

## SESSION 3: BURIED STRUCTURES

### Papers:

#### VIBROFLOTATION OF CALCAREOUS SANDS

D.C. Andrews and D.B. McInnes, Vol 1, 45-52

#### A STUDY OF PIPELINE STABILITY WITH AN OSCILLATING WATER TABLE

P.J. Moore and P.M. Dight, Vol 1, 53-59

#### AN EXPERIMENTAL INVESTIGATION OF THE PHENOMENON OF PIPE JACKING

P.J. Yttrup, Vol 1, 61-63

Paper by D.C. Andrews and D.B. McInnes

Mr P.J. Wilcox said the Ministry of Works and Development had similar problems with vibroflotation in beach sands at Thames, NZ. He suspected the problem was related to the silt content, which reduced permeability. Mr Wilcox also thought that soils which could not be liquified by vibroflotation might liquify under seismic loading.

Prof Ladanyi asked if dynamic compaction was used and received the reply that this was not used as it was not practical.

Dr Parry said care was necessary in calcareous sands with tests such as SPT etc, in which current interpretations were based on experience with quartz sands. A different interpretation might be necessary in calcareous sands. His second point was that vibrations might be rapidly attenuated, if lateral stresses and lateral stiffness were low, as in calcareous sands. He said the vibroflotation method was attempted at another site in Western Australia, and was at least partly successful.

Paper by P.J. Moore and P.M. Dight

Mr G. Salt said that with heavy compaction, the pipe could be held down, preventing uplift. He also said that failure was approximately vertical, but that higher K values would cause failure to occur in other planes. He pointed out that the equations for determining vertical equilibrium and a rational K value (equations 3 and 5) inherently assumed that the critical failure planes would be vertical. It could be argued that K values greater than the rational value could be achieved by adequate compaction (say K = 1.0). Critical failure planes would then be inclined. Failure would thus occur with greater uplift than suggested in the paper, but would be brittle in nature. He agreed that caution would be needed when using K values greater than  $K_r$  for this reason. He wondered whether the authors had tried plotting  $K_v$  values on Fig.5 and whether they had tried model tests with compaction to K = 1.0 (say) induced in the overburden.

Prof Ladanyi continued the discussion at that point with the following comments. In their interesting study of upward pipeline jacking in sand, the authors had used the passive bin theory for estimating the

resistance of cover against the heaving of the pipe. He would only like to comment on equation 3 in the paper. The equation was a correct form of the passive bin formula, but it should be noted that it was valid only when the bin or the trench was limited by vertical rigid walls. If, on the contrary, there were no walls but only two nearly vertical failure planes passing through the sand mass, equation 3 lost validity, because it did not take into account the fact that the resultant stress acting at any point of the vertical failure planes could not exceed an upper limit which was given by the passive earth pressure of the mass located outside the vertical planes. As shown by Ladanyi and Hoyaux (1969), a more correct uplift resistance formula would be:

$$\sigma_v = \gamma z_B (1 + z_B \sin^2 \phi / 4B)$$

It should be noted, however, that for shallow depths of burial, ie, down to about  $z_B = 2B$ , which was the depth interval considered by the authors, the two formulas gave nearly identical results. The conclusions made by the authors were therefore correct, but for the pipes buried deeper, ie for  $z_B > 2B$ , equation 3a should be used because it yielded considerably lower uplift resistances.

### Reference:

Ladanyi, B and Hoyaux, B. (1969)  
A study of the trap-door problem in a granular mass; Canadian Geotechnical Journal, Vol 6, No 1; pp 1-14.

The authors were asked how the compaction of the fill was performed and was the intensity of compaction varied to see if the theory held. Dr Moore replied that vibration was used to compact the fill but that the failure surface was linear so he considered the methods appropriate. The clients wanted the tests carried out so as to represent normal pipe laying procedure, so the compaction effort was not varied.

Dr Johnston queried how the failure planes were identified during the tests. Dr Moore explained that bands of different coloured material were observed through the sides of a glass sided flume.

Paper by Mr P.J. Yttrup

Mr T.A.M. Dodd asked the author to indicate

the relative magnitude and frequency of pressure changes occurring in a gas pipeline. Mr Yttrup said the experiments simulated field procedure, in that pressures within the pipes fluctuated over 24 hours. This was directly related to consumption rates as gas inputs were constant. The pressures typically ranged between 5 and 7 megapascals.

Dr Poulos suggested that the mechanism of pipe jacking might be more a result of liquefaction. With cyclic loading over 24 hours, pore pressures would increase, resulting in liquefaction. This could mean that pipe jacking was directly related to pipe size and permeability of the soil. If

so, it would be an impossible situation to simulate in the laboratory. Mr Yttrup said they used beach sands, which had high permeability, in the experiments. He also said that the rate of pressure change within the pipes was slow, therefore pore pressures were unlikely to build up within the soil. Pore pressure build up did not occur within the laboratory tests; he doubted that it would occur in the field.

The author was then asked how much distortion of the pipe occurred. He replied that it depended on the pipeline internal pressure but was about the diameter of a sand grain, ie, 0.20mm.