CENTRIFUGAL MODEL TESTS OF EMBANKMENTS ON SOFT ALLUVIAL FOUNDATIONS

DES ESSAIS CENTRIFUGALS DES MODELES DE LEVEES SUR DES FONDATIONS ALLUVIONALES ET MOLLES

ИСПЫТАНИЯ МОДЕЛИ НАСЫПИ НА МЯГКОМ АЛЛЮВИАЛЬНОМ ОСНОВАНИИ В ЦЕНТРИФУГЕ

R.H. BASSETT, B.Sc. (Eng.) M.A.Ph.D., Senior Assistant in Research, Cambridge University, Engineering Department (England)
(Formerly Senior Soils Engineer with Balfour Beatty & Co.Ltd.)

SUMMARY. The testing of soil models in a centrifuge was initiated at Cambridge seven years ago. This paper describes two series of tests carried out on a low embankment section constructed on soft alluvial foundations. The particular features of the Cambridge model technique are well illustrated by the data presented. The data shows the development of fields of shear and volumetric strain and illustrates the formation of failure mechanisms. The models described represented a prototype embankment and were constructed with actual soil from the site. The sampling, building and testing of the models are outlined and the performance of the model is shown to compare well with an instrumented section in the field.

INTRODUCTION

The analytical solution of problems in soil mechanics is commonly dominated by the following three features:-

(i) The self weight stresses in the soil

(ii) The complexity of the stress-strain laws that govern the behaviour of each soil element. In contrast to most other engineering materials soil behaviour is highly dependent on stress level, soil density or voids ratio and is commonly anisotropic.

(iii) Soils of very different characteristics occur adjacent to each other both as naturally occurring stratified layers and as placed material in different zones of a construction project. The composite interaction and the compatibility on such soil interfaces are difficult to assess analytically.

A scale model constructed with undisturbed soil samples from the actual location of the prototype would come some way to reproducing the soil interaction characteristics (iii) and would possess the correct stress-strain relationships, including anisotropy, provided all points in the model experienced the prototype stress level. This requirement and the self weight stress condition of item (i) obviously cannot be achieved in a small scale static laboratory model even built with real materials. If, however, the small scale model (scale factor 1/F x prototype) can be subjected to a false gravitational field with a magnitude of F x G then the apparent density of all materials will be increased by the acceleration factor F and the stress at a scaled depth (1/F)(D) will be identical to the stress in the prototype at depth D, thus satisfying item (i). If the boundary stresses and restraints of the prototype are correctly modelled then the correct stress levels due to applied loadings will also be achieved.

Convenient physical scales for models lie between 1/20th and 1/200th of full prototype size. False accellerations between 20G and 200G are therefore required to model prototype conditions. Although several methods can be devised for producing these acceleration rates for short durations, there is an additional advantage in maintaining the model under the increased gravitational field for as long as possible for the following reason:- If prototype soils are used in a model at the prototype stress level the permeability K in the model and prototype will be identical. The time involved in the primary consolidation of compressible cohesive materials is related to the rate of dissipation of excess pore water pressures. The rate of dissipation of pore water pressure is governed by two factors, (a) the pore pressure gradient existing along a drainage path at any instant, and (b) the length of the drainage path, which can be expressed in the form $Tv = \frac{C_v . t}{H^2}$ (Schofield and Wroth 1968).

As the stress levels in the model will be identical to the prototype, so presumably will be the pore pressures generated, but the distances between similar points will be 1/F x the prototype distance. The pore pressure gradient will therefore be F times as high as the prototype. Similarly the physical lengths of the drainage paths will be 1/F times those of the prototype. The time for a specific degree of consolidation will be reduced in the model by $1/F^2$. A numerical example will illustrate the significance of this factor:- a 1/60th scale model will achieve any specific degree of consolidation in 1/3600 of the prototype time or 1 second in the model ≡ 1 hour in the prototype or 10 years of prototype life can be modelled in 24 hours of running time.

THE CENTRIFUGE AND MODEL CONTAINERS

The obvious method of providing a steady continuous acceleration is to use the radial component of acceleration (w^2R) of a mass revolving at a steady angular velocity (w) on an arm of radius (R). This is the principle of centrifugal modelling which was initiated at Cambridge by Professor Schofield (Avgherinos and Schofield 1969) and at Manchester (Lyndon and Schofield 1970) and has since been actively pursued by Avgherinos (1969), Endicott (1971) and Beasley (1973). However, some comments must be made to illustrate the restrictions peculiar to the method as used at Cambridge.

Fig.1 shows a model (M) size 2ℓ x $2h$ being rotated at a constant speed (w) at a mean radius (R). For a constant w the gravitational field experienced by the model will vary proportionally with R from $w^2(R - h)$ to $w^2(R+h)$. Also the acceleration field is at all points radial. The angular error in direction of the gravitational field experienced by the model will be $\pm\tan^{-1}(\ell/R)$. To keep these two factors within acceptable limits the model size must be chosen to suit the available centrifuge. The centrifuge at present used by the Cambridge group* is hired from Messrs. Lucas Aerospace Division at Luton. A general view of this machine is shown in Fig.2. The arm has a radius of 2.7 m and can be run at any steady speed to give a range of acceleration values of 20-85G. To balance the machine models are run in pairs. Each model is carried in strong containers, one of which can be seen in Fig.2. The space available for a model is 2ℓ (Fig.1) = 72 cm and $2h$ = 27 cm.

These dimensions give a variation in G at bottom and top of the container of ± 4½% or of ± 2½% across the critical area of the models being studied in this paper. The

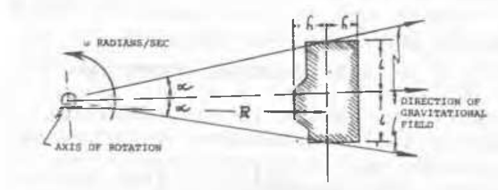


Fig.1 Diagrammatic plan of a centrifugal model.

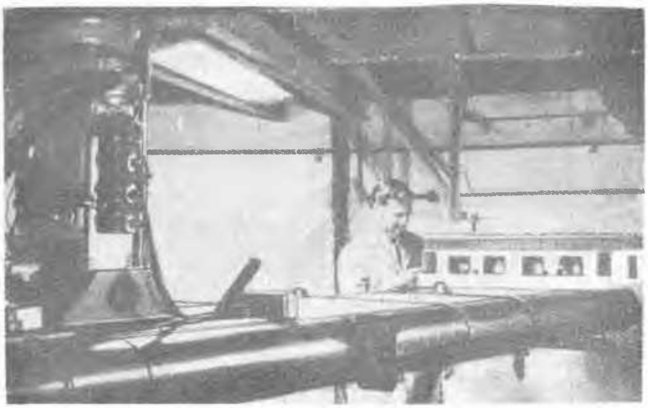


Fig.2 View of the Luton centrifuge.

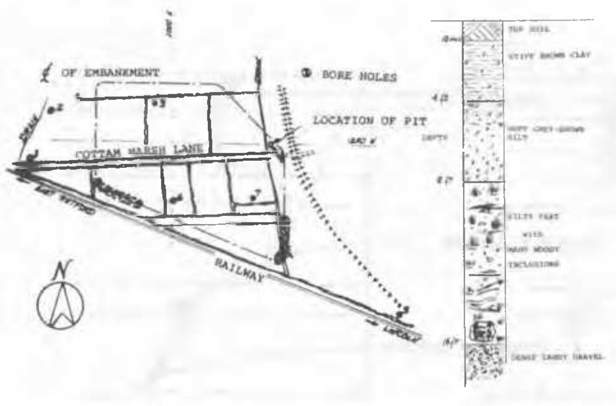
angular divergence at the extremities is ± 7% or a straight surface in the model has an apparent ground slope of 1 on 8 at the edges. The 27 cm dimension at 60G enables a combined prototype embankment height and foundation thickness of approximately 15 m to be represented. In the Cambridge technique the centrifuge axis is vertical so the model is rotated while lying on its side. This means that the face of the model which is exposed at the top of the container is in the prototype a vertical section through the embankment and the foundation materials. During a test this section is observed through the thick perspex cover (visible in Fig.2) by means of a synchronised stroboscope. At specific time intervals glass plate photographs of this vertical section are taken with a Lindhoff camera. Details of the data observed in these photographs will become apparent from the description of an actual test series.

THE PROTOTYPE

The prototype modelled in the series of tests described in the next sections of this paper was a low (5 m high 2200 m long) composite section embankment founded on soft compressible alluvial materials. The site is typical of many mature river valleys and estuaries and details of the location of exploratory boreholes and the typical stratification found in the sampling pit are shown in Figs. 3a and 3b respectively.

The embankment forms an additional ash lagoon for Cottam Power Station. The power station is owned by the Central Electricity Generating Board and is located on the flood plain of the River Trent. The Board's consultants, Balfour Beatty & Co.Ltd. had anticipated both large settlements and differential settlements from the soft silt and peat layers. However, they found that conventional analytical methods were not helpful in evaluating the influence of the differential settlements on the long

*A 10 m diameter machine with a capacity of 250 G tons and maximum acceleration of 200G is currently under construction at the University.



(a) (b)

Fig.3 (a) Plan of the site showing boreholes and trial pit.
(b) Strata in trial pit.

term performance of the relatively stiff composite section embankment. It was therefore arranged to use the centrifugal modelling technique to assist them in their examination of this prototype.

UNDISTURBED SAMPLES

The most suitable acceleration value for modelling this prototype on the Luton centrifuge was 60G. It was proposed to test three model embankment sections and one foundation model with no embankment section (as a control). Sufficient undisturbed material was thus required from each of the four main strata to construct four complete, identical models of the foundations. Blocks of each soil type were obtained in stiff steel containers (Fig.4) and from a 3 m square trial pit excavated adjacent to borehole No.4 (see Fig.3(a)). The containers measure 1 m long x 30 cm wide x 35 cm deep and they were filled with undisturbed material from the

Fig.4 Container for obtaining undisturbed soil samples.

top part of each different strata, stiff clay, soft silt and peat, by carefully jacking the sample container into the soil using a hydraulic excavator. Excess material on the top and bottom surfaces was cleared away with a cheese wire and these surfaces were then greased and sealed with a polythene sheet and a steel cover plate. Every care was taken to disturb the enclosed samples as little as possible. The underlying sand was collected as a disturbed sample.

CONSTRUCTION OF THE MODEL FORMATIONS

The model scale was to be 1/60th full size. The strong containers in which the models are run on the centrifuge are 2ℓ (Fig.1) = 72 cm and depth = 15 cm. Blocks of soil with these two dimensions in plan and with a thickness equal to 1/60th of the prototype strata depth had therefore to be cut from the large sample blocks. Each sample of soil was slowly extruded from the containers using screw jacks. The top 7-8 cm was discarded. The correct thickness for the modelled strata was then cut from the sample with a cheese wire, supported on a rigid tray and trimmed to the correct size with a template. The correctly sized block of soil was lifted into the strong container and carefully bedded down. This procedure was repeated for each successive strata working upwards from the bottom:- peat, soft silt, stiff clay crust, until four complete formations had been constructed. The exposed face was then trimmed to the surface of the strong container and a 2 cm grid of 2 mm diameter perspex marker spheres was embedded into the various soil strata. The perspex markers were silver plated to give them good reflection qualities. The finished model foundation is shown in Fig.5.

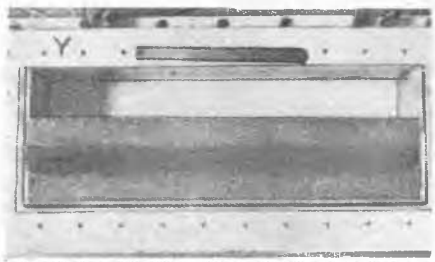


Fig.5 Model of the foundation strata showing the grid of silvered markers.

Finally the thick perspex cover plate was greased, to minimise side friction, and bolted into position.

TEST RUNNING IN THE CENTRIFUGE

The grid of silvered markers was photographed at intervals throughout the test run and it is the photographic images of the markers which provided the basic data for the subsequent analysis of strains.

The undrained strength to depth relationship which existed in the newly built model is shown in Fig.6(b) compared to the prototype relationship shown in 6(a). Each model of the foundation strata was then run for several hours at the working acceleration, 60G, with ground water reproduced at normal prototype level. During this period consolidation, and in some cases swelling, occurred until a strength-depth profile similar to Fig.6(c) was achieved. This relationship was an acceptable representation of the prototype conditions and the model foundations were then

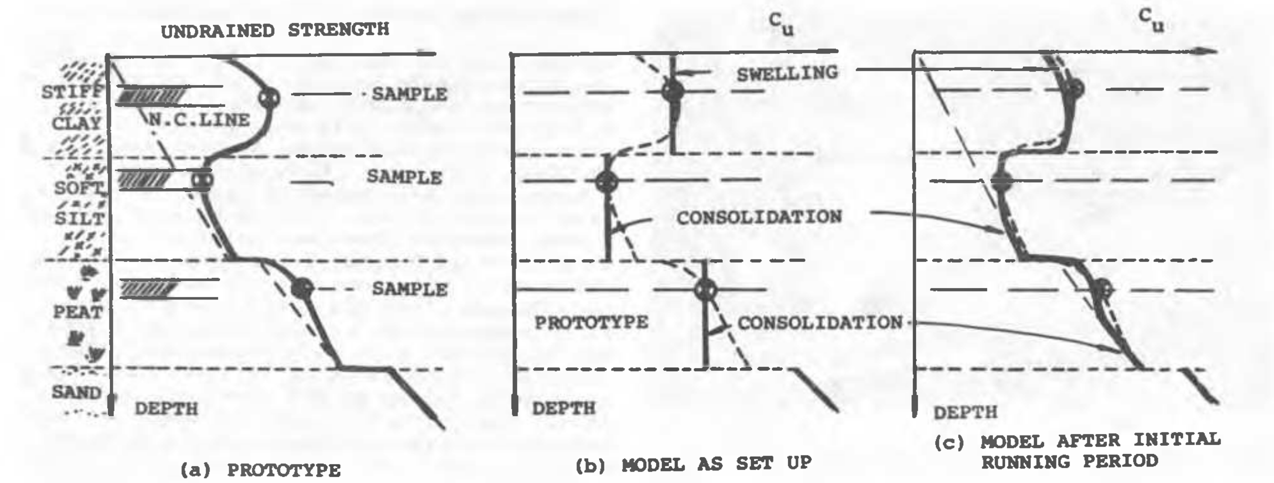


Fig.6 Distribution of undrained strength with depth.

ready to be subjected to the embankment sections.

The centrifuge was rapidly stopped, the perspex front cover plate carefully removed and the model embankment sections constructed using templates as shown in Fig.7.

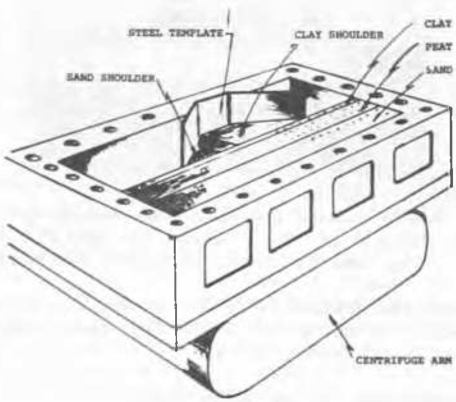
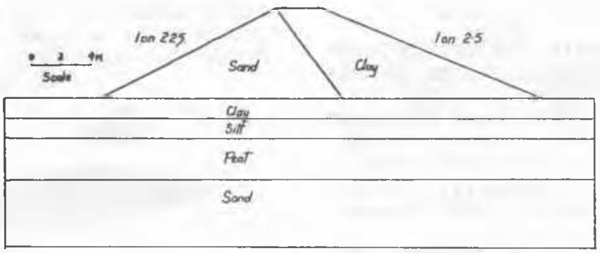


Fig.7 Diagram showing embankment section being placed.

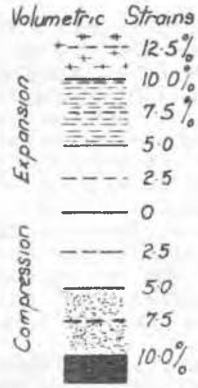
The grid of silvered markers was extended to cover the embankment section, the perspex cover regreased and again bolted in place. The centrifuge was restarted immediately construction of the embankment sections had been completed. The gravitational field was increased to 60G using a predetermined speed-time relationship. This technique was adopted to simulate the construction period of 3 months. A speed-time relationship is not an ideal technique as any particular relationship only models the construction stresses and the duration of construction correctly at one particular level. Below this level, as far as the foundations are concerned, the construction starts later and above this level the construction starts earlier. All levels in the model experience completion of construction when 60G is reached. The typical run up sequence involved rapid acceleration to 30-35G, followed by a linear increase to the running speed of 60G. Only short construction periods can be modelled by this technique and careful choice must be made of the controlling level. In the series described the control level chosen was the top of the soft silt layer. It was assumed that any major shear distortions associated with instability would be largely controlled by this horizon and correct pore pressure response at this level would therefore be important

PHOTOGRAPHIC DATA AND ITS PROCESSING

Initially at 5G and thereafter at predetermined intervals during both the construction run up and during the steady state running period plate photographs were taken of the silvered marker grid. Silvered reference markers were also embedded in the strong box and a photographic plate was thus a record of the physical location of each grid point with respect to these references at the particular point in time. Orthogonal coordinates of the centre of each grid marker with respect to fixed reference axes are measured by means of a special film measuring machine which had been built at Cambridge by James (1972). This equipment was designed for automatic measurement of X-ray negatives, but used manually on the photographic plates it produced a punch tape record of the measurements to an accuracy of between $\pm$ 5 and $\pm$ 8 microns. On a 2 cm grid length this accuracy of measurement represented an error of $\pm$ ¼ to ½% in the calculated linear strain between two points. All markers were recorded in this way and the punched tapes processed on the Department computer. In the computation the grid was arranged into near equilateral triangular elements. Assuming each element was a constant strain triangle throughout the test run it is possible by calculating the x and y displacements of each apex of a triangle to determine the volumetric and shear strains experienced by the triangular element. Details of the calculations are well documented by Bransby (1968).

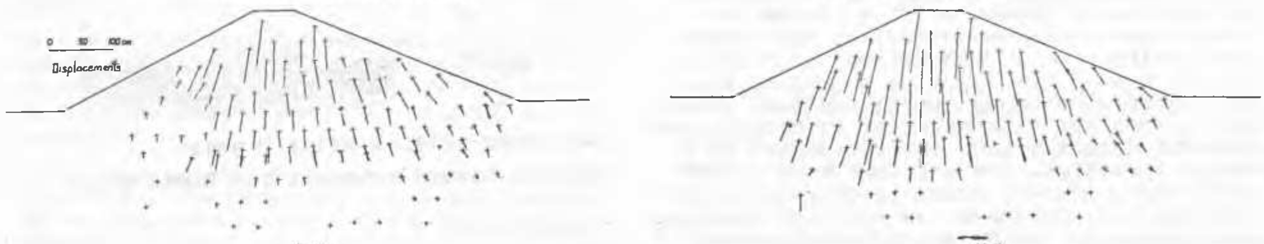


Model Section
Fig. 8.



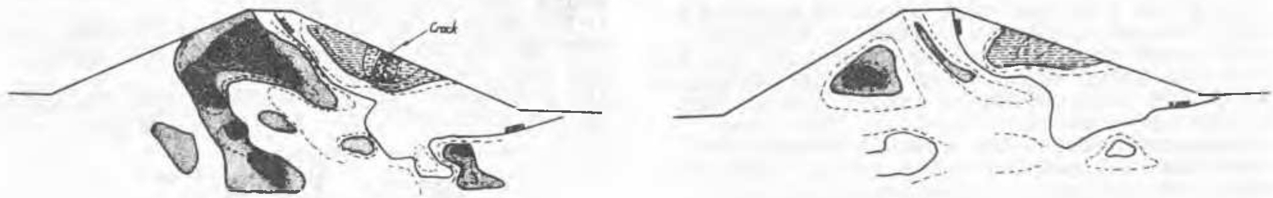
End of Construction

End of 2 years

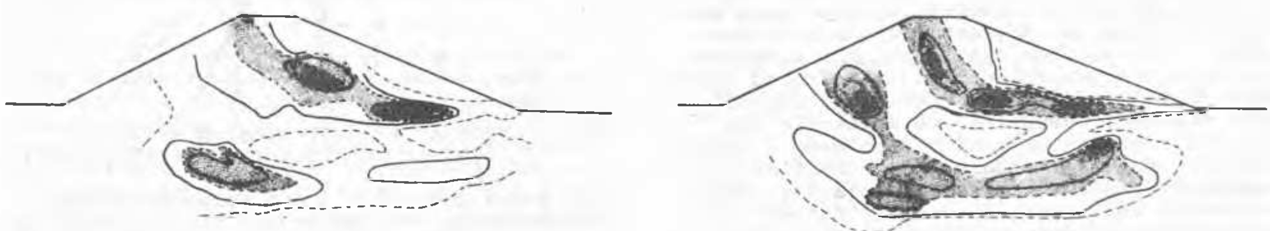


(a) (b)

Displacements of the Silvered Markers
Fig. 9.



(a) Volumetric Strains (b) Volumetric Strains



(c) Shear Strains (d) Shear Strains

Fig.10 Strain contour computed from the measured displacements for Model No.1

COMPUTED DATA FOR MODEL No.1

The three embankment sections modelled were:

Model No.1	1 on 2.25 downstream slope	1 on 2.5 upstream
Model No.2	1 on 2.25 downstream slope	1 on 3.5 upstream
Model No.3	1 on 2.0 downstream slope	1 on 2.5 upstream

The data from model No.1 at (a) end of construction, and (b) at the end of 2 years of design life, will be presented in detail.

Fig.8 shows the embankment section and the foundations, Fig.9(a) and (b) show the displacement at the two stages and Figs.10(a), (b), (c) and (d) show the shear and volumetric strain contours. These contours are of particular interest in assessing the performance of the embankment and the following points can be observed.

(i)There was a considerable volumetric compression* in the downstream shoulder at the end of construction and this continued to develop during the model time that represent- ed the next two years of the prototype life.
(ii) Expansion and tensile strain occurred in the upstream clay face eventually resulting in a crack at approximately half the embankment height.
(iii) Shear strains as large as 15% were developed at the bank/foundation interface even by the end of construction and the intensely sheared zones extend during the subsequent two years to form an apparent failure band.

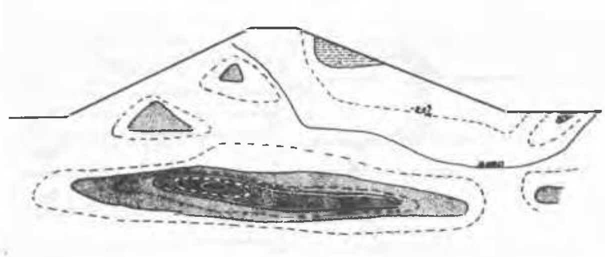
The explanation of these two features was in the nature and condition of the sand used in the construction of the downstream shoulder. The sand was as excavated from inside the site, it was fully saturated and had a very fine grading with a silt content of 5-8%. It was not a very free draining material. In the prototype construction it had been inten**ded to excavate this material by dragline and place it directly into the embankment in a single operation. The moisture content was** considerably above optimum proctor value and this had been considered as possibly advantageous because it would reduce the embankment's stiffness on such soft foundations. Model 1 had therefore been formed with nearly saturated sand with a low density.

During the test the sand shoulder appears to have drained and consolidated or suffered sufficient shear strain to experience the fall in voids ratio experienced by loose sands, and therefore it provided an inadequate support to the upstream clay shoulder. This loss of support reduced the normal stresses on both the inclined interface and on the hori-zontal foundation. In the model the horizontal formation had also not been cleaned of soft material as in the prototype a winter construction period was visualised and only minimal stripping of vegetation had been proposed. These two factors combined in the model to give a non-circular failure mechanism with the resulting tensile zone and cracking in the upstream clay face. The consultants decided, therefore, to stockpile and drain the excavated sand and to keep a close check on the condition of the formation immediately prior to embankment placing. Models Nos.2 and 3 were therefore treated in this manner and showed improved stability, the predominant feature being considerable one dimensional consolidation* in the peat layer (Fig.11).

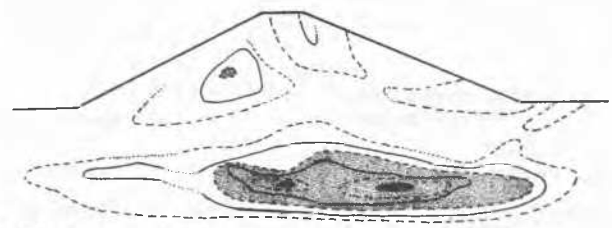
PREDICTIONS FOR INSTRUMENTATION

The consultants proposed to instrument those sections of the prototype thought to be the most critical including one adjacent to the centrifuge sample pit (Fig.3(a)). This instrumented section is shown in Fig.12 and it

* It should be noted that for one dimensional consolidation an increment of volumetric strain δV is accompanies by an equal increment of shear strain $\delta\gamma$



(a) Volumetric strains after 2 years



(b) Shear strains after 2 years

Fig.11 Strain contours for Model No.3

incorporated
**(a)induction type plate displacement gauges arranged along vertical,horizontal and inclined polythene tubes.
(b) plinths for measuring surface displacements and level changes,
(c) 3 mercury settlement gauges located under the centre line of the bank and under the half height positions,
(d) piezometer tips located at various levels in the clay,soft silt and peat layers.**

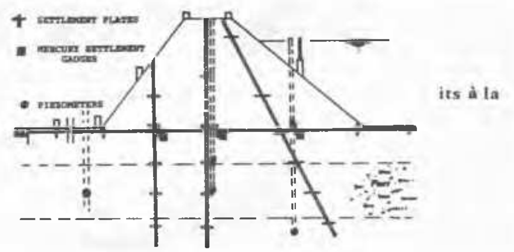


Fig.12 Details of instrumented section adjacent to the trial pit (Fig.3(a)).

With model data plots of displacement-time relationships were drawn for the specific instrument points and these gave limits on behaviour as:-
(a) apparently safe as predicted by Model 3,
(b) unsafe with the failure mechanism of Model 1.

PROTOTYPE-MODEL COMPARISON

The embankment was constructed during the autumn of 1970 but unfortunately, other than the mercury settlement gauges, the instrumentation was not fully recorded until completion of construction and therefore comparison of displacement data is not clear cut nor dramatic, as the major displacements occurred during construction. The settlement gauge records are however complete, and Fig.13 shows the model predictions for these points by the broken lines and the actual prototype settlements by the full lines. The agreement is striking.

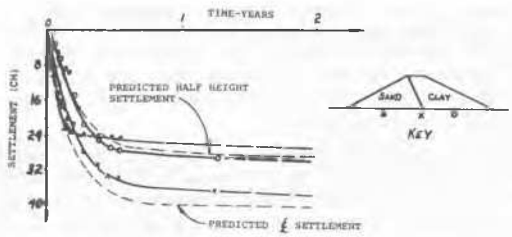


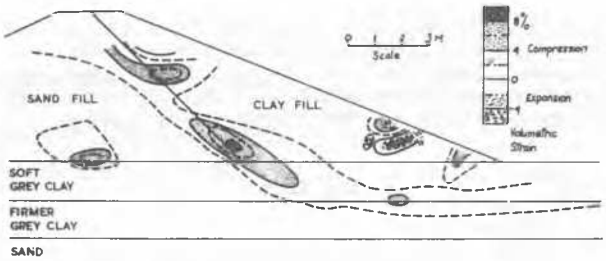
Fig.13 Predicted and measured settlements of the mercury settlement gauges.

A PROTOTYPE SLIP FAILURE AND FURTHER MODEL WORK

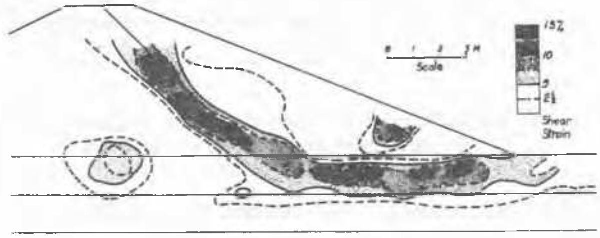
This work would therefore seem to conclude a successful centrifugal model-prototype comparison. However, circumstances are rarely so straightforward and this project was no exception.

4-5 months after completion of the construction of the prototype two 150 m long lengths of the embankment showed signs of severe distortion followed very quickly by a classical slip failure. The two areas concerned are shown hatched in Fig.3(a). They were both in areas poorly covered by the original boreholes and a comprehensive study of the two locations made after the failures indicated significant differences from the conditions assumed. The stiff upper crust was either negligibly thin or non-existent; it was replaced by a soft grey organic silty clay and the underlying stiffer silty clay contained bands of very soft material. The very reduced thickness of the stiff crust was attributed to the slightly lower elevation of the ground surface and the consequent reduction in desiccation and consolidation. The absence of the crust appeared to have been the major factor in the slips.

The client generously supported a further series of centrifugal model tests to examine how closely the centrifuge tests could model failure at working load, and if successful to use the failed models to examine the efficiency of a stabilising berm. New models were built with samples from the vicinities of the failures and tested with the prototype embankment section for a scaled 4 months. When examined visually at the 4 month stage the models showed some distortion but no rupture surfaces could be seen which could be correlated with those observed in the prototype. The derivation of strains from the photographic plates takes several weeks, so no prediction of possible failure could therefore be made during the actual testing period. However, when complete the analysis gave the shear and volumetric strain contours shown in Fig.14(a) and (b An almost continuous band of 7½% shear strain is apparent with major parts strained to more than 10%. Careful undrained triaxial compression and extension tests at various stress



(a) Volumetric strains



(b) Shear strains

Fig.14. Strain contours showing the formation of a failure band in Model 4

levels were carried out on the soft silt by Nadarajah (1972) and these indicated shear strains to peak stress ratio (q/p') of between 8 to 10%. Although these undrained data were not directly relevant it suggested that the centrifugal model had experienced strains more than sufficient to mobilise peak strength and a failure had in fact occurred. This observation suggests that this rupture formation is related to physical displacement, not to shear strain. Rupture failures are therefore not well modelled at the small scale employed in the centrifuge. This possibility has also been implied for fine sands in model tests by Kerisel (1972). It is worth noting, however, that the centre of the highly strained band aligned closely with the slip features observed in the prototype, although the large prototype displacements were not reproduced in the model

These strain data were not available during the test run and in order to continue as planned with the investigation of the berm the following procedure was adopted.

A slip plane of smeared or remoulded material was deliberately introduced on the line of the prototype failure surface by drawing a very thin sharp wet steel blade through the model. The cut model was then rerun at 60G for approximately 20 minutes. This ensured that any physical gaps caused by the cutting were closed and that the slip section moved on the cut plane to a position of stable equilibrium. It was observed that the movements of the model were now of very similar magnitude to those shown by the prototype. It was hoped that this procedure would develop a shear strength on the cut plane approaching the material's critical state or residual value.

The centrifuge was stopped, the damaged crest and toe areas cut away and replaced. A low berm section was also formed. The test was run to simulate berm construction followed by two years of prototype life. Analysis of the data showed that even with the stabilising berm in place considerable movements occured along the slip plane, both immediately after reconstruction and during flooding of the section to operating water level.

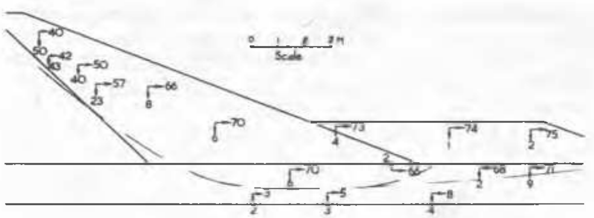


Fig.15 Displacements observed in Model 4 with an induced rupture surface and a stabilising berm.

With a deliberately preformed slip plane shear and volumetric strains are not relevant and Fig.15 therefore shows the predicted vertical and horizontal displacements (in cm) of the prototype. The distinct difference in displacements above and below the slip plane is obvious and the downward displacement at the crest of 50 cm should be noted.

THE REPAIRED PROTOTYPE

To repair the prototype the consultants adopted a higher and wider berm than that modelled above. Since repair this berm section has behaved acceptably along the majority of its length. However, over a short- length of 25 m at the south-east corner minor signs of shear distortion have again become apparent. A drop of between 10-20 cm, with slight cracking, has occurred along the inside of the crest and careful records of this are showing continued, though reducing, movement. Meanwhile the originally instrumented section at BH4 (Fig.1) has continued to remain completely stable.

CONCLUDING COMMENTS

This paper is intended to illustrate the centrifugal modelling techniques developed at Cambridge, in particular the detailed examination of a vertical section through both the model embankment and the formation strata.

Small scale model work using the prototype soils is commonly criticised on account of the changed ratio of particle size to physical dimensions. The author feels that the close comparison between model settlements and the prototype performance obtained from the first series of tests indicates that, under working stress conditions where general shear and volumetric strains are predominant, the small scale models in the centrifuge give a reliable picture of prototype behaviour and particle size is not significant. The second series of tests illustrates the limitation of this approach when rupture conditions are possible and presumably particle size is influential.

In the test data in the second series the displacements of post failure conditions were considerably underestimated. Despite this limitation sensible use of conventional stress-strain data from triaxial equipment would have enabled a failure situation to be predicted. It was interesting to study the qualitative behaviour of the deliberately induced rupture condition although the author is not at present satisfied that any great significance can be assigned to the qualitative displacement data obtained under these artificial conditions.

Finally it must be emphasised that in common with all analytical solutions and model testing methods, the conditions assumed for the analysis or for the model test must be those for which the answer is required. Sufficient information in the form of site investigation data must be available for the correct engineering judgement to be reached of which are the most critical conditions and properties to be incorporated in the analysis or in the model. A solution is relevant only to the input data.

ACKNOWLEDGEMENTS

The author would like to thank the Consultants, Balfour Beatty & Co., for enabling centrifugal model testing to be used in the investigation of a real engineering problem, and also the clients, the Central Electricity Generating Board's Midland Project Group, for financing the work. A considerable part of the data included in this paper was obtained and processed by Dr.L.J.Endicott and Mr.D.H.Beasley as part of their research work at Cambridge.

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