tation, Albany, Soil Design Procedure SDP-1, 1970.
10. I.M. Smith and R. Hobbs. Finite Element Analysis of Centrifuged and Built-Up Slopes. Geotechnique, Vol. 24, No. 4, 1974, pp. 531-559.

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Rectangular Open-Pit Excavations Modeled in Geotechnical Centrifuge

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Tests of models made of soil in geotechnical centrifuges have become accepted as a method of study of mechanisms of ground deformation with less expense or delay and with more control of ground conditions than tests of prototype scale. Centrifuge test results are reported for four different rectangular open pits excavated rapidly in saturated clay soil of uniform strength with depth. It is found that the mechanism by which such excavation will cause road pavements and buried services to fail will fit the axisymmetric mechanism, where the upper part of the pit wall tends only to move vertically and the wall movement is dominated by plastic deformation of the lower part of the pit wall. Support in the upper region will have little effect on this axisymmetric mechanism, and the stability of the pit will be controlled by the strength of the ground near the base of the excavation. The observed mechanism induces tension in the upper portion of the ground and induces compression failure at depth. Flexible road construction is rather weak in tension, so the observed mechanism is probably relevant to pits passing through a flexible road construction and entering a lower layer of soft ground.

The excavation of trenches or of pits in roads can cause damage to road pavements or to buried services beside the excavation. Ground movements at failure can be fitted to failure mechanisms of the theory of plasticity. Studies of ground movements before failure show that the incipient failure mechanism is established well before failure occurs and that damaging ground movements increase rapidly as the factor of safety against failure is reduced. Tests of models made of soil in geotechnical centrifuges have become accepted as a method of study of mechanisms of ground deformation with less expense or delay and with more control of ground conditions than tests of prototype scale. This paper is concerned with the modeling of rectangular open pits excavated rapidly in saturated clay soil of uniform strength with depth.

The failure of a long trench is illustrated in Figure 1a, with a vertical face ABC moving down into the trench as soil slips on an inclined plane CD and a tension crack DE opens. The failure of a circular shaft is illustrated in Figure 1b and 1c. In the rapid undrained failure (Figure 1b), the lower portion BC of the vertical face squeezes in and an annular ring of soil of section BCD is plastically compressed and deformed. Above the plastic zone BCD a rigid block ABDE descends vertically; there is shearing on DE as well as on CD, on DB, and within BCD (1). In the long term, shown in Figure 1c, a shaft that is safe against rapid failure may exhibit a crack at B and subsequently begin to cave in. This paper reports tests of four different model rectangular open-trench or pit excavations; their behavior can be compared with these plane and axisymmetric cases.

The tests used Speswhite kaolin clay soil reconstituted from a slurry. It was consolidated with vertical effective stress $\sigma_v' = 140$ kN/m² and allowed to swell back into equilibrium in centrifuge flight with stresses $0 \leq \sigma_v' < 140$ kN/m² as shown in Figure 2. When rapidly sheared, such soil has shear strength $24 < c_u < 32$ kN/m² throughout the model depth. In such rapid shearing the effective mean normal stress in the clay approaches a critical state pressure (in this case, say, $p' = 62$ kN/m²) and the pore-water pressures take whatever value is needed to balance externally applied total pressure. In the longer term, excess pore-water pressure gradients will lead to the flow of pore water. A point of particular interest in these tests is the observation of pore-water pressure changes in the clay during and after the process of excavation.

CENTRIFUGE MODEL TEST SYSTEM

In centrifuge model tests, the weight of soil is increased and the scale of the model is reduced, both by a factor n (2). The result is identical similarity at corresponding points in a model and in a notional full-scale prototype of the total and effective stresses and strains in the soil and of the pore-water pressures. In addition, the reduction of model scale by a factor of n means that pore-water diffusion to achieve a given time factor ($T_v = C_v t/h^2$ in Terzaghi's consolidation theory) requires times t_m in the model greatly reduced in comparison with times t_p in the prototype in the ratio $t_m/t_p = 1/n^2$. In this paper, tests will be reported with axes on graphs both at the model and at the notional prototype scale. Details of the model excavation dimensions are shown in Figure 3; positions of pore-water pressure transducers (PA, PB, and PC) and of displacement transducers [linear variable differential transformers (LVDTs), LR, LS, and LT] are shown in the plan for each model at model scale.

The models were made in a circular tub of internal diameter 850 mm (designated 1 in Figure 4). The clay (2) was consolidated and the pit (3) excavated and lined with a rubber bag (4) containing a bag pressure transducer (5). The bag was filled with a heavy fluid $ZnCl_2$ solution (3). Pore-water transducers A, B, and C had been consolidated into position 7. Lead powder threads were injected into the clay (8), and lead shot was placed on various surfaces. These would allow study of deformation during dissection of the model. In flight there were LVDT measurements that followed surface movements.

In flight the upper surface of the clay was partly covered by water maintained at a constant level D as sensed by a transducer (10). A cross beam (11) carried transducers and a junction box (12). After the models were made, the system was brought into equilibrium with about 9 h of centrifuge flight. The process of excavation was modeled by draining the $ZnCl_2$ solution down a vertical pipe (13). A solenoid valve (14) controlled the dumping of the $ZnCl_2$ solution into a reservoir (15). A second solenoid valve (16) isolated the surface level in a first reservoir (17), maintained from the slip-ring supply pipe (18), from the excavation base where there were filter papers (19) below the rubber bag. The excavation base drain pipe (20) was connected to a second reservoir (21), originally maintained at the same level (D) as the first reservoir. But on dumping, the level in the second reservoir fell to E, the excavation base level. It was significant to know the rate at which water seeped from the ground into the excavation and into the second reservoir, so a third reservoir (22) with a pressure transducer in it and a control valve (23) was placed between the second reservoir (21) and the waste pipe (24).

Features of the test system can be seen in Figure 5.

RECORD OF TEST 2

After a 9-h consolidation run in the centrifuge to monitor pore-water pressure and settlements and see when equilibrium was achieved, the excavation process was carried out. The excavation process was simulated by dumping the $ZnCl_2$ solution from the rubber bag in flight. The time required for excavation varies according to the size of the pits excavated. In these tests under a centrifuge acceleration of 75 g, from 15 to 30 s was needed, which corresponds to between one and two days at the notional prototype scale.

A magnetic tape recorder capable of recording a maximum of 14 channels simultaneously was used to record data from the pore-water pressure and settlement gauges. Figure 6 shows the data from pit test 2 plotted against time. As may be seen in Figure 6a, the bag pressure (measuring the $ZnCl_2$ solution pressure at the base of the pit) fell to approximately 10 kN/m² within 18 s and then gradually increased with time. After the centrifuge had been stopped, it was observed that the lower part of the pit wall was closed up and the bag pressure transducer was trapped, which probably explains the subsequent rise in the bag pressure.

The first change in settlement to occur (up to about 10 s) during the reduction of the bag pressure is very slight, as shown in Figure 6b. Further reduction of the bag pressure produced a rapid increase in the settlement rate. During undrained shear deformation of soil, the ground surface settlement has to be compensated by the inward movement of the pit wall, which is likely to alter the rate at which the bag pressure falls. It can be seen in Figure 6a that the rate at which the bag pressure falls has changed after about 10 s. After completion of the excavation, the rate of settlement

Figure 1. Failure mechanisms.

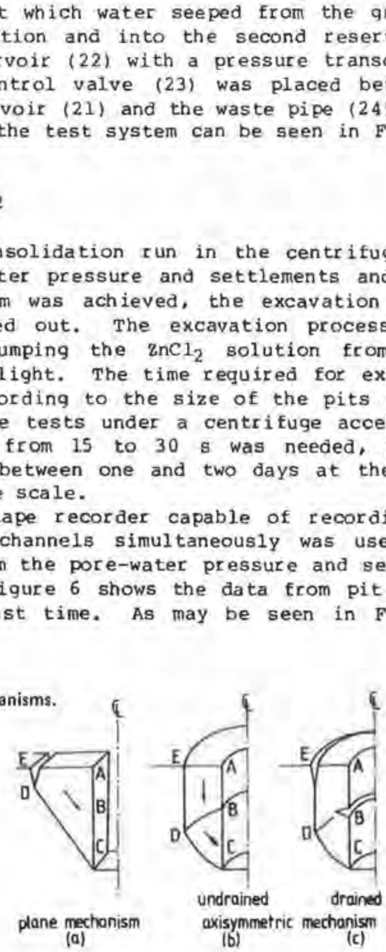


Figure 2. Stress history and strength profile.

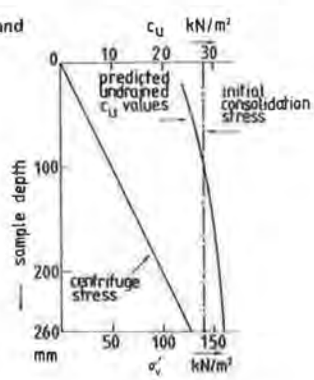


Figure 3. Pit dimensions and positions of LVDTs and pore-water pressure transducers.

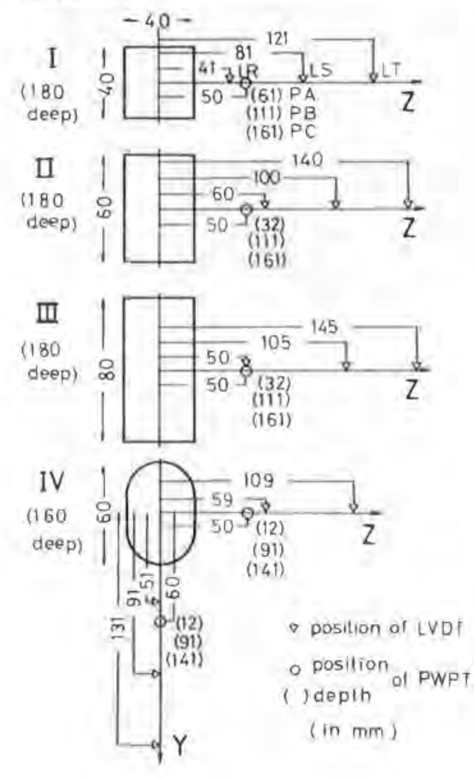
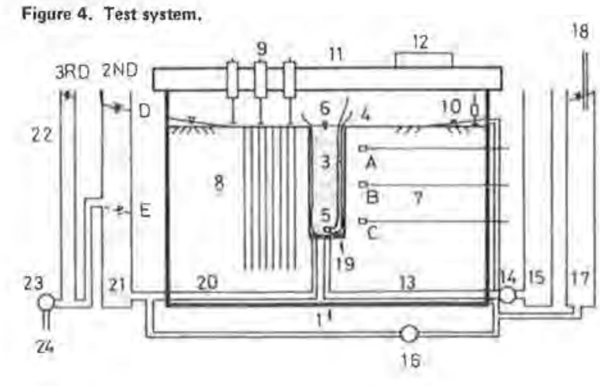


Figure 4. Test system.



quickly levels off. Once the base of the pit has closed up, there will be little further settlement. In this case about 4.0-4.5 mm of surface settlement near the pit seems enough to make the base of the pit close. Since the pit width is 40 mm, the horizontal displacement of each pit wall needs to be 20 mm. Therefore, it appears that the rate of horizontal movement at the pit-wall base is about four to five times faster than that of the surface movement near the pit.

Figure 5. Centrifuge model system.

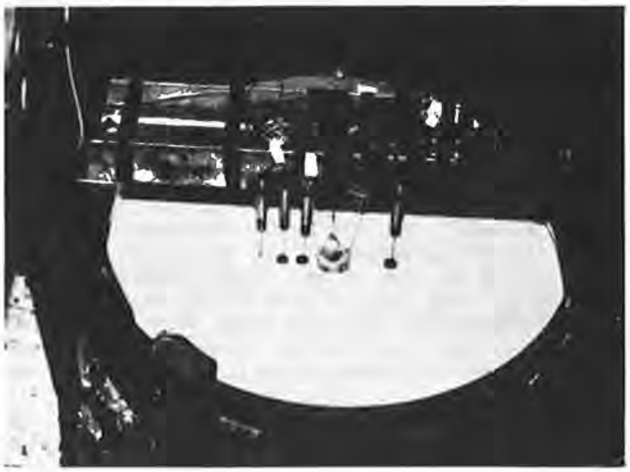


Figure 6. Data from pit test 2 versus time.

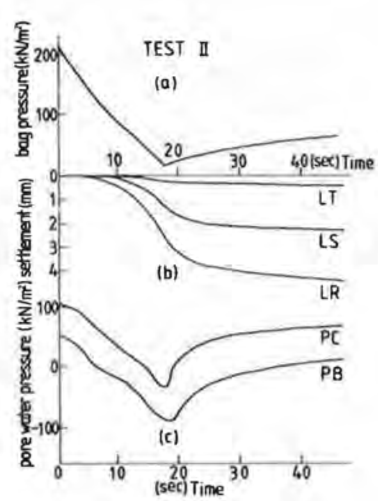


Figure 7. Settlement and pore-water pressure changes with excavation depth.

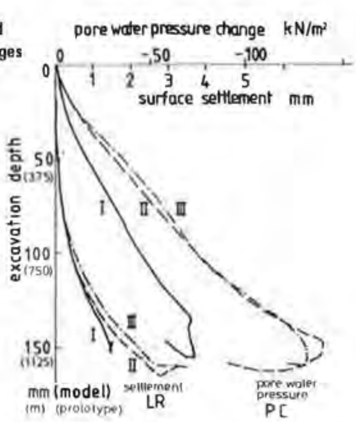


Figure 6c presents pore-water pressure changes during the test: They fall as the bag pressure drops. At about 14 s, a rapid change in the rate of fall of the pore-water pressure appears. This corresponds to the start of the steepest part of the settlement rate. Pore-water pressure suction is developed in soil during the development of a rupture plane.

Figure 8. Pore-water pressure change with excavation depth.

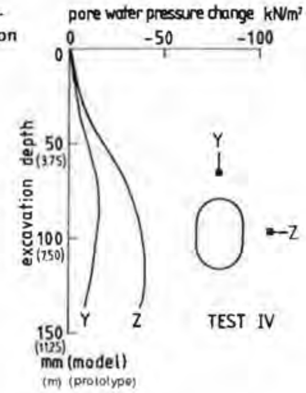


Figure 9. Settlement profiles after excavation.

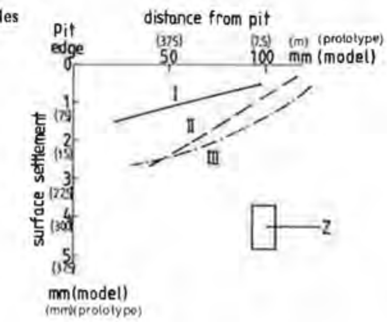


Figure 10. Settlement profile change with time.

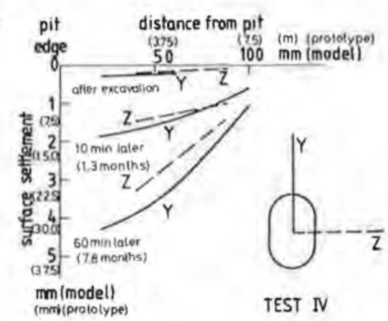
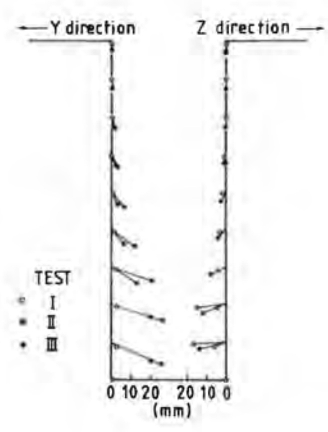


Figure 11. Wall movements.



PORE-WATER PRESSURE CHANGES AND SETTLEMENT RATES

Figure 7 shows how the pore-water pressures and settlements change with the excavation depth for pit tests 1, 2, and 3. The settlements remain small values until the excavation depth reaches about 100 mm. During this period, a linear relationship between the excavation depth and the fall in pore-water pressure can be noticed.

It is important for those engaged in practical excavation work to appreciate that saturated clay soil is a granular material and that whenever it stands with an exposed vertical face there is a suction in the pore water. The safety of the excavation depends on the ability of soil to "hold its breath" for a time, and it is often only a matter of a short time before pore-water suction decays, as is clear from the record in Figure 6c. Also, in the undrained deformation the magnitude of the suction that is generated will be greatest where the greatest shear stress is mobilized.

In the test of pit 4, the pore-water pressure changes in the two directions (perpendicular to the long side and to the short side of the pit but at the same depth) were measured as is shown in Figure 8. The pore-water pressure change in the Z direction is larger than that in the Y direction. This is consistent with the trends illustrated in Figure 7, that is to say, the large face of excavation results in the large total stress change and causes the large pore-water pressure change in the vicinity of the excavation; the longer side is subjected to the larger deviator stress.

EXTENT OF DAMAGE

Soil engineers are always concerned about a subsidence of the surrounding area induced by excavation work. According to Peck (4), in the problem of plane-trench excavation in soft clay ground, the settlement may extend as far as four times the excavation depth. The settlement damage zone for an axisymmetric excavation is smaller than that for the plane-trench excavation. In the case of our rectangular open pits, Figure 9 shows the settlement profiles (on a line perpendicular to the longer face) at the moment of completion of the excavation for pits 1, 2, and 3. It is apparent that as the side length of the pit increases, the settlement profile has a flatter shape and the depression zone extends farther, although it does not extend beyond one excavation depth so far as this series of tests is concerned.

Pit 4 was left standing (with steady slow seepage into the excavation) for 1 h, corresponding to 7.8 months in the notional prototype scale. Figure 10 presents the change of settlement profiles during this period. Here the surface settlement results not only from the inward displacement of the pit wall but also from the compression of the soil due to the lowering of the ground-water level. At the moment the excavation is completed, the settlements in both directions are very small, less than 0.3 mm, indicating no signs that large deformations have taken place during the excavation. Subsequently the settlement gradually increases and reaches more than 4 mm (0.3 m in the prototype scale) at the point near the pit edge after 60 min (7.8 months in the prototype scale). The settlement becomes more localized toward the pit edge and its shape becomes a reverse dome shaped with time. This may be because the region nearer the pit is being affected more by the lowering of the ground-water level. The depression zone, however, was about the same as those observed under undrained conditions.

Our interpretation of centrifuge model tests (5) is based on a supposition that the effectively stressed soil behaves as an elastoplastic and not as a viscous material: All time effects are supposed to arise from transient pore-water pressure gradients. It is also supposed that the geometry of the failure mechanism is independent of the scale of the test. If a full test series were to be undertaken and reported (at much greater length than is available in this paper), that would normally include similar tests at different scales. This modeling of models would allow us to check the correctness of the supposition that diffusion time in models will scale with the square of the dimension scale.

OBSERVED FAILURE MECHANISM

For undrained deformations, the surface settlement is a direct indication of the wall movement during the excavation process. Figure 11 presents the vectors of wall movement obtained from observation of the movements of the lead shot shown on the X-ray radiograph. It is clear that the wall movement is dominated by plastic deformation of the lower part of the pit wall. For pit 2, for instance, the maximum horizontal movements in the Y and the Z directions are 27 mm and 16 mm, respectively. Another important feature here is that the direction of the movement vectors changes with depth from vertical to horizontal. This corresponds more to the axisymmetric problem in Figure 1b and 1c than to the plane problem in Figure 1a. This is clearly confirmed in Figure 12, which shows the radiograph of pit 3 in the Z direction, where the lead powder threads show clear discontinuities forming a triangular shearing zone.

It is common practice to provide temporary support for the excavation work to prevent collapse of the pit wall. However, it is often the case that the lower part of the wall is left unsupported. The use of support for only the upper part of the wall prevents mobilization of the plane failure mechanism (Figure 1a) but not of the axisymmetric failure mechanism (Figure 1b or 1c). As may be seen in Figure 1, in the case of axisymmetric mechanisms the upper part of the wall will tend only to move vertically. Support in the upper region would have had little effect on a failure such as that illustrated in Figure 11.

Although in our model tests the upper portion of the ground was clay and in a typical road construction the upper portion is made much stronger in compression, the observed mechanism induces tension in the upper portion and induces compression failure at depth. A series of calculations has been made to study the significance of cracking of flexible pavement layers. Before cracking of the layer, its strength not only increases the factor of safety but also tends to increase the radius of the critical failure mode AE in Figure 1b compared with the case of a single soft layer. When the layer cracks, it acts as a surface load and reduces the factor of safety significantly and also reduces the dimension AE to the same size as that of the single layer. Flexible road pavement construction is rather weak in tension, so the observed mechanism probably is quite relevant to pits passing through a flexible road construction and entering a lower layer of soft ground.

In addition to the loss of strength associated with vertical cracks in a pavement layer, it also appears to be possible in drained long-term conditions for horizontal cracks into the vertical pit face to lead to failure. Figure 13 was taken after pit 4 was dissected to see the internal deformation. Here a horizontal crack located just above

Figure 12. Radiograph of pit 3 Z-direction section.

Figure 13. Inside view of pit 4 dissected.

the triangular shearing zone has developed. Further tests are required to determine at what stage of the centrifuge run this horizontal crack occurred. However, when full-scale pits are excavated in ground with a high ground-water level, it is sometimes observed that in the longer term the lower part of the pit face is wet and that the seepage flow sloughs off the soil from the pit wall. This is the start of caving, and the debris falls to the full-scale pit base just as was observed after the test of model pit 4. The phenomenon of caving may be one of the critical design questions for the long-term stability of vertically sided pit excavations and is to be the subject of further tests in the current series.

CONCLUSION

Well-instrumented geotechnical centrifuge model tests stand as physical events in their own right with no less significance to the geotechnical engineer than full-scale trials of prototypes. This brief series of rectangular-pit excavation models forms a small part of a large program. However, even from this brief series of tests it is possible to deduce that the damage such excavations will cause to road pavements and buried services will fit the axisymmetric rather than the plane mechanism and will be controlled by the strength of the ground near the base of the excavation rather than the strength of the overlying pavement.

ACKNOWLEDGMENT

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REFERENCES

1. A.M. Britto and O. Kusakabe. Stability of Unsupported Axisymmetric Excavation in Soft Clay. Engineering Department, Cambridge Univ., England, Tech. Rept. CUED/D/-Soils/TR105, 1981.
2. A.N. Schofield. Cambridge Geotechnical Centrifuge Operation. Geotechnique, Vol. 30, No. 3, 1980, pp. 227-268.
3. P.V. Lade, H.L. Jessberger, E. Makowski, and P. Jordan. Modeling of Deep Shafts in Centrifuge Tests. Proc., 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 1, 1981, pp. 683-692.
4. R.B. Peck. Deep Excavation and Tunneling in Soft Ground. Proc., Seventh International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, 1969, pp. 225-290.
5. A. Lyndon and A.N. Schofield. Centrifuge Model Test of a Short Term Failure in London Clay. Geotechnique, Vol. 20, No. 4, 1970, pp. 440-442.

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