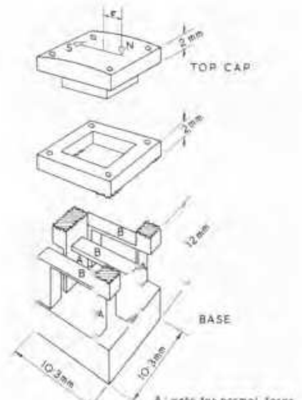
e
S
N
2 mm
TOP CAP
2 mm
B
B
A
B
A
12 mm
BASE
10.3mm
10.3mm
A webs for normal force
B webs for shear force

overall dimensions of the load cell itself are 1.02x1.02x1.63 cm and including the housing they are 1.44x1.44x1.98 cm. The pressure range for the load cells is 0-600 kN/m^2 for normal stress and 0-300 kN/m^2 for shear stress. On the basis of the calibrations it may be concluded that the error of measurement is less than five per cent of the applied stress.

The model was prepared by pouring the sand into the strong box in the longitudinal direction of the pipe from a hopper with approximately constant height of fall and rate of pouring. This method gave a uniform deposit around the pipe with a voids ratio of about 0.52 for a slow rate of pouring. The sand that was used was Leighton Buzzard sand (25 < B.S. sieve < 14). To reduce unwanted side friction in the strong box glass plates were used between the sand and the sides of the box.

SOME TEST RESULTS

The test program comprised different geometries of the trench, two different wall thicknesses of the pipe and dense and loose sand. Only two tests will be described in detail here and some of the results from a few others will be mentioned briefly. The geometry and the boundary conditions of the two tests are given in Fig. 2. Test HL 17 corresponds to what is generally known as the embankment case and test HL 25 to the ditch case.

The vertical and horizontal loads on the pipe vary linearly with acceleration as can be seen in Fig. 3. In test HL 17 the pipe is attracting more load than the hydrostatic overburden at the crown level because it is more rigid than the sand, while in test HL 25 the sand is hanging in the trench walls and the pipe therefore receives less load than the hydrostatic overburden. The sudden increase of the horizontal load in test HL 25 at 40 g is due to the development of cracks in the pipe wall resulting in an increase of the horizontal diameter.

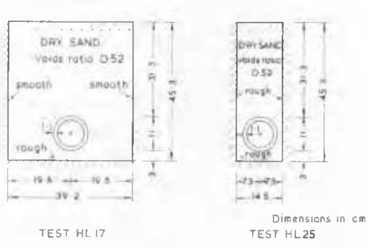

Fig. 2: Geometry and boundary conditions for tests HL 17 and HL 25.

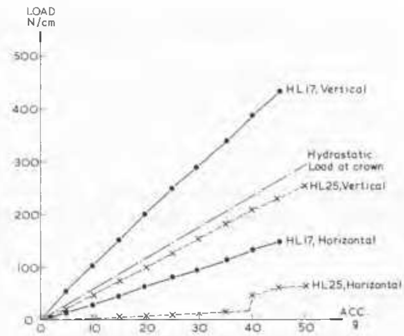

Fig. 3: Load on pipe as function of acceleration.

The radial earth pressure distributions on the pipe show that the pressure at the invert is significantly greater than the crown pressure (see Fig. 4). The pressure is slightly higher at 30^0 than at the crown and then it drops off to a minimum value around the springing, whereafter it increases steeply to a maximum value at the invert. The kinks in the curve for test HL 17 at 135^0 and 150^0 have been found in other similar tests also, and it is not thought to be an error in the measurement.

The redistribution of the pressure after formation of cracks in the pipe wall is clearly seen by comparing the curves for test HL 25 at 35 g and 40 g. When the pipe cracks the pressure at the crown and the

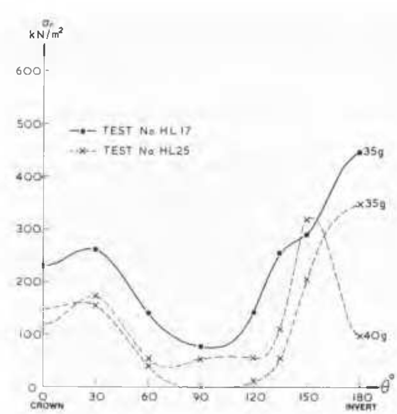

Fig. 4: Radial earth pressure distribution.

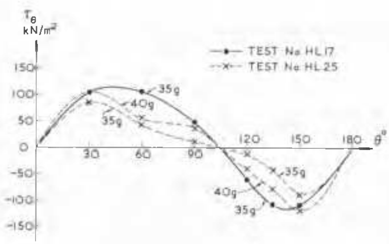

Fig. 5: Tangential earth pressure distribution.

invert drops, most drastically at the invert. At the same time the pressure increases around the springing and a maximum is reached at 150°.

The distribution of the tangential earth pressure is shown in Fig. 5. The sign convention is positive downwards and negative upwards. The distributions have maxima at about 45° and 140° in test HL 17, and at about 30° and 150° in test HL 25. The direction of the tangential pressure changes in both tests at about 105°.

The moment and the thrust in the pipe wall were calculated using the pressure distributions shown in Figs. 4 and 5 and assuming symmetry around the vertical diameter. Fig. 6 shows the moment distributions in the pipe wall. It is interesting to see that although the vertical load on the pipe in test HL 17 is about 1.8 times the load in test HL 25 the moments are virtually identical. The main reason for this is that while the horizontal pressure in test HL 17 is relatively large it is almost zero in test HL 25. The maximum moment is found at the invert in both tests.

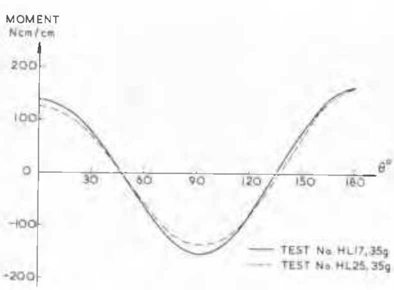

Fig. 6: Moment distribution in pipe wall.

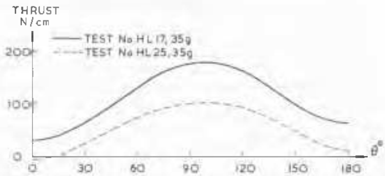

Fig. 7: Thrust in pipe wall.

The distributions of the thrusts are shown in Fig. 7. It can be seen that there is a small tension at the crown in test HL 25, this is caused by the relatively large shear stresses acting on the pipe.

To investigate the influence of the wall thickness a few tests were done with pipes with strain gauges on the wall. The wall thicknesses were 0.6 cm and 1.1 cm, and the geometry and boundary conditions were like in test HL 17 and the sand was dense (voids ratio = 0.52). A comparison between the tests shows that the moments in the thin-walled pipe were only about 50% of those in the thick-walled pipe. This suggests that there must be a considerable difference between the magnitude and/or the distribution of the earth pressure in the two cases, which is somewhat surprising as both pipes are quite rigid compared with the sand. An elastic analysis with reasonable Young's moduli for concrete and sand gives almost identical moments for the two wall thicknesses.

CENTRIFUGE TEST CONTRA CONVENTIONAL MODEL TEST

In a conventional model test with buried pipes the self weight of the soil will be substituted with a uniform surcharge on the ground surface. For comparison with the

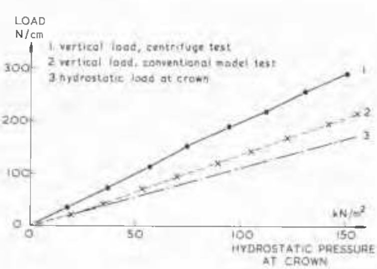

Fig. 8: Comparison between centrifuge test and conventional model test.

centrifuge tests such a test was performed, and part of the result is shown in Fig. 8. The applied surcharge was chosen so it would give the same hydrostatic pressure at the pipe crown as the centrifuge test. It can be seen that the vertical load in the centrifuge test is about 40% greater than in the conventional model test.It should be noted that the results are corrected for side friction in the strong box corresponding to a wall friction angle of 7^0 between sand and glass. The difference between the loads is surprisingly large and although some of it may be due to errors, there nevertheless seems to be a major difference between a centrifuge test and a conventional model test.

CONCLUSION

The tests have shown that it is possible to model soil structure interaction problems of this type in a centrifuge, and that there is a significant difference between results obtained in the centrifuge and results from a conventional test. Since in the conventional model the soil stress distribution is nominally uniform, whereas in the centrifuge the stresses increase linearly with "depth" (radius), i.e. similar to the prototype situation, it is concluded that the centrifuge test represents the real case more accurately.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the assistance of the staff of the Cambridge University Engineering Department during the course of their work and the British Science Research Council for the grant which they gave in aid of centrifugal model testing which made their work possible. The authors also thank the British Council, the Otto Mønsteds Foundation and Danmarks Ingeniørakademi with others for financial support of the junior author during the course of his work.

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