Basic Principles, Applications and Relevance of Centrifuge Model Testing—A Review

by

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Introduction

Centrifuge model testing was pioneered in USSR (Pokvovsky and Fyodorov, 1936). Japan, U.K. and U.S.A. are the first to follow the USSR example. The centrifuge facility has now been established in many countries. The need and utility of centrifuge are being realised by a number of research groups and the contributions on centrifuge testing have been increasing steadily since 1969. The Cambridge University research group has made a significant contribution on the subject (Schofield, 1980). Realising the significance of centrifuge testing technique, the University of California is setting up a large Geotechnical Centrifuge testing facility. In spite of these developments, the centrifuge model testing has not so far received the attention of research workers in India. In what follows, the basic principles of centrifuge model testing are briefly described and the need for a centrifuge testing facility in India is brought out.

Conventional Model Testing

Model testing in Civil Engineering and particularly in Geotechnical Engineering has been accepted as a means of investigating the behaviour of foundation structures. The aim of the model testing can be one or more of the following:

(*a*) to study the failure mechanism,

(*b*) to study the influence of various parameters governing the behaviour,

(*c*) to verify the predictions of analytical and numerical solutions, and

(*d*) *to predict the prototype* behaviour.

In model tests, a scale model of the prototype is prepared and tested in an environment as close to the prototype as possible. If the results of such model tests are to serve any of the objectives stated above, complete similarity between the model and the prototype should be established by means of dimensional analysis. Usually, it is not feasible to impose complete similarity in a model test and certain aspects which might have secondary influence are allowed to deviate. However, the experimenter should prove from available evidence that such departures have insignificant influence on the behaviour.

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Unfortunately, in spite of the fact that most of our research is based on model tests, hardly any experimenter attempts to do the above exercise. It is then little surprise that research findings based on model tests hardly find a place in design practice.

The violation of similarity requirements in the conventional model tests and its consequences on the end results can be demonstrated by some examples.

Stability of Clay Slopes :

The stability of a saturated clay slope or a cut is represented by the non-dimensional stability number, S defined as.

$$S = \frac{C_u}{\gamma H} \qquad ...(1)$$

where,

γ = the unit weight of soil,

C_u = undrained cohesion, and

H = height of the slope or cut.

When an embankment is modelled in the ratio of 1 : N and made of same soil as that of the prototype, the stability number for the model slope does not remain the same as for the prototype, but is increased by N times i.e.,

$$S_m = \frac{C_u}{\gamma H/N} \qquad ...(2)$$

Therefore, if failure is just to occur in the prototype, it will not occur in the model. This is due to the fact that the forces due to the self-weight of the soil which constitute the actuating forces are not simulated in the model whereas the strength remains the same as that in the prototype.

Load Settlement Behaviour of Footings :

The factors affecting the load-settlemenl behaviour of a footing resting on the surface of a dry sand bed are :

γ = the unit weight of sand,

D = diameter of the footing,

e = void ratio of the sand,

ϕ = angle of internal friction of sand grains,

σ_3 = insitu confining stress which is proportional to insitu vertical stress in normally loaded sand deposits,

q = load intensity, and

d = diameter of soil grains.

According to the principles of dimensional analysis, the load-settlement behaviour can be expressed as a function of four independent dimensionless products as follows :

$$F(e, \phi, \frac{q}{\gamma D}, \frac{d}{D}) \qquad ...(3)$$

In order to obtain complete similarity, the above four independent dimensionless parameters should have the same value both in the model and prototype. In conventional model tests, the footing is modelled to scale 1 : N and the tests are conducted on a sand deposit which has the same values of e, ϕ, γ and d as of the field sand deposit. This results in similarity not being satisfied with respect to insitu stress conditions and grain size/diameter ratio (Table 1).

TABLE 1

Comparison of Prototype and Model Conditions (After Ovesen, 1980)

Sl No.	Prototype Scale 1 : 1 Gravity : g	Conventional Model Scale 1 : N Gravity : g	Centrifugal Model Scale 1 : N Gravity : N.g
1	2	3	4
1	e	e similar	e similar
2	ϕ	ϕ similar	ϕ similar
3	$\frac{q}{\gamma D}$	$\frac{q}{\gamma D/N}$ Not similar	$\frac{q}{N\gamma D/N}$ similar
4	$\frac{d}{D}$	$\frac{d}{D/N}$ Not similar	$\frac{d}{D/N}$ not similar

Fig. 1 shows the vertical stress distribution under a footing and a scale model subjected to same stress intensity (Mikasa and Takeda, 1973). *Fig. 1. (a) shows the vertical stress distribution* beneath a footing 5 m wide. The dotted horizontal lines show the vertical stress due to the self-weight of the insitu soil and the curved dotted lines give the vertical stress due to the applied load. The solid lines give the combined vertical stress distribution. Fig. 1 (*b*) shows the combined vertical stress distribution below a model footing of 1 m wide. A comparison of vertical stresses at corresponding points in the footing and model reveal that the stresses are not similar. The consequence of such violation of stress similarity can be seen from the results presented by De Beer (1965) as shown in Fig. 2. This shows the relation between $P_u/\gamma D$ and D(where P_u = ultimate bearing capacity) as obtained from tests conducted by model plates on the surface of prepared dry sand beds. The results show that the $P_u/\gamma D$ ratio (or N_γ) varies with the size of model whereas, for a sand of given physical and strength properties, it should remain constant irrespective of footing size.

It is also found that the settlement vs width relationship observed in

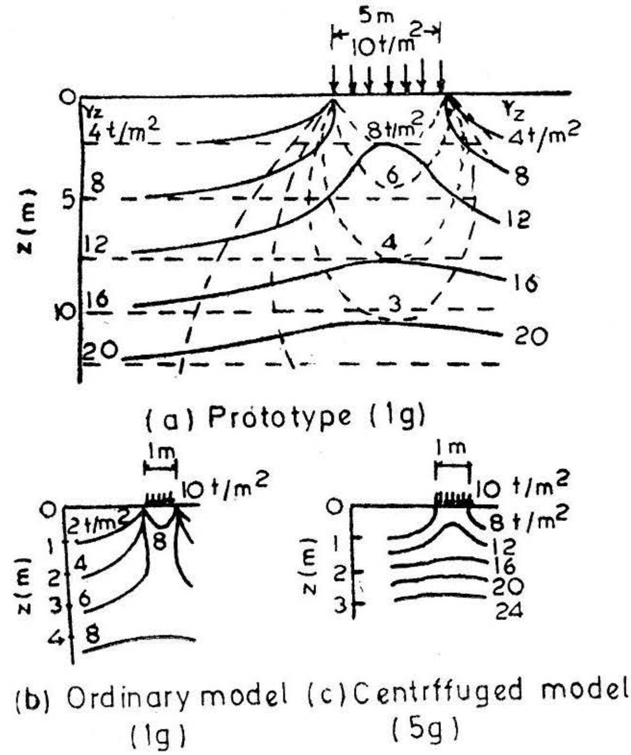

FIGURE 1 Similarity of the Vertical Stress Distribution in the Ground under a Strip Load. (After Misaka and Takeda, 1973)

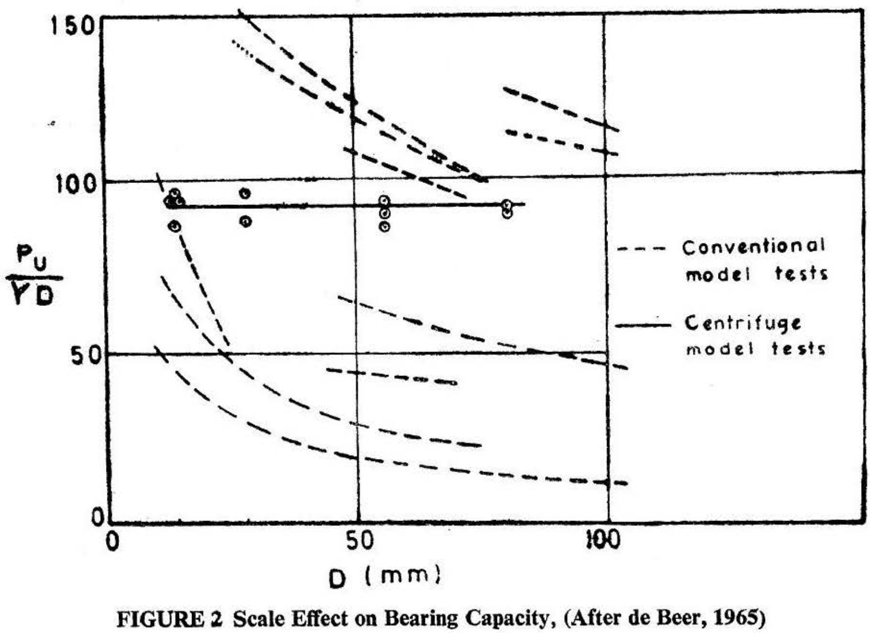

FIGURE 2 Scale Effect on Bearing Capacity, (After de Beer, 1965)

case of small scale model tests is not in agreement with that observed in the case of large footings (Fig. 3). It can be seen from Fig. 3 that for a given load intensity, the settlement decrease with width in the case of model of footings upto 30 cm in size and increases with width in the case of footings larger than 30 cm.

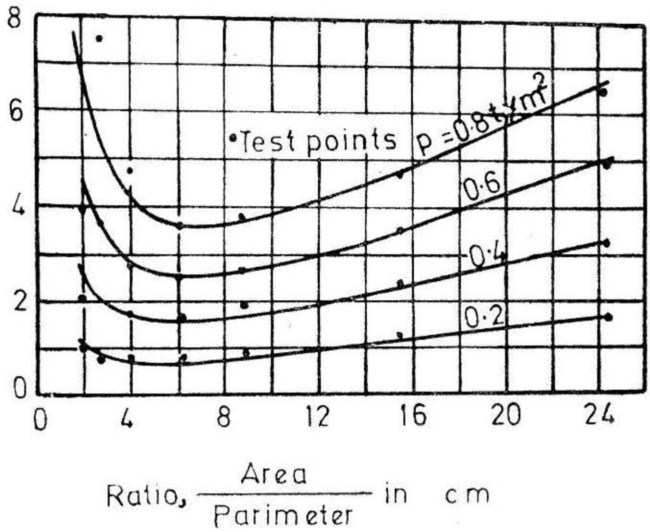

FIGURE 3 Relation Between Settlement and Loaded Area of Sand. (After Koegler, 1933)

Thus, the consequences of violation of stress similarity in model tests are quite severe and the results of such model tests can lead to misleading conclusions. Centrifuge model testing helps to achieve stress similarity and serves the purpose of model testing well.

Basic Principles of Centrifuge Testing

Simulation of Stresses

An element of soil mass in a prototype structure is acted upon by the earth's gravitational field and the soil mass exerts an inertial force called body weight. If a unit volume of soil mass is considered, the force exerted is equal to the unit weight of the soil. In centrifuge testing, the soil model is placed at the tip of a rotating arm of radius R. When the arm rotates at a speed V, the soil model is subjected to a radial acceleration given by Eq. 4.

$$a = \frac{V^2}{R} = (2\pi n)^2 R \text{ or } Ng \qquad \ldots(4)$$

where,

n = revolution per second, and

$N = (2\pi n)^2 R/g$.

As the soil model is held at a fixed radius R, a unit volume of soil mass exerts an inertia force equal to N times the unit weight of the soil, i.e. $N\gamma$. For example, in an earth dam shown in Fig. 4, the vertical stress at a point within the dam is given by,

$$\sigma_{vp} = \gamma H_p$$

where, H_p = height of soil column above the point.

If the dam is modelled geometrically to a scale of 1 in N, prepared of the same soil as of the prototype and placed in a centrifuge and allowed to rotate at a velocity such that the radial acceleration is equal to Ng, then the stress at the corresponding point in the model is given by,

$$\sigma_{vm} = (\gamma \times N) \frac{H_p}{N} \qquad \ldots(5)$$

$$= \gamma H_p$$

Thus, the vertical stress due to the body weight of the soil mass is reproduced in the model at all points as it exists at every corresponding point in the prototype. Therefore, the fundamental action of a centrifuge is to increase the bulk densities of the model materials so that the stress levels in model and prototype are equal at corresponding points.

Referring back to the example of a clay slope cited earlier, if the earth slope is modelled in the ratio of 1 : N and placed in an acceleration field of Ng in a centrifuge the stability number of both the prototype and model slopes remain the same i.e.,

$$S = \left.\frac{C_u}{\gamma H}\right|_{\text{prototype}} = \left.\frac{C_u}{N\gamma H/N}\right|_{\text{model}} \qquad \ldots(6)$$

Therefore, if failure is just to occur in the field, it will occur in the model also.

Similarly, a footing modelled and placed in a centrifuge will satisfy the stress similarity as shown in column 4 of Table 1. Ovesen (1980) has reported that the results of small model tests in centrifuge indicate that the values of bearing capacity factor, N_γ remain constant with width (Fig. 2) which is in confirmity with Terzaghi's theory on bearing capacity.

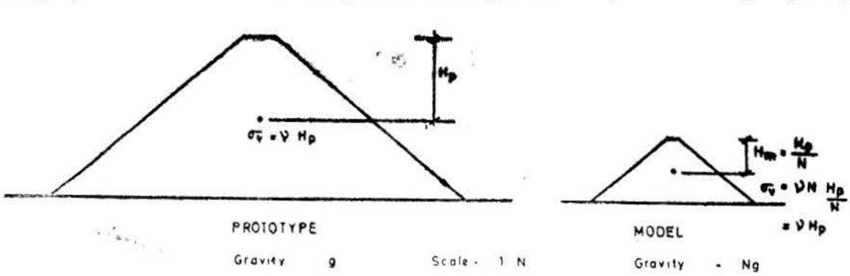

FIGURE 4 Stress Similarity in an Earth Dam Model.

Simualtion of Consolidation Process

Another advantage in a centrifuge is that events involving consolidation can be simulated exactly and the process of consolidation which might take place for years in a prototype structure can be brought about at a very short period. When a geometrically similar model is formed from the actual prototype material, the permeability distribution in model and prototype will be identical. If the imposed water levels are modelled in the same way as the structure dimensions, the steady state pore pressures at corresponding points will be equal in the same way as are the total stresses. The dissipation of excess pore pressures due to the imposed loads is treated as a primary consolidation process and is governed by the Eq. 7,

$$T_v = \frac{c_v t}{H^2} \qquad \ldots(7)$$

in which

c_v = coefficient of consolidation,

T = time factor,

t = time, and

H = length of drainage path.

If the model is prepared of the same material as of the prototype such that c_v corresponds at all points, geometrically similar excess pore pressure distributions in the model and prototype dissipate identically. The degree of dissipation is governed by T_v and the time scale dissipation process is given by,

$$\frac{t_m}{t_p} = \frac{H_m^2}{H_p^2} = \frac{1}{N^2} \qquad \ldots(8)$$

The subscripts p and m refer to prototype and model respectively. According to Eq. (8), a test performed in a centrifuge at an acceleration of 100 g for a period of 53 minutes would model a year of prototype time. The centrifuge model test can therefore be used to study problems of long term stability and deformation.

Description of Centrifuge Set-up

The centrifuge set-up (Fig.5) consists of a rotating arm driven by an electric motor. The model is prepared in a basket and attached to one end of the rotating arm. A dummy basket is attached to the other end of the rotating arm to counter-balance the weight of the model basket. A schematic view of a small combridge *MK* II centrifuge is shown in Fig. 5 and a modern Soviet centrifuge is shown in Fig. 6. A list of centrifuge machines in operation is given in Table 2.

The models can be loaded when the model basket is in flight using appropriate techniques. The loading technique needs to be developed for

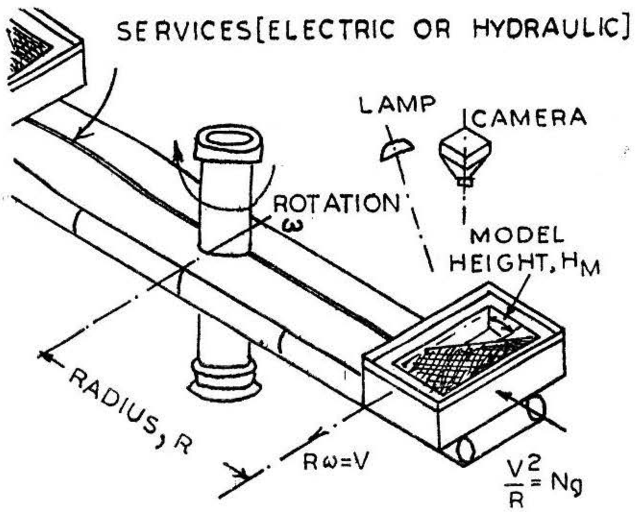

FIGURE 5 Centrifuge set-up (After Avgherinos and Schofield, 1969)

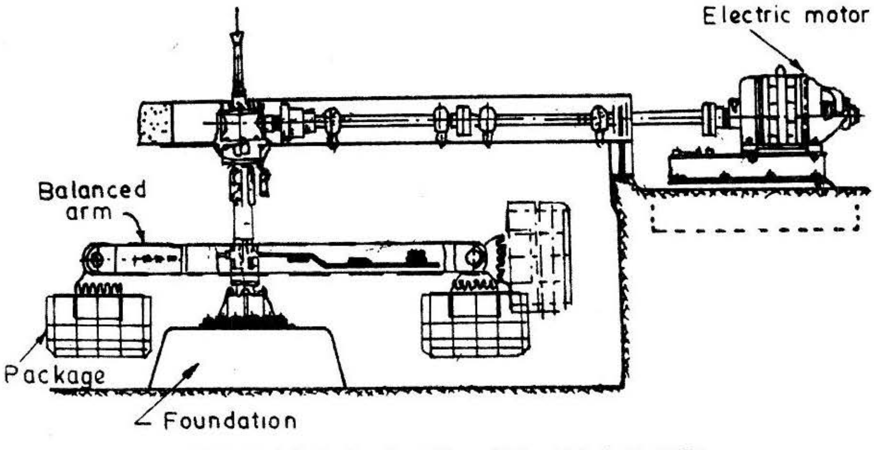

FIGURE 6 A Soviet Centrifuge (After Schofield, 1978)

each problem. Loading techniques adopted in solving a variety of problems have been described in the literature (Scott, 1978 ; Arulanandan, et. al., 1982 ; Hoadley, et. al., 1981, Ovesen, 1975 ; Schofield, 1980 and 1981) and some of these are discussed elsewhere in the paper.

The behaviour of the model during the run can be monitored by using closed circuit *TV*, displacement transducers such as LVDT, strain gauges, piezometers etc.

Applications of Centrifuge Testing

The centrifuge model testing can be used to solve a variety of geotechnical engineering problems. The problems which have been tackled success-

TABLE 2

Geotechnical Centrifuge Machines

Sl. No.	Location	Dimensions and capacity of model package	Radius	N
1.	Gosstroy, Moscow	$0.9 \times 0.5 \times 0.4\ m$ 0.2 T (model)	—	270
2.	Moscow Institute of Railway Engineers	$0.9 \times 0.5 \times 0.4\ m$ 0.2 T (model)	—	320
3.	Moscow Hydro-project	$0.9 \times 0.5 \times 0.4\ m$ 0.2 T (model)	—	320
4.	University of Manchester. Institute of Science and Technology	$0.7 \times 0.7 \times 0.4\ m$ 0.5 m (model)	—	130
5.	Kier	$1.4 \times 0.75 \times 0.5\ m$ 0.55 m (model)	—	320
6.	Cambridge University, U.K.	$0.8 \times 0.8 \times 1.5\ m\ (M)$ $1.0 \times 1.0 \times 1.5\ m\ (P)$ 0.7T (M), 1.3 T(P)	4.0 m	150
7.	University of Manchester Simon Engg. Lab.	$2. \times 1 \times 0.6\ m\ (M)$ $2.2 \times 1.2 \times 0.6\ m\ (P)$ 2 0 T (M), 5.3 T (P)	—	140
8.	Danish Engg. Academy Denmark	(i).53 m dia, 0.36 m deep (ii).14 m dia. 0.11 mm deep	2.25 m 0.72 m	80
9.	Osaka city University, Osaka, Japan	$.50 \times .30 \times .10\ m$ 61 kg+29 kg=90 kg (box) + sample	1.0 m	200
10.	California Institute of Technology, USA	$.45 \times .56\ m$ Pay lgd—45 kg	1.0 m	175
11.	National Geotechnical Centrifuge, Civil Engg. Deptt. *University California,* Davis, CA—95616USA	2700 kg or 18,000 kg	8.0 m	300 or 100
12.	University of California Davis, USA		1.0 m	
13.	University of Missouri, Rolla, USA		0.92	2000 g
14.	Purdue University, U.S.A.	12.5 cm height model	1.20 m	104

fully at the various centrifuge centres are :

1. Stability of slopes

(*a*) Natural slopes

(*b*) Earth dam slopes

2. *Stability of excavations*

3. Stability of tunnel roofs

4. Bearing capacity of footings

5. Piles and pile groups subjected to static and dynamic loads

6. Uplift capacity of anchors

7. Behaviour of offshore gravity platforms and

8. Liquefaction studies.

Some of the applications reported in the literature are briefly presented to illustrate the use of centrifuge model testing technique.

Bearing Capacity of Footings on Sand

Ovesen (1980) has compared the results of small model tests in centrifuge with the results of conventional model tests. The comparison (Fig. 2) shows that whereas in conventional model tests the values of $N\gamma$ are found to be affected by the size of the model, in the centrifuge tests, $N\gamma$ values have been found to remain constant with width. This illustrates that by achieving stress similarity in a centrifuge, useful studies on bearing capacity problems can be carried out.

Stability of a Natural Slope

Lyndon and Schofield (1978) have reported a study on a slide that took place at Lodalen, Oslo. The study demonstrates how the stress history, loading and failure of a natural slope could be simulated in a centrifuge.

The prototype situation is shown in Fig. 7. The slope was deepened to 1 : 2 in 1925 by cutting back 5 to 6 *m* and again by a further 2.5 *m* in 1949. The failure occurred in 1954. The failure was attributed to swelling subsequent to the final excavation and loss of strength.

The prototype situation was simulated in the model assuming the slide area was initially normally consolidated and subsequently eroded to the natural profile that existed in 1925. The shear strength at site was found to increase linearly with depth. The ratio of undrained shear strength c_u to change in effective overburden pressure p was found to be 0.16, Based on this ratio, the location of the original ground profile was estimated. The same is also shown in Fig. 7. The slope before 1925, 1949 and 1954 are also shown Fig. 7.

The modelling constituted the following steps:

1. Block samples of $820 \times 710 \times 310$ *mm* size were obtained from the field. (If the performance of the model is to be similar to that of the prototype, fabric, structure and sensitivity of the soil should be same apart from other things both in model and prototype. This can be achieved only by preparing the model from undisturbed samples).

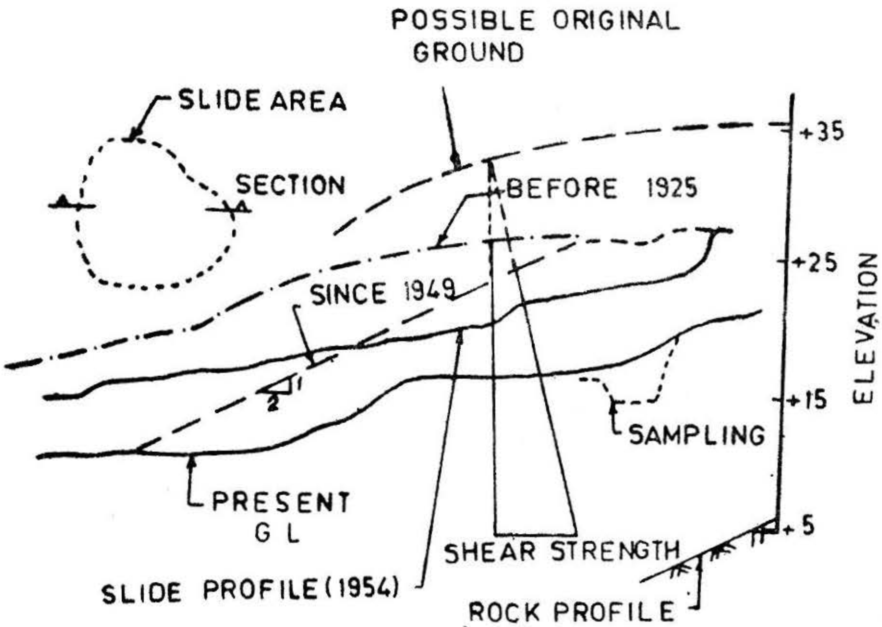

FIGURE 7 Protoype Situation Lodalen Slide, OSLO

2. The stratum affected by the slide extended to a depth of about 35 *m*. *A* 35 *m* deep stratum was simulated by a model depth of 0.3 *m*. The corresponding scale factor is 115. The centrifuge therefore was run at 115 *g*. The model was subjected to 15 *h* (23 years of prototype) of consolidation at 115 *g*. The centrifuge was stopped, 1954 profile was formed and during this period, the models were allowed to swell in earth's gravity for 180 min. On reloading, the failure occurred 11 min. after attaining 115 *g*. The prototype events and the corresponding model events are given in Table 3.

TABLE 3

Correspodance between the Prototype and Model Events

Pootype events	Model events
Consolidation to original effective overburden pressure	Preconsolidation profile estimated was formed and centrifuge run at 115 g.
Cutting back 5 to 6 m in 1925 and 2.5 m in 1949	Bring the machine to rest and form 1954 profile and allow swelling to take place.
Occurrence of landslide	Run the machine as 115 g resulting in loading and failuje,

3. The pore pressure distribution was simulated by providing a rock drain and an overflow drain as shown in Fig. 8. (Pore pressure mearurements were taken shortly after the slide and the pore pressure distribution was determined at four points. These showed artesian conditions in the deposit. It was assumed that the artesian conditions was due to piezometric level in the rock higher than the ground water level).

Closed system hydraulic piezometers connected by nylon tubes to transducers were used for measurements of pore pressure. The test was continuously observed by closed circuit television and recorded on video-tape for subsequent play back. The changing topography of the model was surveyed intermittently by the two stationery photogrammetry cameras and stereoscopic plotting.

The real sliding surface in the model was established with the track of the slip as it emerged at the toe of the slope and the measured displacements of the piezometer installations. The observed model slip circle and the prototype slip circle are shown in Fig. 9 and the close agreement seen indicates that centrifuge testing can simulate prototype events.

Stability of a Flood Bank (Hird et. al. 1978 and Padfield, 1980):

The prototype situation is shown in Fig. 10. Flood embankments were constructed to avoid flooding of the low lying marshy land along the river Thames in U.K. The embankment is resting on soft clay and peat overlying permeable sandy gravel which outcrops in the river bed. The piezometric head in the gravel with the flood embankment follows closely the water level in the river.

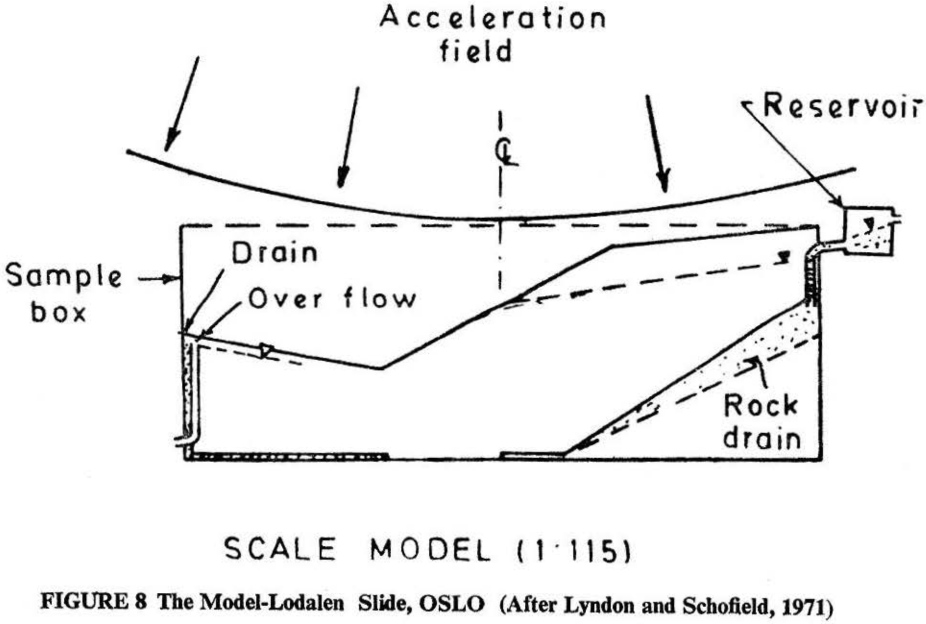

FIGURE 8 The Model-Lodalen Slide, OSLO (After Lyndon and Schofield, 1971)

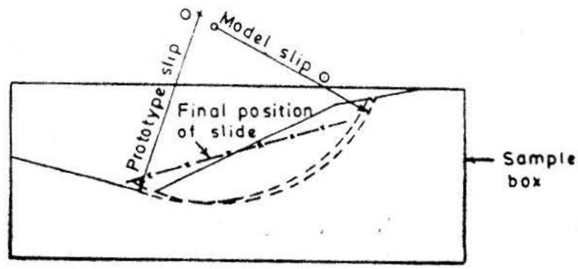

FIGURE 9 Results of Centrifuge Model Tests Lodalen Slide, OSLO

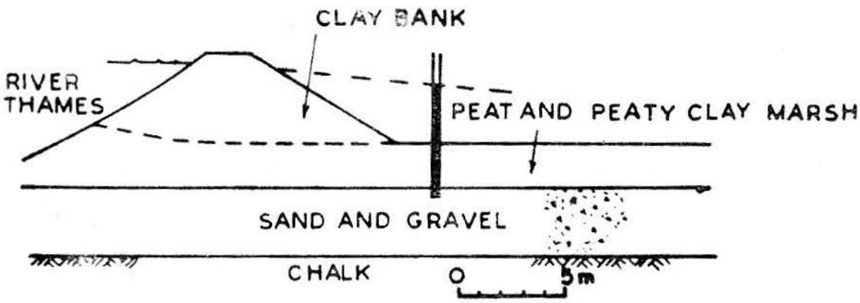

FIGURE 10 Section Flood Bank at Dartford Lock Breach. (After Padfield, 1979)

The problems posed are :

(*i*) Stability of the embankment

(*ii*) Stability of the soft clay layer against uplift

(*iii*) Effect of uplift pressure on the stability of embankment.

It is possible to examine (*i*) and (*ii*) independently of each other. However, the relationship between the embankment stability and uplift pressure is not necessarily a simple one. It is possible that uplift pressures lower than those required to lift the impermeable stratum could impair embankment stability if a slip mechanism was to form which took advantage of the reduction of effective stress and shear strength in the previous uplift zone. The formation of such a slip, which could be non-circular, would depend on the geometry of the bank and underlying layer and on soil properties.

The prototype situation was modelled in a centrifuge and an unexpected failure mechanism was observed. It was found that the failure surface passes along the clay/gravel interface and does not reemerge at the ground surface. Instead, the clay layer got compressed resulting in increased thickness at the vicinity of the toe of the embankment (Fig. 11).

Tunnels in Soft Clay (Kimura and Mair, 1981) :

In soft ground tunneling, the heading remains unlined. The designer is concerned with the temporary support required in the unlined portion

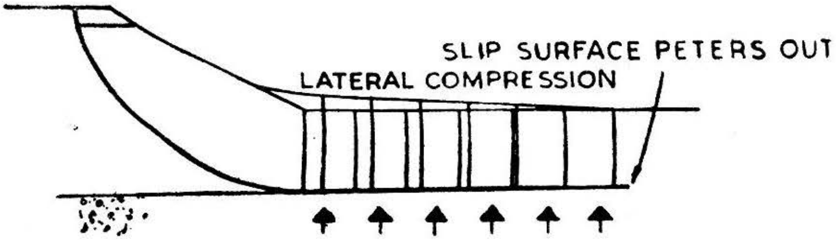

FIGURE 11 Observed Failure Where Uplift and Self Weight Interact in Model. (After Padfield, 1979)

of the tunnel to maintain its stability and the ground movements. The heading is temporarily supported by the use of compressed air. The stability of the tunnel heading is impaired by the self weight of the ground above the tunnel axis. The situation can be as shown in Fig. 12. The stability is a function of C, D, P, and σ_T. Here the loading is due to self-weight and the situation can not be simulated by conventional models.

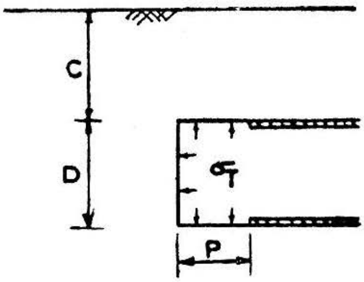

FIGURE 12 Idealized Tunnel Heading

Tunnels of 60 *mm* diameter were cut in clay models and tested at 75 *g* and 125 *g*. The models were equivalent in terms of stability to 4.5 *m* and 7.5 *m* diameter prototype tunnels in soft clay. Tests were caried out varying C/D ratio, and P/D ratio and the pressure σ_T just required for stability was estimated.

As the centrifuge speed was increased to the predetermined level, tunnel stability was maintained by compressed air supplied to a flexible rubble bay within the tunnel. During the increase in speed the compressed air pressure was increased always keeping equal to the total overburden pressure of the tunnel. When the centrifuge acceleration reached a predetermined value, the compressed air pressure was reduced until failure of the tunnel occurred. The measurements of soil movement were made around the tunnel as it deformed under the self weight of the surrounding ground when the temporary support within the tunnel was progressively reduced.

Loading Techniques

Cyclic Lateral Loading of Piles:

Hoadley et. al. (1981) used a simple hydraulic loading system (Fig. 13) to apply reversible lateral force to the piles. Two equal masses were

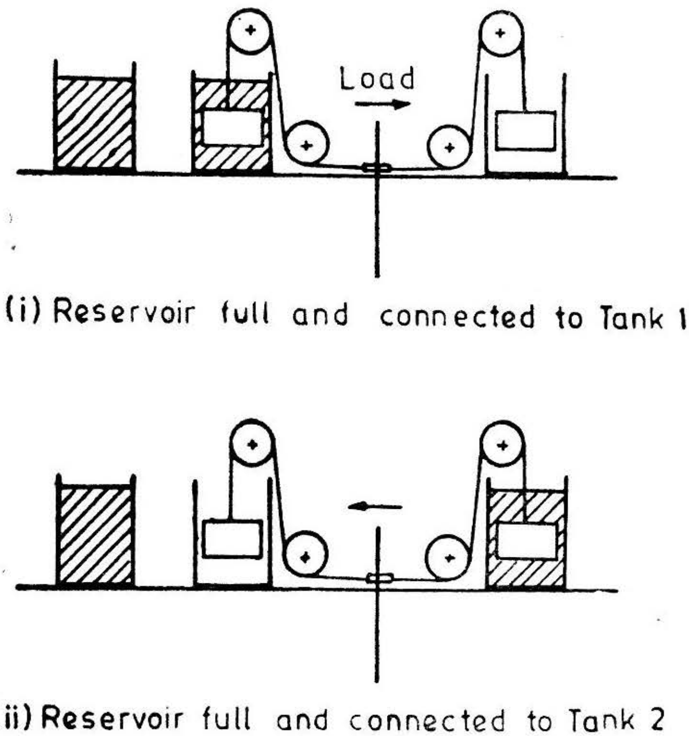

FIGURE 13 Schematic Diagram of Cyclic Loading Apparatus (After Hoadley et. al. 1981)

suspended in a buoyancy chamber either side of the pile and these balanced loads were applied to the pile through a wire and a pulley system. Water introduced alternately into either buoyancy chamber caused an unbalanced force equal to the product of the mass of water displaced and acceleration N_g of the test.

Dynamic Loading on Piles

Scott (1978) reports a study carried out on model piles under dynamic lateral loads. The vibration of the pile was achieved with a magnet/coil arrangement shown in Fig. 14. The magnet was mounted in a rigid aluminium frame work which was bolted to the centrifuge soil container, parallel to and above the soil surface.

A $2.54 \times 1.90 \times 0.81$ cm ($1.00 \times 0.75 \times 0.35$ in) piece of aluminium with 0.64 cm (0.25 in) slot to accomodate the pile was used to attach a pile rigidly to the coil. The necessary supply of current to the coil resulted in inducing the vibration of the pile head.

Earthquake Loading

Arulanandan et. al. (1982) have developed an earthquake simulator for the generation of a sinusoidal motion during a centrifuge test. The basic structure of the simulator is as shown in Fig. 15. The strain in the

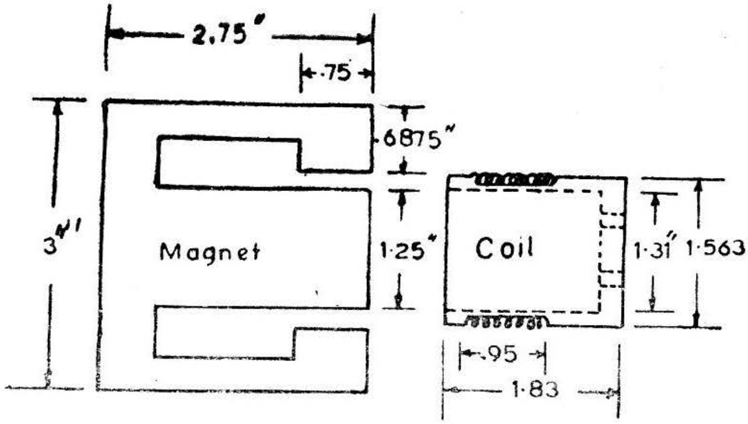

FIGURE 14 Cross Section of Magnet and Soil used in Vibration Tests. (After Scott, 1978)

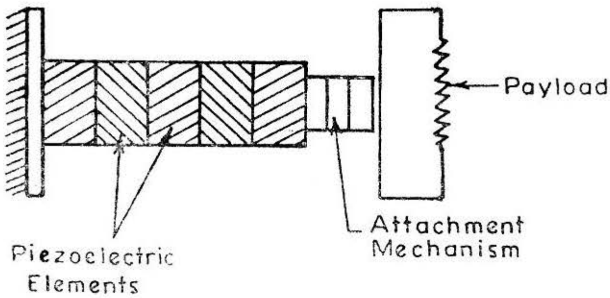

FIGURE 15 Piezoelectric Earth Quake Simulator (After Arulanandan et. al., 1982)

piezoelectric elements is directly proportional to the magnitude of the applied electric field. Thus by controlling the electric input, the strain can be controlled. High voltage sinusoidal signal is fed to the piezoelectric elements producing a corresponding sinusoidal motion proportional to the electric field across them which is transmitted to the test package.

A Bumpy Road apparatus has been developed at Cambridge (Schofield, 1981) to induce earthquake loading on centrifuge models. Cambridge centrifuge is housed in a circular underground pit. A steel track extends around about a third of the pit circumfrance. During the centrifuge flight, a wheel passes the track and is generally kept 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. The movement of the wheel is determined by whatever wave form has been accurately machined on the track segments. Oscillation of the bell crank causes horizontal ground motion of the model as determined by the wave from. The double acting pneumatic jack holds the wheel on the track for one pass and then pushes it clear.

Choice of Major Details of the Centrifuge

The radius of the rotary arm, the size and mass capacity of the model package, the maximum acceleration to be achieved and the maximum speed of the rotary arm to be achieved are the most important details of a centrifuge which one should decide before planning for the creation of the facility.

The acceleration produced in the centrifuge model is given by the Eq. 4 which suggests that the acceleration can be increased by increasing either the speed of rotation or the radius of the rotary arm. Since, the acceleration is increased as a square of n, it can be better achieved by choosing the desired range of n. However, the maximum acceleration needed to be produced depends on the appropriate model scale to be chosen for a given problem. In most of the practical problems such as stability of earth dam slopes and natural slopes, proper modelling of the prototype demands that the model scale should not exceed 100. For example, if a zoned earth dam of 100 m height with a 2 m thick horizontal filter is to be modelled, the model thickness of the filter should atleast be say, 2 cm so that the filter zone can be simulated with the prototype filter material which may be a well graded gravelly sand. To achieve this the model to prototype ratio cannot be less than 1/100, i.e. model should be atleast 1.0 m height. Therefore the acceleration cannot be more than 100 g. Thus. the specifications on the maximum acceleration and the speed should be decided on the above considerations.

The acceleration in the model shall vary within the model as the acceleration is dependent on the radius of rotation of the soil model. For example, if the thicknees of the soil model is say t, and the distance from the centre of rotation and the top of soil surface is r_1, the acceleration (or the gravity field) in the model will vary from $(2\pi n)^2\ r_1$ to $(2\pi n)^2\ (r_1+t)$. This means, when we say that a model is placed at an acceleration field of 100 g, it may mean that the centre of the model is subjected to 100 g and the top of model may experience a lower acceleration and the bottom of the model may experience a higher acceleration, The acceptable limit of variation of acceleration is of the order of 10 percent. Therefore it can be seen that whereas the model simulation requirements suggest a higher value of t, the limiting value of acceleration variation in the model suggests a smaller value of t. If these contradictory requirements are to be satisfied, the radius of rotation r should be large as compared to t. If the aim is to study real problems such as stability of zoned earth dams, natural slopes and tunnel roofs, the model should be prepared with prototype material incorporating all the features of the prototype which demands larger t values (atleast of the order of a metre) and a correspondingly large radius (of the order of 4 to 5 m). However, if the aim is to carryout research studies on prepared soil beds, smaller thickness of model smaller radius will serve the purpose. It may also be noted that the cost of the centrifuge increases roughly with the square of the radius and the size of the package (Rowe, 1975). The size of the centrifuge should be decided taking all the above factors into account.

Relevance of Centrifuge Testing in Indian Context

The most challenging and important problems in Geotechnical Engineering are those concerning earth and rockfill dams, tunnels in rock and soil, natural slopes and excavations. These form the major components in water resources projects. The design and construction of these structures are being carried out at the moment under a great number of uncertainities resulting in conservative design, unpredictable safety, considerable delay in construction and high cost. The uncertainities are due to lack of definite information on the behaviour of these earth structures which are primarily loaded due to self-weight of earth itself. The self-weight loading cannot be simulated by conventional models.

The finite element technique which is currently being used widely has by itself, failed to serve the purpose adequately mainly due to inadequate information on material behaviour under various conditions of loading and inadequate field evidence for corroboration. The centrifuge model testing technique, which can simulate self-weight stresses and loading has emerged as the best means of obtaining adequate information to predict the prototype behaviour of earth structures. The centrifuge model testing can be used to directly predict the prototype behaviour in many situations and in some more complicated situations the results of centrifuge model testing shall be complimentary to the results of a finite element analysis and vice-versa. Thus, centrifuge testing along with finite element analysis can contribute adequate information based on which important earth structure such as embankment dams, tunnels, natural slopes and excavations can be designed with more confidence. This will go a long way in reducing the cost and enhancing the safety of these structures.

India is now in the second phase of exploiting the natural water resources. After having successfully completed water resources projects at sites which are devoid of serious complication, India has now taken up projects in such places as earthquake prone Himalayan region where construction of earth structures are posing serious design and construction problems. Development of a centrifuge facility to match the already existing expertise and facility on the numerical techniques is therefore an urgent need of the country to tackle design and construction problems anticipated in the above projects.

Conclusions

The conventional model testing of earth structures and foundations has serious limitations due to the fact that the similarity between the prototype and model cannot be established with respect to self-weight loading and self-weight stresses. These difficulties can be overcome in centrifuge model testing. A number of studies have been successfully carried out in centrifuge at U.K., Japan, and U.S.A. These studies demonstrate the utility of a centrifuge in geotechnical engineering studies, particularly in the prediction of prototype behaviour of earth structures. In view of the fact that some of the water resources projects, particularly in the Himalayan region are likely to pose challenging design and construction problems concerning earth structures, the development of a geotechnical centrifuge testing facility in the country should help tackle these problems.

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References

ARULANANDAN, K. CANCLINI, J. and ANANDARAJAH, A. (1962), "Simulation of Earthquake Motions in the Centrifuge," *Journal, ASCE*, 108 ; GT5, 730-742.

AVGHERINOS, P.J. and SCHOFIELD. A.N. (1969), "Drawdown Failures of Centrifugal Models, *Proc. of 7th Int. Conference on Soil Mechanics and Foundation Engineering.*

DE BEER, E E, (1965) : "Bearing Capacity and Settlement of Shallow Foundations on Sand", Proc. of a Symposium held at Duke University.

HIRD, C.C., MARSLAND, A. and SCHOFIELD, A.N. (1978) : "The Development of Centrifuge Models to Study the Influence of Uplift Pressures on the Stability of a Flood Bank," *Geotechnique*, : 28 : 85-106.

HOADLEY, P J. BARTON, Y.O., and PARRY, R.HG., (1981): "Cyclic Lateral Load on Model Pile in a Centrifuge," *10th Int. Conf. Sail Mech. and Found. Engg.*, 1 : 621.

KIMURA, T. and Mair, R.J. (1981) : "Centrifugal Testing of Model Tunnels in Soft Clay," *10th Int. Conf. Soil Mech. and Found. Engg.*, 1 : 319.

KOEGLER F., (1933) : "Discussion : Soil Mechanics Research". Transactions *ASCE, 98* : 299-301.

LYNDON, A. and SCHOFIELD, A.N. (1978): "Centrifugal Model Tests of the Lodalen Landslide", *Can, Geotech. J.* 15 : 14-31.

MIKASA, M. and TAKADA, N. (1973) : "Significance of Centrifugal Model Tests in Soil Mechanics," *Proc. 8th International Conf. on SMFE*, 1 : 2 : pp. 273-278.

OVESEN, N.K. (1975) : "Centrifugal Testing Applied to Bearing Capacity Problems of Footings on Sand," *Geotechnique* 25 : No. 2 : 394-401.

OVESEN, N.K. (1980) : "Discussion on the Use of Physical Models in Design" *Proc. of VII European Conference on Soil Mechanics and Foundation Engineering*, 4 : 319-323.

PADFIELD, C.J. (1980) : "Discussion on the Use of Physical Models in Design", *Proc. of VII European Conference on Soil Mechanics and Foundation Engineering*, 4 : 328-332.

POKROVSKY, G.I. and FYODOROV, I.S. (1936) : "Studies of Soil Pressures and Deformations by Means of a Centrifuge", *Proc. Int. Conf. on Soil Mech* 1 : 70.

ROWE, P.W. (1975) : "Applications of Centrifugal Models to Geotechnical Structures," *Symposium on Recent Developments in Soil Mechanics*, University of New South Wales, pp. 1-25.

SCHOFIELD, A.N. (1978), "Use of Centrifugal Model testing to assess slope stability," *Canadian Geotechnical Journal*, : 15 : 14-31.

SCHOFIELD, A.N. (1980) : "Cambridge Geotechnical Centrifuge Operations, Twelth Rankine Lecture," *Geotechnique*, 30 : 3 : 227-268.

SCHOFIELD, A.N. (1981) : "Dynamic and Earthquake Geotechnical Centrifuge Modelling : State of Art Review", International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, University of Missouri-Rolla, U.S.A.