2

GROUND DISPLACEMENTS IN CENTRIFUGAL MODELS

DEPLACEMENTS DE TERRAIN SUR MODELES CENTRIFUGES

ИССЛЕДОВАНИЕ ДЕФОРМАЦИИ ГРУНТА НА МОДЕЛЯХ В ЦЕНТРИФУГЕ

M.D. BOLTON, Lecturer
R. ENGLISH, Research Student
C.C. HIRD, Research Student
A.N. SCHOFIELD, Professor, University of Manchester Institute of Science and Technology (UMIST), England

Proceedings of the Eighth International Conference on Soil Mechanics and Foundation Engineering, Moscow 1973.
Comptes rendus du Hutieme Congres International de Mecanique des Sols et des Travaux de Fondations, Moscou, 1973.
Труды восьмого международного конгресса по механике грунтов и фундаментостроению, Москва, 1973.

GROUND DISPLACEMENTS IN CENTRIFUGAL MODELS

DEPLACEMENTS DE TERRAIN SUR MODELES CENTRIFUGES

ИССЛЕДОВАНИЕ ДЕФОРМАЦИИ ГРУНТА НА МОДЕЛЯХ В ЦЕНТРИФУГЕ

M.D. BOLTON, Lecturer
R. ENGLISH, Research Student
C.C. HIRD, Research Student
A.N. SCHOFIELD, Professor, University of Manchester Institute of Science and Technology (UMIST), England

SYNOPSIS After reviewing the development of modelling with the UMIST centrifugal test facility the paper considers two different cases where problems of modelling the interaction between soil and the adjacent material could have made model behaviour unrealistic. However in neither case did the interaction at the interface pose difficulties, and interacting three dimensional effects were evident which could only otherwise have been produced at considerable cost in construction at prototype scale. At present the accuracy of measurements in the UMIST facility is not sufficient to allow comparison with ground displacement of some prototypes before failure.

INTRODUCTION

The Proceedings of the 7th Conference (1969) included papers by Ter-Stepanian and Goldstein, Mikasa Takada and Yamada, and Avgherinos and Schofield, which referred to current work on centrifugal testing of plane model slopes. Continuing work on plane sections by Avgherinos (1969) and Endicott (1970) was later described by Roscoe (1970). A different centrifuge for testing three dimensional models that was newly built at UMIST in 1969 was also illustrated and described at the end of Speciality Session 16 of the 7th Conference; subsequent work on this centrifuge will now be described briefly below.

The initial tests at UMIST were to see if large blocks of undisturbed soil could be sampled and formed into simple but realistic models of prototype cuttings, foundations and slopes. A grant from the Wolfson Foundation made possible collaboration between UMIST and George Wimpey Limited in a programme of tests, some of which have been described by Lyndon and Schofield (1970) and the rest of which will be described in Lyndon's forthcoming thesis. Another programme of tests on end effects in compacted clay slopes was undertaken in the UMIST centrifuge by Craig (1970) and Fuglsang (1971) in preparation for the recent construction of a second centrifuge in the University of Manchester described by Rowe (1972).

In all this work the broad validity of the following modelling law has been well confirmed: if events and measurements which refer to prototype construction are given suffix p, and events and measurements of a geometrically similar model formed of the actual prototype material are given suffix m, then when

$$(\text{Linear scale})_p = N\,(\text{Linear scale})_m$$

and

$$N\,(\text{Acceleration})_p = (\text{Acceleration})_m$$

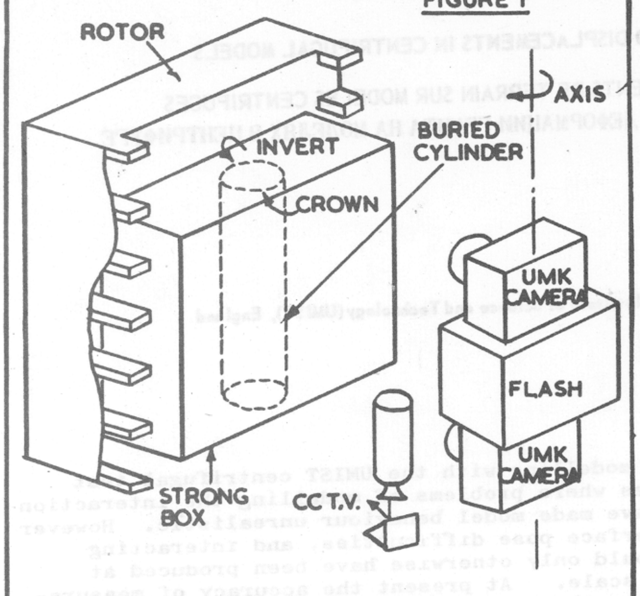
the model comes into similarity with the prototype, and events such as displacements of boundaries due to primary changes of pore water pressure occur at times

$$(\text{Time})_p = N^2 \cdot (\text{Time})_m .$$

By replicating prototype stresses in a block of the actual prototype material the centrifugal model provides a realistic demonstration of the behaviour of the prototype even when many conventionally independent modes of behaviour are interacting. Such is the case when the stability of earthworks is controlled by consolidation characteristics, and when these interactions or the three-dimensional geometry of the boundaries create a problem of analysis that is outside the present bounds of theoretical solution.

The centrifugal testing facility at UMIST illustrated in Figure 1 has a rotor of effective radius 1.5 m rotating at speeds up to 285 r.p.m., generating accelerations in the vicinity of the soil models of up to 135 times gravity. Top speed can be reached in $3\frac{1}{2}$ minutes from start. Test boxes may be 850 mm x 850 mm square on plan and may have

FIGURE I



VIEW ON CENTRIFUGE, CUT AWAY TO REVEAL A BURIED FLEXIBLE CYLINDER.

a mass of 750 kg. Long periods of consolidation in the centrifuge have had to be avoided, but early hopes that specimens could generally be prepared following Zelikson's use of a downward hydraulic gradient were not realised when cracking occurred and flow became non-uniform. Strong press frames were then made in which the specimen surface is loaded by a large inflated butyl rubber bag during preliminary consolidation for a typical period of two weeks which is followed by a brief period of only one or two days' consolidation in the centrifuge. A television camera rotating with the model continuously monitors the general appearance of its surface, and intermittent but much more detailed photogrammetric measurements of the topography are obtained by flash photography. A stationary but high speed flash of the air-spark type provides 100 joules in 2 micro seconds which illuminates the moving model briefly while photographic glass plates are exposed in two stationary Zeiss UMK cameras which are then raised along the axis of the rotor in order to change the glass plates. In addition, pore pressure transducer and other measurements which can be transmitted from the model via slip rings as electrical signals are continuously monitored and recorded in the external console which also contains the T.V. monitor and a video tape recorder. In several sets of tests interest has centred around the accuracy of modelling displacements of ground. Our photogrammetric measurements define the surface of the model to $\pm$ 1/3 mm. In a typical test at 60 gravities this corresponds to levelling to an accuracy of only $\pm$ 20 mm. in the prototype. Since prototype observations may well be much more accurate than this the question arises of what we can learn from centrifugal models of problems such as slurry trenches, or of problems of interaction such as a buried flexible pipe, where rather small ground displacements correspond to major changes in the interacting structure.

THE SLURRY TRENCH PROBLEM

Bentonite slurry is able to seal the walls of a trench by the formation of an impervious boundary layer which permits the hydrostatic pressure of the slurry to be exerted as a total stress on the adjacent soil. The interaction of slurry and soil at the interface is a complex phenomenon dependent upon the type of soil encountered. In clay, penetration of the soil by the slurry is generally small, but with time water lost from the bentonite during the formation of a "filter cake" enables a narrow zone of the surrounding clay to swell and soften. Beyond this, dissipation of the excess pore water pressures (suctions) brought about by trench excavation takes considerably longer. Thus in general the hydrostatic pressure of the slurry on one side of the filter cake is balanced by changing pore-water pressures and effective soil stresses on the other. However, where slurry trenches are excavated sufficiently quickly that substantial changes of effective stress around the trench do not occur whilst the trench is open, and an undrained analysis is appropriate, a centrifugal model test may be carried out sufficiently rapidly to incorporate all the major features of a full scale slurry trench in clay soil. The initial pore pressures and strength profile for a clay site may be produced by consolidating the model in the centrifuge before excavating the trench.

The series of undrained tests at UMIST in which the three dimensional behaviour of slurry trenches in clay has been observed was of a trial nature, and as yet no attempt has been made to model the pore-water pressures and strength profile of a particular site. In analysing the results of the tests an undrained analysis with constant shear strength throughout the model has been adopted.

Each model was prepared by one-dimensional consolidation of a Kaolin clay slurry in the actual test box to give a uniform clay sample in which excavation of one or more trenches could be carried out. Each model was then subjected to a rapid test, acceleration being increased in increments of approximately 10 gravities until collapse of the trenches occurred. Various "slurries" were used to support the trenches. A typical model configuration is given in Figure 2 and a summary table of tests in Table I.

TABLE I: SUMMARY OF SLURRY TRENCH TESTS

Test No.	Soil Parameters: Mean Moisture Content	Predicted Strength (kN/m^2)	Mean Vane Strength (kN/m^2)	Ref.	Excavation (mm): Length L	Width B	Depth D	Slurry: Type	Density	Failure Data: G_1	G_2	$\frac{G_2}{G_1}$
1	33.4	22.5	20.8	A	508	54	127	Water	1.00	78	90	1.15
2	33.4	22.5	21.1	B	178	76	178	Oil	0.88	50	75	1.50
				C	178	76	178	Sugar	1.20	71	93	1.31
3	33.2	23.5	22.5	D	152	38	152	Bent-	1.04	71	95	1.34
				E*	102	51	152	onite	"	71	110	1.55
				F	76	76	152	"	"	71	122	1.72
				G**	76	diam	152	"	"	71	125	1.76
				H	76	38	152	"	"	71	125	1.76
				I***	38	38	152	"	"	71	130+	1,83+

* L shaped excavation
** Circular excavation
*** Not fully collapsed at 130 g

G_1 = Gravities required for failure as predicted by Nash and Jones.
G_2 = Gravities at actual failure ($\pm$ 5 g)

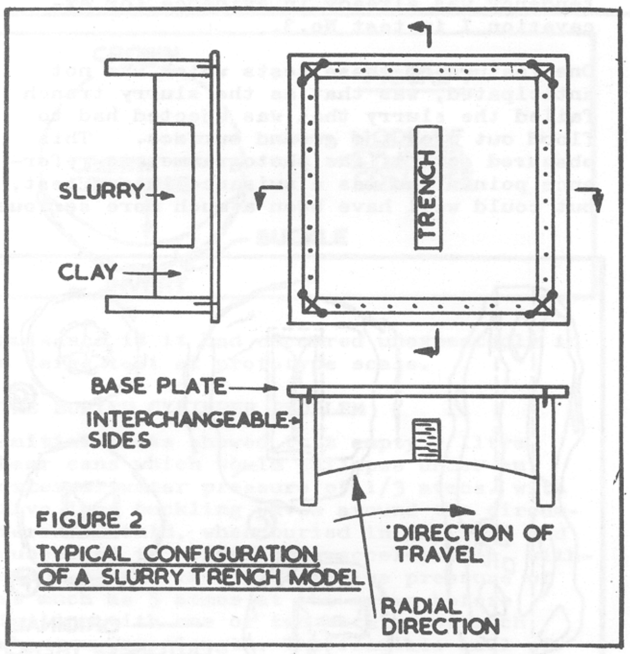


FIGURE 2
TYPICAL CONFIGURATION OF A SLURRY TRENCH MODEL

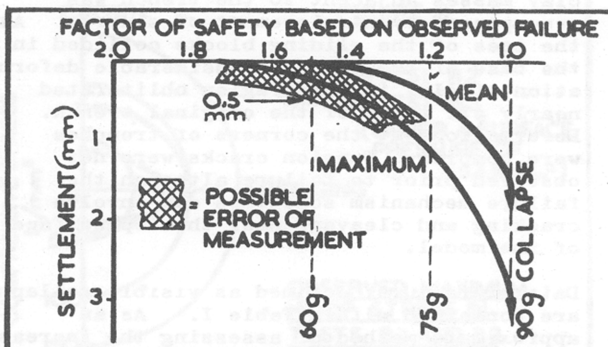


FIGURE 3 SETTLEMENTS AROUND A SLURRY TRENCH
TEST No 1 EXCAVATION A

A series of consolidated undrained 38 mm diameter triaxial tests were carried out on samples taken from the models to establish the correlation between moisture content and undrained shear strength. After each test the uniformity of the model was checked by laboratory vane strength measurements and moisture content determinations which verified that the tests could be regarded as undrained.

RESULTS OF MODEL TESTS ON SLURRY TRENCHES.

(1) Surface deflections prior to failure.

In recent Norwegian Geotechnical Institute (NGI) tests on a number of slurry trenches in Oslo accurate measurements of ground displacements prior to failure have been made both in respect of surface settlements, found to be less than 10 mm., and horizontal deflections of the trench walls at depth. Measurements made by optical level to an accuracy of $\pm$ 1.0 mm were reinforced with precision gauge measurements accurate to $\pm$ 0.01 mm. in the prototype, and clearly no direct comparison between these results and those of centrifugal model tests is possible. Moreover, the modelling of deep and narrow trenches such as those tested by NGI would involve model trenches having typical dimensions 10 mm x 50 mm x 280 mm deep for which modelling techniques have yet to be developed. So a series of altogether larger and squatter trenches had to be chosen for our model tests. Also, because of elastic compression of the clay mass on application of high accelerations, a certain overall settlement of the clay model was observed and therefore interpretation of results has been based on the differential settlement of points adjacent to and points well away from the trench.

Settlements for the long trench of test No.1 are shown in Figure 3. Below 60 gravities when the factor of safety based on the observed failure was above 1.5, settlements were negligible. At higher accelerations, settlements increased in magnitude reaching about 2 mm in the model (or 170 mm at prototype scale) just before total collapse of the trench sides. Had this model represented the ground conditions at a particular site, the test indicates that a prototype trench 30.5 m long, 3.3 m wide and 7.6 m deep could be excavated in that ground using a water slurry, with settlements adjacent to the trench of between 0 and 30 mm and that, once constructed, the trench would have a factor of safety against failure of about 1.5, assuming that an undrained analysis was appropriate.

For the trenches of tests 2 and 3 with smaller length to depth ratios, settlements could not be detected until the factor of safety based on observed failure fell below approximately 1.2. With such trenches at prototype scale there would be little warning at the surface of imminent failure.

(2) Data of failure.

Collapse of the trenches was observed in which initially a rotational slip of the clay masses adjacent to the trench was accompanied by displacement of slurry. As the toes of the sliding blocks collided in the base of the trench, considerable deformation of clay in this region obliterated nearly all trace of the original trench. Deformations at the corners of trenches were complex. Tension cracks were not observed prior to failure although the failure mechanism sometimes did involve cracking and cleavage near the top surface of the model.

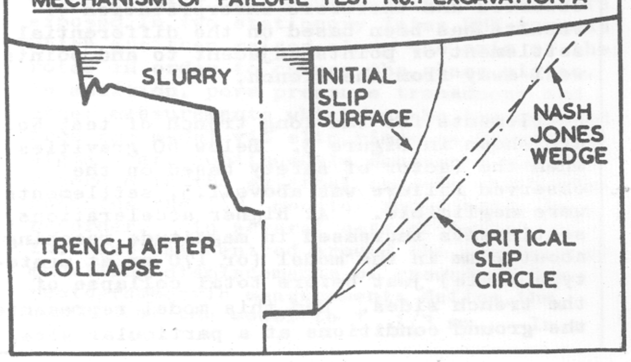
Data of failure, defined as visible collapse, are contained within Table I. As an approximate method of assessing the increased stability of the three-dimensional case over that of the plane strain case, failure points have been predicted for the latter by Coulomb Wedge theory as used by Nash and Jones (1963). These will usually be slightly in excess of the values obtained by slip circle analysis.

In Figure 4 the mechanism of failure for the long trench of test No.1 may be compared to the Nash-Jones wedge and the most critical slip circle. The steepening of the initial slip surface towards the surface of the model, a characteristic of all the tests, was indicative of the restraining influence of the ends of the trench and may also have been associated with the development of a tensile zone near the trench due to displacements at the instant of, or immediately prior to, collapse. The influence of trench shape on the failure mechanism is shown in Figure 5 progressing from the almost plane strain case of a long trench to the truly three dimensional case of a circular excavation. The similarity of the mechanisms for the square and circular excavations should be noted.

Figure 6 illustrates the dominant influence of length/depth ratio on the stability of the trenches. It is also to be expected that as the length/depth ratio is decreased below the range explored in these tests, there would be a tendency towards the failure mechanism observed by NGI where closure of the trench at depth was accompanied by only very small surface settlements. This tendency was already in evidence for excavation I in test No.3.

One feature of these tests which was not anticipated, was that as the slurry trench failed the slurry that was ejected had to flood out over the ground surface. This obscured some of the photogrammetric reference points and was a nuisance in one test, but could well have been a much more serious

FIGURE 4
MECHANISM OF FAILURE TEST No.1 EXCAVATION A



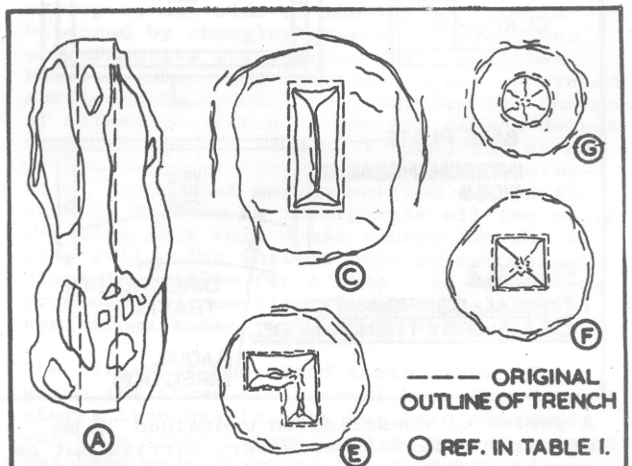


FIGURE 5
SKETCH OF SLURRY TRENCH FAILURES

FIGURE 6
INCREASED STABILITY OF THREE DIMENSIONAL SLURRY TRENCHES

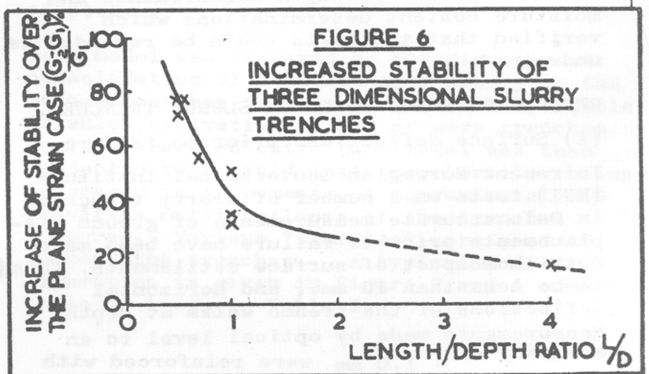


TABLE II: CENTRIFUGAL TESTING OF BURIED FLEXIBLE CYLINDERS PRIOR TO APRIL 1972

Test No.	Relative Density	Prototype Modelled Diameter (m)	Wall (mm)	Crown Cover (m)	Description
A 1-6	Low	Various			Beer cans, various cylinders
A 7	High	13.2	11.2	12.1	Helically wound cylinder
A 9	High	7.5	6.2	6.7	" "
B 1	90%	4.7	3.1	3.6	Vertical diameter "
B 2	89%	13.3	8.5	5.9	1.3% greater than horizontal

TABLE III: PROTOTYPES MODELLED IN TEST B2

Number of Gravities	Diam (m)	Cover to Crown (m)	Wall (mm)	Comments
20	3.1	1.4	1.9	Stable.
40	6.1	2.7	3.9	Shell ripples possibly appear at the crown.
90	13.8	6.1	8.8	Buckled with post buckling stability evident.
98+	15.0+	6.6+	9.6+	Crown collapse.

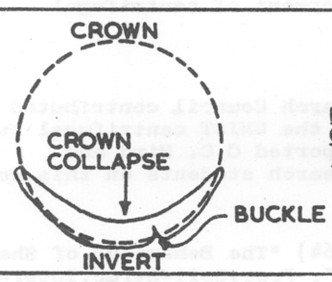


FIGURE 7

MODE OF CYLINDER COLLAPSE

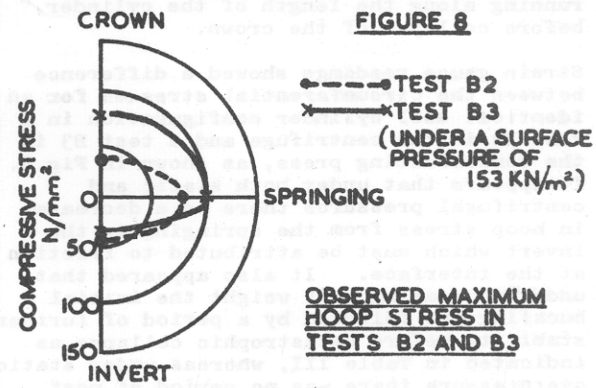


FIGURE 8

OBSERVED MAXIMUM HOOP STRESS IN TESTS B2 AND B3

nuisance if it had occurred unexpectedly in a large test at prototype scale.

THE BURIED CYLINDER PROBLEM

4826

Initial tests showed that empty 2 litre beer cans which would collapse under an external water pressure of 1/3 atmos. with five long buckling waves around the circumference would, when buried in dry sand and subjected to centrifugal acceleration, withstand a centrifugal effective pressure of as much as 5 atmos at the crown before failing with one or two buckles of much shorter wave length. The flexible wall of the buried can clearly acts in the manner of a beam on an elastic foundation for which the wave lengths of the buckles decrease with increase of the relative stiffness of the medium, thereby allowing the can to resist higher external pressures before collapse. The high resistance of buried flexible structures to loads applied at the ground surface has been the subject of review by Allgood (1964).

In actual construction of underground flexible arches selected back-fill is carefully compacted around the structure with the intention that any small lateral displacements will be reduced by the mobilisation of significant "arching" stresses within the soil. Clearly the full analysis of this problem must involve not only the initial curvatures of the structure, densities of the soil and the coefficient of friction between structure and soil but also the rates of changes of density and stiffness of the soil as the interface begins to move. Many tests have been reported of cylinders with various flexural stiffnesses buried in various soils when subjected to static or dynamic over pressure on the ground surface; no reported tests have involved significant self-weight of ground. We decided that in our initial tests we would uniformly place standard sand to bury at various depths some standard flexible steel cylinders fabricated from shim steel. Long strips of shim steel were available, 102 mm wide and 0.102 or 0.127 mm thick which were helically wound round cylindrical wooden formers of 102 mm and 152 mm diameter with a 2 mm wide spiral overlap joint which was soft soldered. The sand had $d_{60} = 0.73$ mm $d_{10} = 0.30$ mm, $e_{max} = 0.75$, $e_{min} = 0.44$ $\phi = 44^{\circ}$, $G_s = 2.65$, and $K_o = 0.48$ at 90% relative density. We stood the box with the cylinder axis vertical when the sand was poured at a rate of 1 kg/min through a flexible rubber hose falling 3 m from a hopper suspended overhead to achieve that relative density. Each cylinder was placed with its 670 mm length standing upright in the test box parallel to the centrifuge axis as shown in Figure 1. Foil strain gauges on the buried steel cylinder gave readings via the rotor sliprings on a strain meter, which indicated the development of buckling and instability

In addition to centrifugal tests on these buried cylinders, tests under static overpressure were made in the clay consolidation press. Pressures of up to 280 kN/m^2 were applied by an air pressurised butyl rubber bag which pressed on the sand surface as the box sat in the press on the laboratory floor

RESULTS OF MODEL TESTS ON BURIED FLEXIBLE CYLINDERS.

In the summary of the tests up to April 1972 in Table II the dimensions of the corresponding prototype structure are calculated simply by application of the modelling law. The prototype structures modelled are much more flexible than any used in current practice. A typical failure as shown in Fig.7 had one or two short wavelength buckling creases running along the length of the cylinder, before collapse of the crown.

Strain gauge readings showed a difference between the circumferential stresses for an identical soil cylinder configuration in a test B2 in the centrifuge and a test B3 in the consolidating press, as shown in Fig.8. It appears that under both static and centrifugal pressures there is a decrease in hoop stress from the springing to the invert which must be attributed to friction at the interface. It also appeared that under increasing self weight the initial buckling was followed by a period of further stability before catastrophic collapse as indicated in Table III, whereas under static overpressure there was no period of post buckling stability before the crown collapsed.

CONCLUSION

At the present stage of development the centrifuge can be used to produce three dimensional failures such as those listed in Tables I and II at a fraction of the cost that would have been involved in trial construction at prototype scale. Apart from enabling the assessment of the relative safety of alternative solutions to a given design problem, centrifugal model tests indicate realistically the mode of failure, and its consequences are dramatically demonstrated on closed circuit television. One indication of the factor of safety against failure of a project is the factor by which the size of the scaled up model at failure exceeds the size of the prototype, simply observed by causing the speed of the machine to rise above the correct modelling speed until failure occurs. This factor of safety is similar in kind to that employed in an undrained stability analysis. If the centrifuge is now felt to test the actual soil from site in a manner which eliminates the feeling of uncertainty regarding soil strength, the remaining design uncertainties would be associated with such factors as water movement. In order to be able to probe into these areas of uncertainty a water movement system is being developed for the UMIST centrifuge.

Measurement techniques in the model such as photogrammetry, strain gauging and pressure transducers already allow an interpretation of the relative importance of the various types and locations of measurement required to investigate a particular problem at prototype scale. It is clearly necessary from the tests described for us to supplement the general photogrammetric system with additional linear displacement transducers to measure small ground displacements.

Whilst much of the value of centrifugal testing lies in its complete independence from mathematical or computer models, it is expected that accurate deformation measurements such as those suggested above will promote a fertile interaction between digital computer finite element models and the continuing development of centrifugal analogue models.

ACKNOWLEDGEMENT

The Science Research Council contributed half the cost of the UMIST centrifugal test facility and supported C.C. Hird and R.English as research students on this work.

REFERENCES

ALLGOOD, J.R.(1964) "The Behaviour of Shallow Buried Cylinders". Proc.Soil-Struc.Inter-Action Symp.Univ.Arizona, p.189.

AVGHERINOS, P.J. (1969) "Centrifugal Testing of Models made of Soil" Ph D Thesis, Cambridge univ

AVGHERINOS P.J. & SCHOFIELD, A.N. (1969) "Drawdown Failures of Centrifugal Models". Proc.VIIth Int.Conf.Soil Mech. & Found.Eng. Mexico,vol.2,p.497.

CRAIG, W, (1970) "The Undrained Shear Strength of a Boulder Clay" , M.Sc.Thesis, University of Manchester.

ENDICOTT,L.J.1970) "Centrifugal Testing of Soil Models Ph.D. Thesis, Cambridge Univ.

FUGLSANG, L. (1971) "Preliminary Centrifugal Studies of the Deformation & Failure of Uniform Earth Slopes". M.Sc. Thesis. Univ. of Manchester.

LYNDON,A and SCHOFIELD,A.N. (1970). "Centrifugal Model Test of a Short Term Failure". Geotechnique, vol.20, p.440.

NASH,J.K.T.L., and JONES, G.K. (1963). "The Support of Trenches using Fluid Mud". Proc Symp.Grouts and Drilling Muds in Eng. Practice,pp.177-180, Butterworth, London.

MIKASA,M., TAKADA,N and YAMADA,K. "Centrifugal Model Test of a Rockfill Dam". Proc. VIIth Int.Conf.S.M.F.E., Mexico,vol.2,p.325.

ROSCOE,K. (1970), Rankine Lecture, "The Influence of Strains in Soil Mechanics" Geotechnique, vol.20,p.129.

ROWE, P.W. (1972), Rankine Lecture. Geotechnique, vol.22.

TER-STAPANIAN,C.J. & GOLDSTEIN, M.N. "Multi-storied Landslides and Strength of Silt Clays". Proc.VIIth Int.Conf.SMFE, Mexico, Vol.2, p.693.

4826