observe in detail changes of pore pressure dissipation in flight, but in modern well instrumented experiments it is possible to follow the whole progress of consolidation. In order to demonstrate the internal consistency of a test series, similar models at different scales are tested: when the observed time-settlement or time-pore pressure data in the different tests are scaled by the appropriate scale factors $1/n^2$ they should agree in their prediction of prototype behaviour. This check is called 'the modelling of models', which in this case is no more than use of the standard settlement prediction of Terzaghi's consolidation theory with times scaled with the square of the thickness of the layer of clay under consideration.

Other approaches to modelling have been developed. In their 1968 book Pokrovsky and Fyodorov report tests by D.Sc. candidate T. G. Yakovleva on deformation of railway embankment models at the M I I T Institute in Moscow. For modelling the process of settlement of embankments during the period of their construction she suggested the use of the time of start-up and acceleration of the machine - the so called 'wind-up' period - to model the increase in stress in the prototype during construction. This approach was also adopted by Yu. J. Malushitsky (1981) for modelling the rapid construction of mine waste heaps of unsaturated soil. He quotes the advice of Professor I S Fyodorov that in the case of models of embankments a constant scale of time equal to the first power of the scale of acceleration should be maintained throughout the whole period of the model testing. To verify this principle, tests involving 'the model testing of the models' were made and gave the following check. A <u>first</u> group of models were of height 21.1 cm and were accelerated up to a scale n = 186.9 in a period of 25.6 minutes at which stage in the 'wind-up' they collapsed. A <u>second</u> group of models were of height 25.9 cm and were acclerated up to a scale n = 148.9 in a period of 32 minutes at which stage in the 'wind-up' they collapsed. A <u>third</u> group of models were of height 31.2 cm and were accelerated up to a scale n = 124.3 in a time of 38.6 minutes at which stage in the 'wind-up' they collapsed. Calculations for the equivalent prototype were made as follows:-

$H_1 = 186.9 \times 21.1 = 3921$ cm;

$H_2 = 148.9 \times 25.9 = 3873$ cm;

$H_3 = 124.3 \times 31.2 = 3896$ cm;

H average = 3897 cm;

$T_1 = 186.9 \times 25.6 = 4785$ minutes;

$T_2 = 148.9 \times 32.0 = 4765$ minutes;

$T_3 = 124.3 \times 38.6 = 4790$ minutes;

T average = 4780 minutes.

While the success of the modelling of models was very satisfactory, Malushitsky commented on the use of that time scale factor 1/n as follows:

'The transition from one scale of modelling to another during the process of the model testing of an unconsolidated waste heap occurs gradually and at different speeds at different levels of a section of the model. Whilst the soil mass in the upper, less heavily laden layers preserves its three phase state throughout the whole model testing process in the lower layers, with the increase of the accelerations, the consolidation of the soil may be attended by a transition to a two phase state with a gradual transposition upwards from below of the boundary between the two-phase and three-phase states of the soil material in the body of the model.'

'Accordingly different scales of time can correspond to different parts of a model at one and the same time.'

'It would be expedient to try to arrive at some intermediate scale which would correspond, at any given moment, to the correlation between the parts of the model which happen to be in different states. It is obvious, however, that it is impossible to make such a calculation during the process of centrifugal model testing and consequently it is impossible to make any alterations in the control of the scale of time, all the more so since with the commencement of the collapse of the model, with the appearance in it of the first cracks, the reverse process begins - the transition of the body of the model from a two-phase to a three-phase state.'

DIFFICULTIES FACED IN APPLICATION OF CENTRIFUGE MODEL TESTS

Malushitsky's model test series proceeded carefully to confirm the similarity between performance of models and observed failures of prototype open-cast mine waste heaps. Finally, recommendations based on a total of 255 model tests led to the elimination of land slips, the reduction of volumes of re-excavated waste material, and development of a safe method of depositing high 'dry' waste heaps on abandoned old hydraulic waste heaps.

Malushitsky's recommendations were implemented by the Chief Engineer of the 'Krasnogor' open-cast coal mine, resulting in annual savings of 725,000 roubles.

Another application of geotechnical centrifuge model testing reported by P W Rowe, W H Craig and D C Procter (1977) has been the study of cyclic loading on offshore gravity platforms and on the caissons for the Oosterschelde closure. The platforms were designed to be not near resonance, with resonant frequencies which were estimated to be between 5 - 0.5 Hz and with wave loading frequencies between 0.1 - 0.03 Hz. In general, model test frequencies were increased by a scale factor n, but to determine the number of model loading cycles needed they took the following steps. Undrained triaxial tests were run at a number of frequencies 0.01 - 10 Hz and the total strain was determined after 10 - 1000 cycles. As the model test frequency is increased so the number of model loading cycles is increased in the ratio that would give the same total strain. And since elastic moduli also change with frequency increase, the displacement amplitudes are increased to compensate for the reduction in time available for creep per cycle. The comparison of their model predictions and observations of prototype performance will continue as data of settlement of offshore structures under wave loading in storms becomes available.

A third application of geotechnical centrifuge model testing reported by R M Schmidt (1977) is the simulation of the JOHNIE BOY 500 ton nuclear cratering event. This involved a nuclear--PETN equivalence calculation, and firing of a 1.2 gm PETN charge at n = 345 as dictated by similarity requirements. The agreement of the scaled crater size with the actual prototype crater was regarded as very satisfactory.

In each of these applications use of a geotechnical centrifuge gave new physical insights into behaviour of large prototype for which there are enough data at prototype scale to make possible some comparisons of model and prototype. The existence of defence related research makes it possible that there have been unreported failures as well as unreported successes of applications of geotechnical centrifuge modelling: for example work in the Soviet Union in 1940 reported in Pokrovsky and Fyodorov's book in 1968 must have been followed by other as yet unreported work. However, in so far as there are published reports, whenever geotechnical centrifuge model tests have been undertaken in connection with a major engineering problem the tests have given reliable information.

The cost of geotechnical centrifuge model tests on each problem can represent only a small fraction of the total cost of research and tests in that field of application, and the fraction of total geotechnical engineering effort put into geotechnical centrifuge model tests over the past half century can not have increased greatly over the years. The forthcoming Tenth International Conference of Soil Mechanics and Foundation Engineering will only include nine papers on geotechnical centrifuge models out of a total of over five hundred papers. Five of these papers relate to tests on the Cambridge Geotechnical Centrifuge and none relates to Soviet work. This raises the question why a technique that appears to have proved successful remains relatively unrecognised and unused.

In their 1968 book Pokrovsky and Fyodorov place emphasis on the problem of choice of suitable sizes of centrifuge. They write:-

'Incorrect solution of this problem led to the failures of both Engineer Bucky and Professor Davidenkov. In the former case so small a machine was built (with a diameter of less than 0.5 m) that it was impossible to achieve any serious quantitative measurements on it. In the second case so massive an installation was planned (with a diameter of 10 m) that even now, even after the value of modelling on a centrifuge has been acknowledged and verified it would be difficult to get it built. ......However, on the basis of acquired experience one can consider the most appropriate dimensions for the rotating part to be in the region of 4 - 6 metres.'

But in spite of their own success in generating dozens of centrifuge test facilities at various Institutes within the Soviet Union it does not appear that even in the U S S R the technique is universally accepted. A study tour* of England, France, Russia and Japan was made from 19 June - 7 July 1977 by M E Harr and W C Sherman for the purpose of obtaining data on centrifuge testing techniques. The Hydroproject Institute, Moscow was the only Institute which they visited in the U S S R at which they considered the operations, personnel, equipment, planning and use of results of geotechnical centrifuge testing that they saw to be central to the work of the Institute. The activity at Hydroproject Institute reflects the interests of V I Vutsel and V I Scherbina (1976), and this in turn suggests that the continued successful operation of any centrifugal test facility depends more on the research team who are involved and the funds provided for centrifuge operations than on the machine.

The operations of the geotechnical centrifuges in the UK have been the subject of recent papers by A N Schofield (1980) and by W H Craig and P W Rowe (1981). Both in Cambridge and Manchester there is a small team of research engineers and technicians with many years of experience, there is a great variety of items of equipment from previous tests, and there are well established procedures for the safe and efficient initiation of new tests. In both places there has been a quarter century of unbroken continuity in geotechnical engineering research by the group involved, on lines independent of contemporaneous main-stream research, and funds for geotechnical centrifuge operation have come mostly from sources independent of the concensus of research opinion. Similar factors appear to be present in all active geotechnical centrifuge operations.

*Trip report - private communication by W C Sherman, U S Army Corps of Engineers

When tasked with reviewing the feasibility and desirability of constructing a very large centrifuge for geotechnical studies R F Scott and N R Morgan (1977) wrote:-

'In summary it is apparent from the effort put into the technique that the Russian workers consider the centrifuge technique well proven although it is not possible to discover from Pokrovsky's work any satisfactory demonstration of the correlation between model and prototype tests for any of the studies cited.'

They then concluded

'There is insufficient evidence of quantitative centrifugal test results or corroboration between prototype data and centrifugal data; until some evidence is developed it would be premature to design or construct a large centrifuge.'

'An orderly program of quantifying centrifugal test data and comparing it with full-scale data should be established.'

This demand for evidence of the capability with smaller centrifuges accurately to predict the performance of prototype events before proceeding to work with larger machines is rational but not easy for the research teams to supply for the following reason.

The design of large public works reflects a concensus of authoritative engineering opinion. The principal members of a design review panel may respect but can not be expected to defer to the opinions or fund the development of theories of independent research groups. There has been modest public expenditure on geotechnical centrifuge operations reported by R H Bassett and J Horner (1979) connected with construction works, but that was to satisfy a minority interest and not the interest of the principal members of the design team. In contrast it has been the groups concerned with geotechnical events concerning disaster or defence or energy and other resource development which have in the past funded geotechnical centrifuge operations. Information on matters of central concern to the principal members of the funding agency is often restricted. Experience with model tests of local disasters such as mine waste heap failure or storm surge levee failure has proved that in such matters public authorities who pay for model tests are sensitive to public release of the results. Nor is it likely that there will be full release of all centrifuge cratering data that have been obtained in the past 40 years despite publication of some recent US data in that field. A successful centrifuge model prediction was made without payment and contributed to the proceedings of a Foundation Deformation Prediction Symposium by R H Bassett (1974), but on the whole neither public nor private funds have been available for tests both at prototype scale and also at geotechnical centrifuge model scales with public release of all results. This reason for lack of evidence does not reduce the force of the demand for it. But the sort of evidence that should reasonably be expected from the research teams is the internal evidence of attempts to model models in well conducted and fully reported experiments.

In the summaries of the nine papers presented to the forthcoming Tenth International Conference in Stockholm only in three instances is model similitude declared to be a major research objective. R F Scott (1981) presents and discusses results of model pile tests scaled to represent certain full size but not very large pile tests. N Krebs Ovesen (1981) presents results of tests of uplift capacity of anchor slabs in sand, comparing model and field tests, with emphasis on similitude, as also in his contribution at the Seventh European Conference in Brighton, N Krebs Ovesen (1979). In contrast with these two summaries that of B Pincent and F Tchocothe (1981) on consolidation of clay layers with observation of settlement, pore pressures, and undrained cohesion after the test makes a point of reporting a departure from similarity being observed in cohesion test data. This may not be too significant. The present practice in Cambridge is to make in-situ vane tests in clay layers while models are in flight, and not to rely on undrained cohesion measurement made while the soil is swelling when removed from the centrifuge after the test. The other paper summaries emphasise the supply of information useful for design. In some cases model similitude is established but it is not made a feature of the summary.

Work is in progress at a number of centres on the development of dynamic and earthquake modelling capabilities, some of which is reported in the present Conference papers. As yet there are no reports of studies with modelling of models in sets of cratering experiments at different scales, though that is probably within present capabilities. The publication of R M Schmidt (1980) continues to extend the coverage of cratering models. There also has been simulation of crustal tectonic processes in centrifuge model tests, using scaled brittle material so that the centrifuge model scales up to a prototype approximately 2.2 km deep x 2.8 km x 3 km by H-P Liu, R L Hagman and R F Scott (1978). Clearly there is a great variety of types of test that are being planned and undertaken.

In the development of a model shaking facility at Cambridge two important questions were choice of shaking input and type of soil material behaviour to be investigated. On the first question there is clearly a better possibility of modelling models with horizontal ground shaking of simple sinusoidal wave form than with spectra resembling real earthquakes. On the second question the size of prototype that could be modelled was limited to no more than 80 m x 40 m plan area with soil depths generally not more than 20 m: the frequency of the prototype input to be modelled was expected to be not more than 2 Hz. The sort of problem that might be studied with such a facility would be failure of a high retaining wall and its backfill, the liquefaction failure of a waste heap embankment retaining a lagoon, or the cracking of a small dam. This therefore brings us first to review our expectations for behaviour of soil, and second to review the simplified ground shaking facilities that have been added to the Cambridge Geotechnical Centrifuge.

THE MECHANICAL BEHAVIOUR OF SOIL

In order to explain our expectations for material behaviour during events called liquefaction it is necessary to go back to first principles.

When elementary volumes of continuous saturated soil material are examined they are found to contain a continuous structure of solid particles of greatly varying sizes and shapes, in mechanical contact and interlocked with each other, and a continuous body of water;

$$(\text{Soil material}) \rightarrow (\text{solid particles}) + (\text{pore water}).$$

A unit volume of solid soil particles is associated with a volume e of pore space, so the specific volume v of space occupied by that given set of particles is

$$v = 1 + e.$$

The mechanical behaviour of a body of soil can be defined at each interior point if the local deformation of the particle structure is known and the movement of pore water relative to the particles is known;

$$(\text{soil mechanics}) \rightarrow (\text{mechanical deformation of particle structure}) + (\text{displacement of pore water relative to soil}).$$

The local pore water pressure u can be measured accurately and quickly with negligible flow of water from an elementary volume of soil into a modern pore pressure transducer. If a total stress tensor σ acts on a soil element, a part acts in the pore water (this hydrostatic pressure also acts around and within each soil particle), and the remaining part must be resisted effectively by the structure of particles in mechanical contact;

$$(\text{total stress}) \rightarrow (\text{pore water pressure}) + (\text{effective stress})$$

$$\sigma = u + \sigma'.$$

It was previously thought that there is an essential difference between silt particles which are in direct contact with each other, and clay particles which were supposed to be totally enveloped in water layers and out of mechanical contact. However, in the decade 1955-65 Critical State Soil Mechanics demonstrated that the mechanical properties of saturated reconstituted clays can be modelled as the elasto-plastic behaviour of a purely frictional aggregate of particles.

In a critical effective stress state an aggregate of particles will deform at constant stress without change of specific volume. Under higher effective stress the soil structure will experience plastic compression during distortion, and will tend to expel pore water or develop positive pore pressure increase. Deformation at less than critical effective stress leads soil to suck in water, and this strain softening behaviour leads to the development of thin slip planes: the increase of water content in the 'slick' or 'gouge' material is due to the mechanical interlocking of the densely packed particles in states 'on the dry side of critical states'. Undrained pore pressure changes can be explained by critical state soil mechanics and 'on the wet side of critical states' using the original Cam-clay model (Roscoe and Schofield 1963, Roscoe, Schofield and Thurairajah 1965) it proved possible to fully explain the phenomena of 'undrained cohesive strength' and even to predict plastic strains, without postulating the existence of any electro-chemical 'cohesion' between individual clay particles.

The development of this mechanical view of the properties of clay, rather than a model basing cohesion on supposed electro-chemical bonds meant that there appeared to be less need to be concerned with 'soil creep'. Our current view remains that at normal ground surface temperatures and pressures the effectively stressed structure of saturated silt or clay does not behave as a viscous continuum. All time effects in such soils are attributed primarily to the slow diffusion and flow of pore water under pore pressure gradients within the soil body;

(soil behaviour) → (elasto-plastic non viscous effectively stressed soil)

\+ (laminar flow of pore water through pore spaces).

Now, although this is our present view, there is experimental evidence of strength increase as time for deformation decreases. A discussion of the effect of this strength increase in modelling by M D Bolton, R J English, C C Hird and A N Schofield (1973) noted that this creep or stress time relaxation effect would mean that model material would be stronger than material at homologous points in the prototype. To a first approximation if the strength increases by ten per cent for each log. cycle of rate of deformation then it must be expected that in a dynamic event at scale n = 100 (when time for deformation is decreased by a factor of 100 which is two log. cycles of rate of testing) the model strength should be increased by twenty per cent. If such an effect were to be observable in modelling of dynamic models this would lead us to reconsider our decision to omit creep or stress relaxation from our constitutive models for soil.

In our elasto-plastic non-viscous model the deformation of an elementary volume of soil in a small strain increment $\delta\varepsilon$ is partly recoverable (called elastic $\delta\varepsilon^e$) and partly irrecoverable (called plastic $\delta\varepsilon^p$). The difference between these parts is most significant in stressing cases where the principle axes of the effective stress tensor σ do not coincide with the principle axes of the effective stress-increment $\delta\sigma'$. If the soil is isotropic the principle axes of the two components $\delta\varepsilon^e$ and $\delta\varepsilon^p$ of the strain increment tensor are aligned as follows;

(elastic strain increment $\delta\varepsilon^e$ direction) coincides with (stress increment $\delta\sigma'$)

(plastic strain increment $\delta\varepsilon^p$ direction) coincides with (stress σ').

This results in a subtle difference between alternative studies which treat soil as a non-linear elastic continuum, and our Cambridge studies which treat it as an elasto-plastic continuum. In the former case any non-coincidence of axes of stress increment and observed strain increment has to imply stress induced anisotropy of material properties. In the latter case it is possible to predict non-coincidence of axes while still postulating isotropic elastic and plastic properties for soil. However, if finite element calculations using isotropic constitutive models in our latest critical state package CRISP prove incapable of closely fitting data of deformations observed in model tests, this would lead us to reconsideration of our decision to omit anistropy from the constitutive models for soil which we adopt at present, which make no allowance for 'structure' in the soil except in so far as this varies with the current values of p' and v.

Our position at present remains therefore as is recorded in 'Critical State Soil Mechanics' Schofield and Wroth (1968). The earlier view that gave Mohr's circle of stress special prominence, and supposed that one could identify an 'incipient' rupture plane on which one observed 'mobilisation' of friction and cohesion has been discarded. The stress and stress increment tensors are expressed in terms of properly constituted invariants (p'q) and ($\delta p'\delta q$) where:

$$p' = (\sigma_1' + \sigma_2' + \sigma_3')/3$$

$$2q^2 = (\sigma_2' - \sigma_3')^2 + (\sigma_3'-\sigma_1')^2 + (\sigma_1' - \sigma_2')^2$$

These parameters allowed a unified treatment of elastic and plastic behaviour, and in particular the compression and rebound of soil under increase and decrease of p' was idealised into a family of lines

$$v + \kappa \ln p' = \text{const.}$$

The concept of a critical voids ratio independent of effective pressure has been replaced by a concept of critical states of effective stress and specific volume satisfying equations

$$v + \lambda \ln p' = \Gamma \;; \quad q = Mp' \;.$$

Continuous plastic deformation of soil in critical states is considered

inviscid,

frictional,

isotropic.

Adoption of a slope λ of the critical state line which exceeded the slope of the elastic compression lines has allowed Casagrande's first approach to interpretation of pore pressure changes in undrained tests to be corrected, Schofield and Togrol (1966).

The behaviour of a set of soil specimens is seen to be similar if they have the same equivalent specific volume v_c, where

$$v_c = v + \lambda \ln p'$$

Comparing behaviour of different specimens, the variations of v_c play a role comparable to 'overconsolidation ratio'. In particular soil behaviour to either side of the critical state line can be distinguished as follows:

$v_c > \Gamma$, 'wet' side : stable yielding with positive pore pressures or reduction of volume and plastic hardening; relatively 'loose'.

$v_c < \Gamma$, 'dry' side : unstable rupture with dilation on thin slip bands and increase of volume and strain softening: relatively 'dense'.

On the 'wet' side of critical states the original Cam-clay model predicts an end or limit to the elastic compression lines at which it predicts plastic compression under isotropic effective stress at states

$$v_c = \Gamma + \lambda - \kappa \quad ,$$

So the model proves capable of treating 'consolidation' and 'shear' as different aspects of the behaviour to be expected from an aggregate of frictional particles under effective stress, without any cohesion or bonds or cementing at individual particle contacts.

MODELLING LIQUEFACTION

The original theory of liquefaction, introduced to explain sudden action at a distance through a rather impermeable mass of soil, was the 'card-house-structure-collapse' or 'domino' theory. It was suggested that the tendency of soils on the 'wet' side of critical states to exhibit positive pore pressures in continuous shear deformation was evidence of partial liquefaction; and that the more 'loose' the structure the more likely the occurrence of liquefaction. This view was rejected in the 1980 Rankine lecture in connection with model tests of flow slide phenomena. In discussion of Cambridge geotechnical centrifuge operations A N Schofield (1980) suggested that what appeared to be sudden action at a distance in a flowslide avalanche was not a demonstration of the supposed card house collapse phenomenon, but the result of rapid transmission of pore fluid pressures through soil at states near zero effective stress. Soil yielding and flowing in critical stress states is a ductile plastic material with continuous energy dissipation in friction. Its permeability is associated with transmission of water through the pores of the deforming soil structure. On the 'loose' side of critical states the soil structure yields at lower than critical deviator stress, and there may be massive plastic deformation, but this is not the phenomenon which is properly described as liquefaction. The test paths that lead to liquefaction are those which exhibit falling effective stress and head away from critical states on the dry side. The stress ratio must not be too high during the test path or this will lead to rupture and to shear distortion and to dilation. But provided there is no rupture then it is cyclic loading that can cause continuous reduction of effective stress and eventual liquefaction.

The approach of soil to this relatively 'dense' and 'dry' state during cyclic loading with stress reversals can be seen for example in the work of M P Loung and J F Sidaner (1981). On the near approach to zero effective stress fracture becomes more likely than rupture. Cyclic stress relaxation of the effectively stressed particle structure does not mean that the particles are less interlocked geometrically. If at any stage in the test path they are made to undergo a shear distortion in rupture they will tend to dilate and to develop the full effective stresses that are required to reach a critical state at that particle packing. However, if they experience such a complete stress relaxation that there is virtually no contact stress between particles, then micro fissures can open. By themselves these micro fissures are not too important, but in the presence of an excess pore pressure gradient the approach to zero effective stress leads to one or other of the phenomena given the general name of liquefaction.

The essence of our approach to soil mechanics is the distinction between different forms of soil behaviour that are observed when the effective stresses change. The two questions in soil mechanics that we try to answer are how the pore fluid moves relative to the structure of soil particles, and how that structure itself is deformed. These questions can not be resolved independently. In all deformations under effective stresses not far from critical values the storage of pore fluids and their transmission through the soil structure under the influence of pore pressure gradients is governed by the conditions of flow through interconnecting pore spaces. But at mean effective stresses an order of magnitude below critical values the structure of soil particles is no longer held sufficiently closely together and is liable to fall apart with fractures or fissures. When the soil is about to crumble its behaviour under pore pressure gradient changes greatly. It can develop secondary permeability through cracks or fissures or pipes or channels which can allow the rapid transmission of pore fluids and of pore pressures.

The phenomena called 'liquefaction' or 'hydraulic fracture' or 'piping' or 'channelling' or 'boiling' or 'fissuring' or 'fluidisation' form a closely related group. They are all manifestations of soil behaviour at mean effective stresses an order of magnitude below critical values. They differ from each other only in the way that the pore pressure gradients and the transmission of pore fluids affect the body of soil in question in the period under consideration. When any of these manifestations appear in a soil specimen it is no longer a homogeneous body, and integrated effects such as total displacement or total transmission of pore fluids no longer relate to well-defined continuum properties of the material under test.

Liquefaction involves instability, as also does buckling of a steel column. One does not talk of the 'buckling potential' of mild steel. The word 'buckle' implies a body of some defined geometry undergoing a distinct change of form under load. Equally, whenever the word 'liquefaction' is written there is some geometry implicit in the writers mind. Even in liquefaction's archetypal form when fluidised soil acts as hydraulic fill, where we envisage a column of effectively unstressed soil as a homogeneous continuum, we have to introduce an upward pore pressure gradient equal to the total weight of the soil. This automatically implies non-uniformity and is the reason why we can not study elemental volumes of homogeneous hydraulic fill in laboratory conditions under the acceleration of earth's gravity (though it could perhaps be studied well in a space laboratory).

There are good reasons therefore for centrifuge models to be used for the study of liquefaction. We have already with our existing equipment observed tensile cracking of soil in centrifuge models, hydraulic fracture of wells, voids migrating during internal erosion of compacted dam cores in models in flight. We have observed cyclic rise in pore pressures in tests of a model saturated sand embankment on a resonant shaking table (to be described below) in flight in centrifuge model tests. During a brief period with high pore pressures in the centre of the embankment the embankment top was observed to be isolated from the motion of the base. After appreciable dissipation of these pore pressures, the top then gradually reverted to moving together with the base of the embankment. We intend to continue such tests and extend the range of topics to cover various problems of interest to designers. In these model tests we will need to be careful in considering time scale factors.

When considering for example the problem of filling or emptying an oil tank on saturated reclaimed land, the prototype load increment and its cyclic variation in normal service does not involve any inertial forces: the scale of time in modelling is governed by diffusion - a scale factor $1/n^2$. However, if seismic loading on a full tank is to be modelled then inertial effects must be considered and a scale factor of $1/n$ will require that seismic loading say at 1 Hz is modelled at 100 Hz in a model at scale $n = 100$. If the earthquake lasts say 10 seconds then there may be a problem with the model materials. The model must be shaken ten times at 100 Hz so the model earthquake will last one tenth of a second. But this corresponds to a period of pore water pressure dissipation of n^2 longer time in the prototype (i.e. $\frac{1}{10} \times 100 \times 100 = 1000$ seconds $\simeq \frac{1}{4}$ hour). Whether or not this creates a problem for the model depends on the foundation material. If the tank rests on a layer of highly permeable sand over a soft clay layer, then there could well be much more significant pore pressure dissipation from the permeable layer in a scaled prototype time of $\frac{1}{4}$ hour than could occur in the actual 10 second earthquake. If the question was one of possible liquefaction in the sand layer during the earthquake then special steps would be appropriate to reduce the pore pressure dissipation from the model permeable layer by reducing its permeability. A simple method by which one can reduce permeability is reduction of particle size. If ten per cent of the particles are smaller than a size denoted d_{10}, the permeability will depend on $d_{10}{}^2$. So sieving to reduce d_{10} by a factor of 10 will reduce permeability by a factor of 100. This allows retention of high pore pressures in the layer during the model earthquake in a similar way to their retention during the prototype event.

We would hope to see model sand boils in the surface of the ground under appropriate circumstances. If we were to model seismic loading of a layer of 'loose' sand under a layer of 'dense' impermeable overburden we would expect to see the overburden crack in appropriate circumstances. As the underlying sand layer collapsed we would expect to see water and sand rising through the cracks creating a sort of clastic liquefaction of the overburden. If there were a slope of the model surface we would expect to see blocks of clastic liquefied soil moving in the form of a bouyant debris flow. P W Rowe, W H Craig and D C Proctor (1977) report that the presence of 5 - 10 per cent of 'loose' zone material in a bed of saturated sand led to dramatic displacements of a cyclically loaded foundation. It would be possible for us to model the effects of 'loose' zones or of stiff inclusions in embankments during an earthquake.

THE CAMBRIDGE FACILITY AND INITIAL DYNAMIC TESTS

The design, construction and operation of the Cambridge Geotechnical Centrifuge have been described by A N Schofield (1980). The balanced arm of the rotor is 10 m tip to tip, and it operates with models at a working radius not more than 4 m, at accelerations up to 155g. The model and its container has mass not more than 900 kg. A type of container extensively used is a thin walled steel tub of 850 mm diameter and 400 mm depth into which sand or clay can be placed. Various soils have been used over the years but experience has shown that if research students are to make progress with model tests they need to work with standard soil. Two readily available commercially processed soils are standard Leighton Buzzard sand, and dry Speswhite Kaolin clay powder. This powder can be mixed in vacuum with deionised water in our laboratory to form a saturated reconstituted clay typically at 120 per cent water content. If coarse dry sand is to be used it will pour from a hopper at a controlled rate to form a sample of uniform density. If clay is to be used it is mixed in two or three batches, each of 50 kg dry clay, poured as a liquid into the tub and compressed with a large piston typically to 50 - 60 per cent water content. This clay consolidation process takes over a week in the laboratory. When the sand or clay specimens have been formed in the laboratory they are transferred to a swinging platform at the end of the centrifuge arm. In flight the model swings up and is seated against the strong end of the centrifuge arm, where it can be observed by closed circuit television. Models with layers of clay and sand have been tested Padfield (1978).

Miniature pore pressure transducers for burial in centrifuge models were first developed ten years ago in conjunction with a UK company* who supply engine pressure transducers for Rolls Royce research. They have a diffused silicon strain gauge diaphragm as the stiff elastic sensing member - the four strain gauges forming active arms of a bridge of approximately 1000 ohms. Being diffused into the elastic material of the silicon diaphragm there is full strain gauge adhesion. The stiff diaphragm is placed behind a thin porous ceramic disc and therefore senses the pore pressure in the soil within a millimetre of the location in which it occurs. A transducer with sensitivity about 0.5 mV/kPa linear to 350 kPa was found by D V Morris (1979) to be capable of measuring a rate of increase of pore pressure as high as 10^7 kPa/s. The device of approximately 0.3 gm mass and 5 mm diameter 10 mm long has wires attached by waterproof connection. It is installed in an augered hole in clay which is backfilled with clay slurry, which consolidates by the end of the consolidation process. The use of such transducers has developed over the past decade. Their role is central to the evolution of effective stress analysis of centrifuge models of saturated soil, and they appear to be fully capable of meeting the demands of dynamic model tests.

Miniature piezo-electric accelerometers with mass 4 gm and external dimensions 9.5 x 9.5 x 7.2 mm^3 were obtained from a UK company.** Their signals had to be amplified at the junction box at the end of the arm, the circuits being designed and constructed by D V Morris (1979). All transducer signals were sent through slip rings to the control room and data centre where they were amplified if necessary and recorded on a 14 track frequency modulated tape recorder obtained from a UK company.*** It was possible to play back the recordings through an XY plotter or an oscillograph to obtain plots of the analog data stored on the magnetic tape. To use the XY plotter the model oscillations had to be slowed down by playing the tape back at a slower speed than the recording speed.

* Druck Limited, Fir Tree Lane, Groby, Leicestershire; Telex 341743

** D J Birchall Ltd. Mildenhall, Suffolk

*** Racal-Thermionic Ltd. Hythe, Southampton, Hampshire

Displacements of model surfaces were measured with linear variable differential transformers (LVDTs) and recorded either on magnetic tape as above or on punched paper tape. Our data centre is being upgraded to incorporate magnetic disc store. Although a system with the specifications outlined above has proved adequate for our needs the rapid advance in the state of the art in matters of electronic data gathering makes it likely that more sophisticated systems will be developed. The contrast between our system and the systems with which the Soviet centrifuge specialists were working in the early 1970s may explain part of their difficulty in publishing contributions which advanced the technique of centrifuge model testing in that period.

Our first series of dynamic experiments at Cambridge was designed to investigate the soil-structure interaction of a model tower rocking on a foundation of dry Leighton Buzzard standard sand with particle sizes 0.6 - 1.2 mm. The towers were to be simple rigid tubes 250 mm high with bases either circular or suqare, of different sizes dimension 60 - 90 mm, resting on the level surface of the sand. The sand had uniform voids ratio about 0.65. An accelerometer mounted on the tower top signalled its motion, the outputs passing through screened leads to amplifiers on the centrifuge arm, then through the low noise electrical slip rings to the ultra violet recording oscillograph outside where the signals were recorded on sensitised paper. The original plan was to rock each tower by detonating about 20 mg of thermally sensitive explosive powder in a short length of steel tube welded to the tower top. This experimental system was designed and used by D V Morris (1979). Morris explains in an appendix to his thesis how he prepared his own silver azide explosive. A brief series of other experiments were conducted at that time using detonators. We did not continue with use of explosive, feeling it better if possible to produce dynamic excitation by mechanical means in subsequent experiments.

In the tests it was unexpectedly observed that random gusts of wind and turbulence in the centrifuge chamber excited the system of the tower and its foundation with small amplitudes at its natural frequency. Before each experiment the tower was primed with a few grains of explosive. It was then accelerated up to the planned acceleration, and after the detonation the centrifuge was slowed down and stopped. It was easy to measure the natural frequency of the tower under wind induced oscillation, pausing at intervals in the change of speed before and after each test detonation. By this means Morris tested a total of 13 different models at different modelling scales between 1 and 100 and got a large amount of quantitative information. Perhaps other geotechnical centrifuge workers may think it worth repeating these wind buffeted tower experiments and devising variations on them so a digest of D V Morris' (1981) results will be given below.

The natural frequency f_n of a rigid tower of moment of inertia I resting on a foundation system of rotational spring stiffness K is

$$f_n = \frac{1}{2\pi} \left(\frac{K}{I}\right)^{1/2} \tag{a}$$

where for a circular base of radius r resting on a uniform elastic half-space of shear modulus G and Poissons ratio ν the stiffness at low frequency is calculated by Borowicka (1963) to be

$$K = \frac{8}{3} \frac{Gr^3}{(1-\nu)} \tag{b}$$

The effective shear modulus G varies non-linearly with the mean effective stresss p' in the foundation

$$G \propto p'^x \tag{c}$$

where x is an index which could be determined from the results. The stress depended on the mass M of the tower, the acceleration ng of the model, and the base area of semi dimension r

$$p' \propto Mng/r^2 \tag{d}$$

From equations (a) to (d) it can be seen that for a given tower the natural frequency must vary with n as follows ;

$$f_n \propto n^{x/2} \tag{e}$$

Graphs of the measured variation of natural frequency with centrifugal acceleration are plotted in Figure 1 to a logarithmetric base. The value at 1g was found by tapping the tower when the centrifuge was at rest, and the values from 10 to 100 g were found from recording wind induced oscillations. The slopes of the lines are 0.25, so that in equation (c) the index x has the value of ½, agreeing with the result of Hardin and Drnevich (1972). Their resonant column tests led to a formula

$$G = 10^5 \frac{(3-e)^2}{(1+e)} p'^{\frac{1}{2}} \quad . \tag{f}$$

With e = 0.65 for these sand models equations (a) (b) and (f) gave a predicted frequency

$$f_n = 172\, r^{3/2}\, p'^{\frac{1}{4}}\, I^{-\frac{1}{2}} \tag{g}$$

which could be compared with the observed frequency.

It was consistently observed that the experimental natural frequency was about 17 per cent less than the predicted value, and similar behaviour in full scale tests has been reported by Richart and Whitman (1967). Morris explained this observed reduction in foundation stiffness as being a consequence of the peak soil stresses in elastic theory not being developed at the edges of the base. He suggested introduction of a factor α to calculate a reduced average mean effective pressure $\alpha p'$, which could be adopted as being more appropriate for estimation of shear modulus.

Since natural frequency in equation (g) varies with the quarter power of p', a value of α that would account for that observed result is

$$\alpha = (1.37/1.67)^4 = 0.45,$$

and from other results that Morris had obtained he proposed that he take α = 0.5, for his tests.†

The test series included tests on three geometrically similar model towers were made at scales in the ratio 1:2:3. When tested at accelerations in the ratio 3:2:1 these all proved to agree in their predictions of the natural frequency of a 12 m high prototype. For a 150 mm tower at 80g the observed natural frequency was 119.5Hz; for a 300 mm tower at 40g it was 61Hz; for a 450 mm tower at 26g it was 38.5Hz. The predictions for prototype frequency were therefore

$$\left.\begin{aligned} 119.5/80 &= 1.49\text{ Hz} \\ 61\ /40 &= 1.53\text{ Hz} \\ 38.5/26 &= 1.48\text{ Hz} \end{aligned}\right\}\ \text{which agree within experimental error.}$$

This check of similarity at different scales gives internal confirmation of the validity of the results. It is also appropriate to comment that in these tests the largest particle size was 1.2 mm and the smallest base size was 60 mm. With a base fifty times the size of the largest particle it was acceptable to analyse soil-structure interaction in terms of a soil continuum. We have not made a specific study of soil-structure ineraction in models with large soil particles, but so far we have had no problems with grain size.

System damping measurement by the logarithmic decrement method from the trace of oscillatory decay after detonation of explosive on the tower gave a value of material damping in agreement with values found from resonant column tests. Reduction of frequency with increase of amplitude of rotation was observed in the trace. Tests of towers on saturated fine sand 0.07 - 0.13 mm size at 1860 kg/m³, 25 per cent relative density, showed cyclic pore pressure rise under the tower. Tests of pairs of towers on a clay foundation showed cross coupling between companion towers. The clay shear modulus was measured in flight by timing a surface wave impulse between two accelerometers on the clay attached to a twin trace oscilloscope. This side of D V Morris (1979) work could have been extended to study machine foundation dynamics more generally with centrifuge models, but it was decided instead to continue to give priority to development of an in-flight shaking table with which to study a problem of liquefaction.

Gravity stress in a layer of saturated soil brings the top into a 'dry' state and the bottom into a 'wet' state. Could earthquake shaking cause the 'wet' soil to discharge water, and this liquefy the 'dry' overburden?

† This result would need to be confirmed in a variety of other tests before it could be recommended for general use.

Development of a shaking system for centrifuge models was envisaged in some detail by Scott and Morgan (1977). The paper to the present Conference by A Zelikson, B Devaure and D Badel (1981) reports the simulation of an earthquake by a programmed series of explosions. Other systems using piezo-ceramic shakers have been discussed in connection with the proposed U.S. National Centrifuge facility, but its design is not yet finalized. Pokrovsky and Fyodorov (1968 and 1969) discuss a variety of systems without reporting data of their use. The most useful contribution that can be made here in the final section of this review is to supply a brief outline of current developments at Cambridge.

CAMBRIDGE CENTRIFUGE SHAKER DEVELOPMENT

A low-cost resonant shaking box for use with models on the Cambridge centrifuge was designed by Morris to resonate at 61 Hz with a maximum acceleration of 20g, corresponding to an amplitude of 1.3 mm of movement. These values correspond to differing prototype values, depending on the modelling scale (centrifuge acceleration) at which the ground motion is induced, as follows:-

	Centrifuge	model	scale
	25	50	80
Prototype			
Frequency (Hz)	2.4	1.2	0.76
Max. amplitude (mm)	33	65	105
Max. acceleration (%g)	80	40	25

To avoid any risk to the centrifuge bearings it was decided to provide a reaction mass, attached to the shaking box by stiff leaf springs made of hardened steel in which the energy for shaking could be stored, and to isolate the whole system from the centrifuge arm. Before a test, the springs were energised by jacking the reaction mass away from the shaking box. Insertion of a preset thickness of catch pieces determined the magnitude of the horizontal shaking that would occur. The event was triggered by pushing the catch pieces out of this position and releasing the spring and reaction mass. The mass and the box then shook with a half life of between 50 to 100 cycles.

Tests with this system by D V Morris (1979) include study of rocking towers with ground shaking. In that study the natural frequency of some of the towers was below and of others was above the ground shaking frequency (tower natural frequency being experimentally observed in wind induced oscillation). The high frequency towers in theory should initially have moved in phase and the low frequency towers out of phase with the ground shaking. What was observed was that all towers initially moved in phase at first, that near the end of the shaking their behaviour accorded with theory, and that the end of their motion was unexpectedly abrupt. Other tests involved saturated fine sand of particle size 0.07 - 0.13 mm at low density of 1860 kg/m^3, in which pore pressures were measured under rocking towers. A study of towers falling over in ground shaking provided experimental support for an analysis by Morris which indicated that quasi-static calculations greatly overestimate the risk of toppling.

Tests on dry sand embankments and on saturated sand embankments led to observation of the cyclically induced pore pressures referred to previously. Those tests were followed by a further series of tests, designed to study the applicability of N M Newmark (1965) analysis to the slipping of clay slopes, which have showed that dynamic slope motions are more complex than is proposed in the sliding block analysis. The experimental investigations have included X ray photography of lead powder threads injected into the clay model during its preparation. Quasi static failure causes a distinct surface of rupture which can be observed because it causes displacement of threads. In contrast base shaking causes deformation in a relatively wide shear band. However when a phase diagram of base acceleration is plotted against observed crest acceleration the experimental observations Figure 2 approximate well to the predictions of a rigid-plastic sliding block with a 90^o phase lag Kutter (1980). There is clear evidence in the phase diagram of a yield acceleration, and of uphill deformation. At the end of the motion crest and base accelerations are in phase and are equal and the phase diagram becomes a line at 45^o. That research continues but is now being transferred from the original shaking box to the new 'bumpy road' system that will be described below.

Meanwhile the original resonant shaking box continues to be used for various tests. In particular, tests on a model made of stacked rings in the middle of the shaking box have been reported by P C Lambe and R V Whitman (1980). Specimens of saturated sand contained within the stacked rings have been observed to liquefy during base shaking. That research project will be continued on the new 'bumpy road' apparatus. Another study in progress in the shaking box concerns backfilled retaining walls, made of micro concrete scaled for n = 80. The aim is to observe ratcheting motions of the wall rotating outwards with plastic yielding of a hinge in the under-reinforced concrete wall. That study is planned to develop into a comparison of the earthquake resistance of backfilled stiff retaining walls with that of reinforced earth models.

The new 'bumpy road' apparatus was designed to meet the following needs; (a) longer embankment slopes and a better view of a model cross section than could be obtained in the shaking box; (b) variable input displacements with a shorter total duration of shaking than was obtained with the leaf springs. The apparatus is illustrated in Figure 3. A steel track (1) extends around about a third of the pit circumference. During the centrifuge flight a wheel (2) passes the track at a speed of about 60 m/s and is generally held clear of it. To cause shaking of the model the wheel is moved radially outward and when it lands on the track it is held firmly in contact. There are very slight 'bumps' on the track and this then forms the bumpy road along which the wheel races at speeds typically of 216 kilometres = 134 miles per hour. The movement of the wheel is determined by whatever waveform has been accurately machined on the track segments - the first track has sine wave profile with half amplitude 2.68 mm. Oscillation of the bell crank (3) causes 'horizontal ground motion' of the model as determined by the track wave form. The double acting pneumatic jack (4) holds the wheel on the track for one pass and then pushes it clear.

The slider (5) and shaft (6) can be adjusted in flight by the screw jack (7) to vary the ratio between the amplitude of motion of the wheel and of the model. There appears to be some dynamic amplification of model motions at certain frequencies, but by appropriate adjustment of the screw jack a model motion of desired amplitude can be obtained. During the motion the centrifugal forces between the model and the centrifuge arm are transmitted radially through linear roller bearings (8). The actuator beam (9) and the crank are mounted on the end of the arm, and on the acuator there is a rack (10). The swinging platform (11) has a matching rack that engaged the actuator rack when the platform swings up at the beginning of the test flight. The model shown in Figure 3 is an embankment, which could for example be a waste heap with continuous seepage from a lagoon at one side to a toe drain at the other side. Steady seepage can be established and the reservoir or lagoon level can be maintained by running a water supply into the model from the centrifuge hydraulic slip rings, and venting seepage flow water into the centrifuge chamber - a standard practice in our tests.

To actuate the mechanism the solenoid valves (13) are operated to allow compressed air to flow from the reservoirs (14) into the double acting jack. The subsequent shaking of the model may be monitored with LVDTs mounted on the model container roof (15), and viewed through the clear polycarbonate side wall of the container. The swing arms offer no resistance to the lateral shaking motion being flexible straps (16) hanging from the torsion bar fittings (17). The force on the model causes a reaction into the centrifuge arm (18) - not present in the original shaking box - but it is now calculated that this load will not reduce the main bearing life. There is also a reaction against the track, but this is carried by the brackets (~~18~~ 19) into the pit wall (20).

The system has now been operational for several months and has been used in three separate periods of successful testing. Soon a full discussion of the design and performance of the system will be included in the thesis of B L Kutter. The system has the ability to test a model of a dam 20 m high with slopes of 1 on 2, founded on a rigid rock base, with steady seepage through flow, subject to an earthquake with frequencies of about 1 Hz. A test series which is also beginning is the observation of water pressures on and in permeable slopes or on a vertical dam face, having say 15 m water depth with a 20 per cent earthquake at 1 Hz. The results of these researches will soon be published in theses.

At the same time in parallel with these dynamic loading tests, quasi static loading problems such as tunnel and trench excavation, and embankment construction, continue to be studied. There is a continual advance in detailed effective stress analysis in quasi static loading so the models which will be the subject of these future ground shaking tests should be of high quality.

It was previously thought that an excessive length of centrifuge running time would be needed to initialise effective stresses at all points in large model tests. However, it now appears that the combination of a small surface preload and an appropriate downward hydraulic gradient field through a model will bring stresses sufficiently close to the required values that very large models could be made and tested within relatively short periods of centrifuge acquisition. In effect it is the magnitude and wave length of whatever perturbations of the stress field exist before initialisation and must be corrected, that determines the time needed for initialisation on the centrifuge, and not the overall pressures and dimensions of the model. Enough is known about the behaviour of reconstituted soil in geotechnical models to make available to any group requiring research associated with national needs or private interests, the option of conducting tests on much larger models than those which have been discussed in this review.

CONCLUSION

This state of art review discusses the scaling principles for dynamic and earthquake geotechnical centrifuge models that have been known for many years, and shows how the new emphasis on modelling of models using reconstituted soil helps to reduce the difficulties of verification of those principles.

It is suggested that models made of reconstituted soil can exhibit a wide range of behaviour that is important in the field. In particular it is suggested that liquefaction and cracking are related phenomena which can both be modelled. Dynamic events such as cratering and earthquakes are considered, the facilities for shaking models are described, and the instrumentation specified.

It is to be expected that in the future a wide variety of dynamic and earthquake model tests will be conducted on geotechnical centrifuges, and that these tests will provide an abundant source of data, relevant to design of prototype works and analysis of damage caused by strong ground motions.

ACKNOWLEDGMENTS

The writer wishes to acknowledge the role of his colleague, Dr R G James, Assistant Director of Research, in the construction and development of the Cambridge Geotechnical Centrifuge and in particular in the development of dynamic test capabilities and comments on this review. The Design Engineer of Cambridge University Engineering Department, Mr P W Turner, made the design of these facilities, and has greatly assisted others in design of components for which they have been responsible. Two research students, Dr D V Morris followed by Mr B L Kutter, have made major contributions to these developments and experiments.

We have been helped and encouraged throughout this period of development by advice and comment from Professor R V Whitman, both when a visitor to Cambridge and also in continued contact from M I T. A research contract for centrifuge study of environmental problems of liquefaction from the European Research Office of the U S Army Corps of Engineers, and close continued contact with staff of their Waterways Experiment Station have provided support and helpful contacts. The US National Science Foundation, as part of a research contract with Professor Whitman, recently has paid a share of the cost of development and of operation of the 'bumpy road' shaker system. However, in general, over the past fifteen years the funds for centrifuge developments in the UK have been raised from UK sources and our research has been directed towards meeting UK research requirements.

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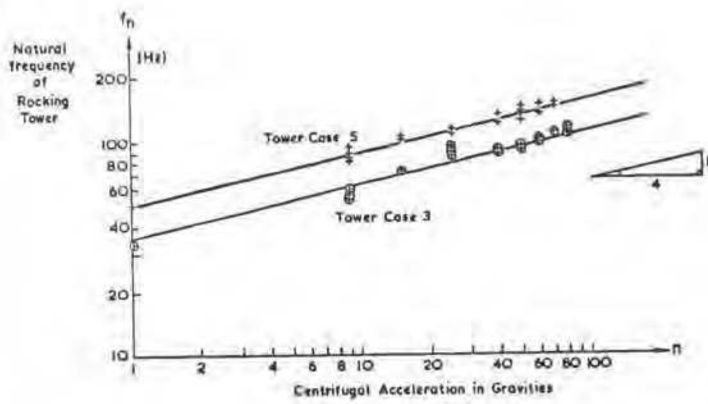


Figure 1. Observed variation of natural frequency of model rocking towers with centrifugal acceleration (after D V Morris (1979)).

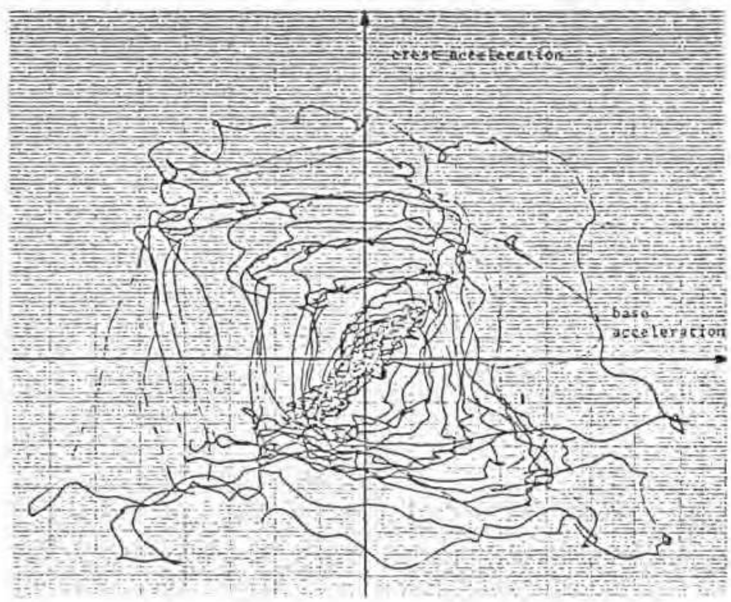
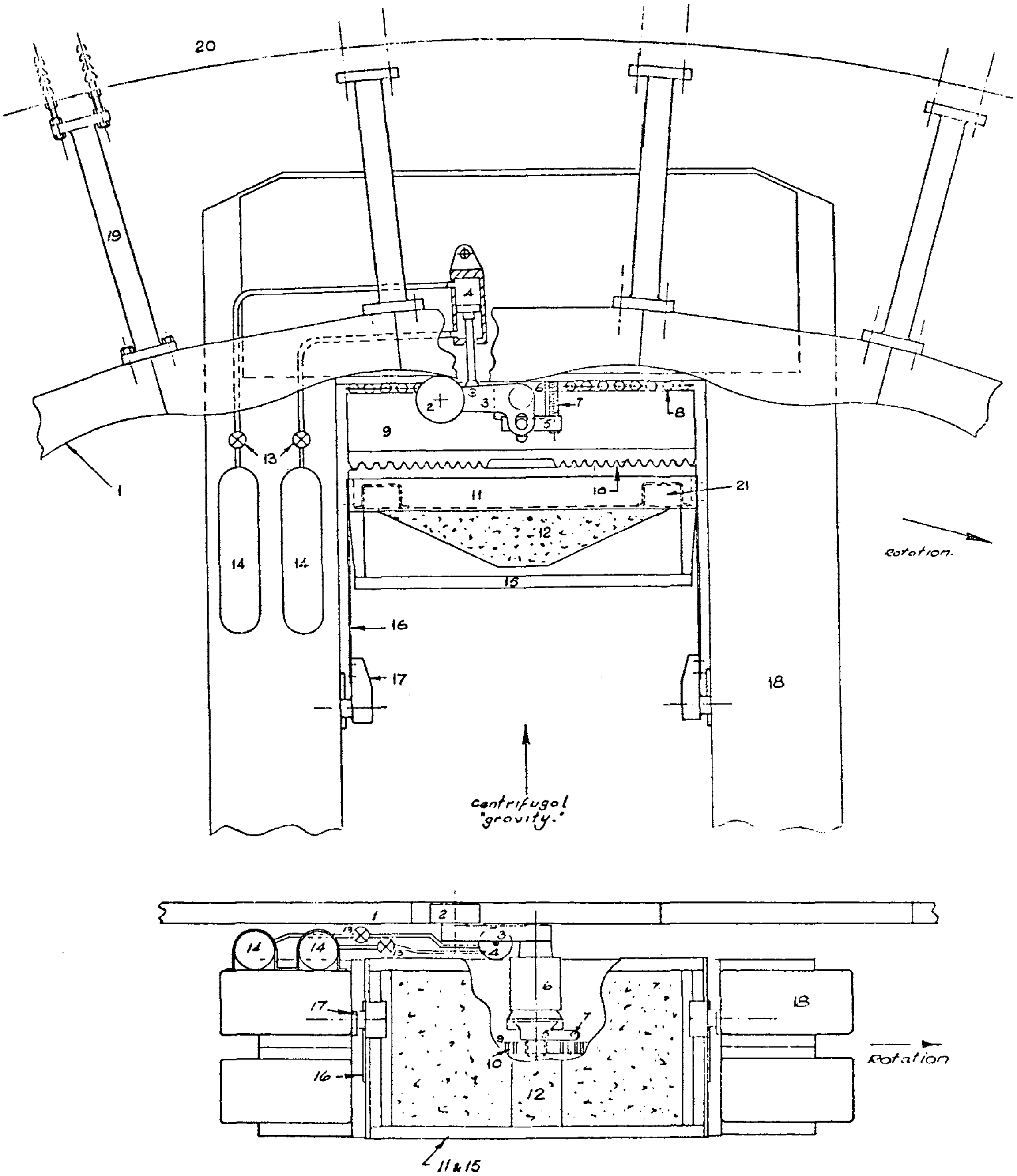


Figure 2. Observed crest and base acceleration phase diagram during shaking of model clay slope (after B L Kutter (1980)).



Key

1. track
2. wheel
3. crank
4. pneumatic jack
5. slider
6. shaft
7. screw jack
8. bearings
9. actuator
10. rack
11. platform
12. model
13. valves
14. reservoirs
15. roof
16. straps
17. torsion bar
18. arm
19. brackets
20. wall

Figure 3. The 'bumpy road' shaker on the Cambridge Geotechnical Centrifuge.

Presenting the above paper, the following contribution was added by A N Schofield:

DISCUSSION ON LIQUEFACTION

Several words including 'critical' or 'stable' or 'liquefied' have been used in soil mechanics for forty or fifty years with meanings that have undergone subtle changes as an ever expanding framework of theory has accommodated new experimental results.

The difficulty with the original notion of a constant critical void ratio, depicted by the line BD in Figure 1(a), can be seen by considering the work done in the anti-clockwise cycle ABCD. Work done in shearing soil could be extracted during volume changes, if soil could dilate AB when sheared under high pressure and collapse CD when sheared under low pressure. In contrast in Figure 1(b), a critical state line SU sloping more steeply than elastic compression lines avoids this difficulty. In the clockwise cycle RSTU, when soil is sheared TU external work is done both in distortion by shearing forces and in plastic compression under high effective pressure. When soil is sheared RS under low effective pressure the soil ruptures on a thin shear zone, minimising the work emitted by the soil during dilation. Experimental evidence strongly supports the notion of critical states(v,p') satisfying an equation:

$$v + \lambda \ln p' = \Gamma.$$

However, many geotechnical engineers still use the word 'critical' in the incorrect original sense of a critical void ratio independent of pressure.

Instead of using words 'more loose than critical' or 'more dense than critical' in Figure 2(a) it becomes necessary to use words like 'above' or 'to the right' or 'below' or 'to the left' of the sloping line of critical states. The words 'wet' or 'dry' were introduced by Roscoe and Schofield (1963) to impart the understanding that soil yielding in 'wet' states tends to discharge water and soil rupturing in 'dry' states tends to suck it up. To be more precise in Figure 2(b) the word 'wet' was applied to soil yielding in states defined by the line BC, and the word 'dry' was applied to soil rupturing in states defined by the line AB, but the general behaviour of soil in states inside the area OABC in Figure 2(b) was considered elastic at that time, even though it was appreciated that there would be significant in-elastic hysteresis in supposedly 'elastic' stress cycles. Cyclic behaviour has recently been clarified by triaxial tests of Luong and Sidaner (1981), summarised in Figure 2(b) for cyclic tests of soil all at the specific volume indicated by ABC in Figure 2(a). Luong defined a 'characteristic' stress ratio as that ratio at which a stress cycle produced no change of stress or volume; this characteristic ratio has more or less the same value as the ultimate critical stress ratio at which continuous deformation produces no change of stress or volume. The tests showed that cyclic loading in I the subcharacteristic region OBC in Figure 2(b) caused a discharge of water or a fall of p', and in II the surcharacteristic region OAB in Figure 2(b) it caused water to be sucked in or an increase of p'. Clarification of this point is a development that enriches the significance of Figure 2, without in any way conflicting with the notions of yield in wet states and rupture in dry states at the boundaries of regions I and II, where strength is the question at issue.

In dry states at a point such as F in Figure 3(a) the peak strength of soil involves a contribution from the dilatancy of the interlocked stressed particles. In the long term the dilating gouge material on slip planes will soften, FG in Figure 3, but suction can persist for many years provided the soil does not fissure or crumble. In contrast, in wet states soil yields with plastic compression. The work done by external (cell) pressure in a drained triaxial test, or the energy released by the effectively stressed soil during unloading in an undrained test, reduces the work the deviator stress needs to do against internal friction. The yield strength therefore falls as the effective stress rises, BC in Figure 3(a). Whatever path is taken by soil from the wet or dry side, once it is flowing in a critical state B or G the soil ultimately has only the strength of a frictional material. The suggestion in Critical State Soil Mechanics by Schofield & Wroth (1968) was to identify the behaviour of soil yielding in wet states with the short term stability problem, and soil rupturing in dry states with the long term stability problem. In one case the ultimate critical state strength would correspond to a stability analysis with $c=c_u$, and in the other case with $\phi=\phi_d$. The characterisation of soil as 'cohesive' or 'frictional' could not therefore be regarded as a fixed soil property, but depended on the state of stress and the specific volume of the soil. And although the undrained stability problem of wet soil was clearly the problem of failure immediately on loading, it proved to be incorrect to infer that all sudden failures or flowslides were evidence of soil having been in wet states at the time of failure.

When a continuous soil body is unloaded, following a stress path in Figure 3(a) which intersects the line OA, it can disintegrate into a clastic body with a sudden increase of permeability. In that case the average specific volume of the clastic mass can increase greatly in a very short time. Whenever soil is dug or is crumbled, for ease of handling or for mixing with water, an unloading stress path reduces a principal effective stress component to zero in a controlled manner. If there is a hydraulic gradient across the soil body at the time that it cracks or crumbles the event is less controlled, and has the character of sudden hydraulic fracture or fluidisation. The effective stress may be reduced towards zero by imposing tensile strain, or by increasing pore water pressure, or by cyclic loading leading to relaxation of stresses between particles. In each case the soil particles remain geometrically interlocked with each other even though the effective stress falls. In this class of unloading path if at any stage a large shear distortion were to be imposed on the interlocked but lightly stressed particles, they would respond by dilation, with increase of effective stress, and the effective stress path would head back towards the critical state line BG in Figure 3(a). But when in Figure 3(a) the effective stress path reaches OA the continuum begins to disintegrate with unstressed particles sliding apart.

The word 'interlock' is helpful in thinking about this event. A lock is made with a sliding bolt which is mechanically secure even though it is unstressed. Any attempt to shear a pair of locked doors apart will generate resistance by the bolt. However, if a pair of interlocked doors are pushed open their lock or bolt will offer no resistance as the doors swing apart. In the same way a hydraulic gradient across a soil body at very low effective stress will open cracks or channels in the soil. In the case of a pair of locked doors where the bolts are jammed tight, it may be possible to free a bolt by joggling it in different directions until the forces that hold the bolt are relaxed. In the same way cyclic loading on interlocked soil particles can reduce the effective stresses between them until even the closest fitting interlocked particles become free to slide apart.

The opening within the body may be an extensive crack or a local pipe or channel. In the case of a local pipe being formed, the seepage forces of water following tortuous paths round interlocked particles may be able to dislodge particles in a direction perpendicular to the axis in which the pipe is developing: the hydraulic pressures transmitted along the pipe would form hydraulic gradients in these directions. So in general the dilative response, that makes interlocked particles brace themselves to resist shearing deformation in soil in dry states and makes them suck water in among themselves as they dilate, turns into a disintegrative response in soil in which effective stress components fall to zero. In some cases the cracks are self healing with soil particles from the walls of cracks forming a mud which limits the speed of pressure transmission. In other cases there can be sudden transmission of pressures, and a body of crumbling soil can disintegrate into a sort of soil avalanche, or several pipes can break through a sand layer and vigorous sand boiling can occur.

The word 'liquefaction' has been linked with two different notions. In one notion the state of soil before liquefaction is to the right or above the critical state line in Figure 3(b), possibly even being a metastable state defined by a point to the right of line BC in Figure 3(a), and events called liquefaction are considered to propagate by a chain reaction. In the other notion the state of soil before liquefaction moves to the left almost to zero effective stress, to the dashed line through A in Figure 3(b), far more dry than and below the critical state line, and events called liquefaction are linked with hydraulic fracture or sand boiling. The second notion appears to fit the macroscopic phenomen reported by Z Q Wang (1981).

Consider in Figure 4(a) the deposition of a column of fluidised sand as a hydraulic fill. At first (i) there is zero effective stress $u = \gamma z$ but soon (ii) the lower part of the column has settled and as the upper fluidised part F continues to settle the whole column (iii) has vertical effective stress $\sigma' = (\gamma-\gamma_w)z$ and pore pressure $u=\gamma_w z$. Next let the base of the column be shaken gently so that at every point in the column some deviator stress is applied with stress ratios in the contractive zone I of Figure 2(b). The effective pressure p' everywhere will fall and the particles will begin to settle. There will be least settlement at the bottom of the column, but higher in the column the rate of settlement will accumulate. There may be a point at which the rate of settlement equals the settling velocity of hydraulic fill in which case (iv) the upper part F of the column becomes fluidised. At that stage the isochrone of pore pressure - which corresponds to the settling rate - may continue to move to the right and (v) the depth of fluidised soil may increase, but soon there will be a point at which the rate of settlement of the lower part of the column is less than the settling velocity of hydraulic fill. Thereafter (vi) the lower part of the sand column will be rather more dense than it was before, but the upper part of the column - settling as hydraulic fill - could well be refluidised by subsequent shaking. In Figure 4(b) an extra layer of overburden is shown. Any layer of low permeability will be brought to zero effective stress relatively early in the process of settlement, and in Figure 4(b) this layer is shown cracked and with fluidised sand rising within the clastic mass. This is called clastic liquefaction, and if there is a slope the mass can begin to move as a buoyant debris flow. In Figure 4(b) the sand above the liquefaction front is fluidised F and below the front is still contractive. Strong shaking that brings sand into the dilative region II of Figure 2(b) would affect the shape of the isochrone but would not affect the general picture. In general, the discharge of water from underlying contractive sand through overlying dilating soil is associated with the fluidisation of the overlying soil in states near zero effective stress. Exactly the same effect could be produced by a fracture of a buried water main. The overlying soil would be fluidised by the discharge of water through it. In the case shown in Figure 4(b) the settlement of the contractive sand is enough to cause the settling stiff overburden to exhibit phenomena generally called 'liquefaction'.

In centrifuge model tests the mechanism of fluidisation of a sand column when shaken at the base, in the manner shown in Figure 4(a), can be confirmed by observations of pore pressures with transducers located at various depths. The stability of fluidised beds has been the study of chemical engineers for many years, and centrifuge model tests are only newly beginning, so there is much to learn from previous research as well as from the new model tests. Many tests are possible. For example it might be possible to reproduce the phenomenon of the massive talus avalanche in the European Alps. A mass of rather dense scaled-down talus might have a model brook flowing through voids in its interior. The interior could become contractive and the upper part dilative - the upper part could also be strained laterally and crack. The brook might then flow into the cracked upper part of the model, and cause a model avalanche. By observation of models and comparison with field records it may be possible to link field events with the notion of liquefaction as a phenomenon of soil near zero effective stress in the presence of a hydraulic gradient. The earlier notion of liquefaction was of an event propagating by chain reaction through a meta-stable body of collapsing sand. If that truly occurs in nature then it should also be possible to reproduce it in centrifuge models, although as yet this has not proved possible.

If it were possible to prove that all reported cases of liquefaction were phenomena near zero effective stress in the presence of a hydraulic gradient this might enable definite advice to be given on design of foundations to resist liquefaction. The advice might be to increase effective stresses in, and to reduce the possible discharge of water through, the body of soil at risk. For example, in the case of an offshore gravity platform of 150 x 150 m^2 plan area in 150 m water depth it might be possible by underdrainage to generate a vertical load by suction anchorage which could be four times the weight of the platform. There are various patents relating to skirts and drainage systems which might enable this great increase of load to be effective: the problem of liquefaction might then be replaced by the problem of settlement. However at present much basic research on mechanisms of liquefaction remains to be done, and this is a major objective of current centrifugal model tests.

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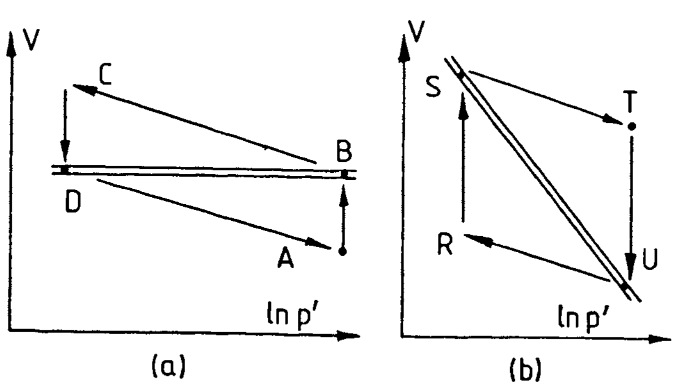

Figure 1

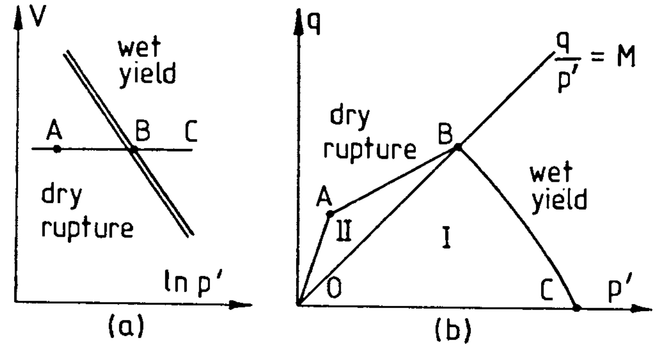

Figure 2

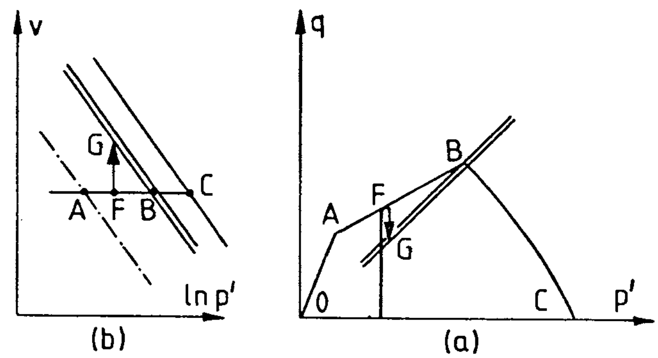

Figure 3

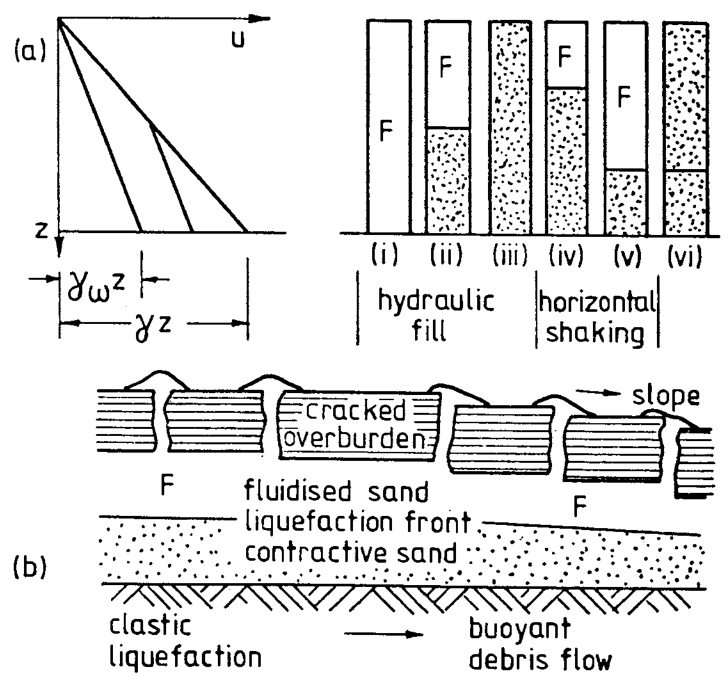

Figure 4