For both the immediate post-installation test and the submerged sand tests, qv, the effective overburden pressure at the pile toe was about 5,0 kPa. By substituting this value into equation (7-4), the Nq values applicable to each test can be estimated as Nqa = 134 and and Nqb = 120, respectively. Triaxial tests carried out on specimens of the test sand at densities ranging from 16,4 $kN/m^3$ to 16,[illegible] $kN/m^3$ (Figures 7.2a and 7.2b) indicated that the internal friction angle of the undisturbed soil was of the order of 38°. For a soil with this friction angle, the Skempton et al (1953) Nq factor has a value of about Nq = 160. Although somewhat high, this agrees reasonably well with the calculated model test values given above. The Coyle and Castello (1981, p.979) Nq value for a pile installed to a depth with an L/d ratio of 12,5 in a sand with a friction angle of 38° is about Nq = 75. Coyle and Castello's data was obtained mostly from driven pipe piles. This, and scale effects (de Beer [1963], Meyerhof [1983]) as noted above, may have contributed to the discrepancy between the model test Nq, and Coyle and Castello's empirical Nq. Meyerhof (1983) presents a method of calculating ultimate unit point resistances for piles in sand. Using this method, and various pile load test data, he defines a "reduction factor" Rb, where Rb is the ratio of observed ultimate unit point resistance, qpo, to the calculated ultimate unit point resistance, qpt. Thus:

$$Rb = qpo/qpt$$

In investigating scale effects in piles, he then presents a diagram (op. cit., Figure 4, p.802) relating the reduction factor to pile base diameter. The diagram shows that with decreasing pile base diameter the reduction factor increases. By extrapolating the data in Meyerhof's diagram to a pile toe diameter of 0,06m a reduction factor of between 1,5 and 2,0, say 1,75, can be estimated. Most of the piles used by Coyle and Castello (op.cit.) for their empirical design correlation for Nq had a toe diameter of about 0,5m. Pile base diameters of 0,5m obtain on Meyerhof's diagram a reduction factor value of 1. In order to correlate Nq from the model pile test with Coyle and Castello's proposed Nq, it is therefore suggested that the model test results should be modified by a reduction factor of 1,75. When this is applied to the previously noted model test Nq values of Nqa = 134 and Nqb = 120, the revised Nq values become Nqa = 77 and Nqb = 69.

Due to possible lateral stress relief which may have occurred when draining the sand box of water before carrying out the "drained" static load test (see Section 6.3.5), it is considered that comparison of a calculated Nq for this test with Coyle and Castello's data is not valid.

Until an improved theoretical model for calculating pile and bearing resistance is developed, it is suggested that empirical design correlations for Nq, modified by Meyerhof's reduction factor may constitute the most reliable approach to the estimation of pile end bearing capacity. The model pile test results support this suggestion.

## 7.4 Comparison with Field Data

All the factors affecting the bearing capacity of the model MV pile noted in 7.2 and 7.3 cannot at this stage be compared with the bearing capacity characteristics of full-scale MV piles as no suitably instrumented tests have been carried out. A rough indication of the development of high skin friction capacities may be obtained by reference to the results of a number of pile load tests. Table 7.4 presents a summary of some load test results. The estimated quantities given in this table were obtained as follows:

1 Toe capacity in sand estimated using Coyle and Castello's (1981, Figure 6) empirical correlations for Nq and L/d.

2 Toe capacity in clay estimated from the undrained cohesion of the soil, "C", and the bearing capacity factor Nc, where Nc = 9.

Table 7.4 Summary of some MV pile test load data.

Compression tests

| Locality | Durban City | Durban Bayhead 1 | Durban Bayhead 2 | DUBAI U.A.R. | Durban Wentworth |
|---|---|---|---|---|---|
| Shaft length m | 8,7 | 14,1 | 11,5 | 6,0 | 19,5 |
| Lateral dimensions mm | 3000 | 2270 | 4000 | 4000 | 3000 |
| Failure load kN | 800 | +2000 | +2200 | 1700 | 1500 |
| Estimated toe capacity kPa | 1800 | 750 | 5100 | 8600 | 5300 |
| Estimated toe load kN | 127 | 39 | 651 | 1080 | 375 |
| Estimated shaft load kN | 673 | +2000 | +1548 | 620 | 1125 |
| Average shaft friction kPa | 82 | +156 | +107 | 82 | 126 |
| Average grout pressure kPa ($\gamma d$ = 20 kN/$m^3$ $\gamma sub$ = 10 kN/$m^3$) | 43,5 | 70,5 | 57,5 | 30 | 100 |
| Estimated average shaft friction angle ( $\delta = \phi$) | 34° | 38° | 37° | 40° | 35° |
| Estimated Ks | 2,8 | +2,8 | +2,7 | 3,3 | 1,8 |
| Ratio of Ks/Kp | 0,79 | +0,67 | +0,67 | 0,71 | 0,49 |
| Average soil profile | Loose | Medium dense | Medium dense | Dense | Loose/ medium dense |
| Assumed average dry density of soil kN/$m^3$ | 14 | 15 | 15 | 16 | 15 |

3 Soil friction angles obtained from Meyerhofs (1976, Figure 2) correlation with static cone penetrometer resistance.

4 Soil densities were estimated using data from Bowles (1982 p.100), Tables 3-2 and 3-3.

For the calculation of skin friction capacities, it was assumed that the correct average overburden pressure to use was that due to the hydrostatic pressure of the grout column.

Meyerhof (1956, Figure 4) presents data relating average unit skin friction capacities of driven piles to static cone penetration resistance. Comparison of the data in Table 7.4 to Meyerhof's data indicates that the average unit skin friction capacity of MV piles is of the order of 2X to 3X that of preformed driven piles. The shaft friction stresses in Table 7.21 are similar to those given by Jelinek (1962).

On the basis of the data in table 7.4 a preliminary recommendation can be made for the design of MV piles in relatively uniform cohesionless sands. It is recommended that average shaft capacity can be estimated conservatively from the equation:

$$fs = \overline{qv}.Ks.\tan \phi$$

where $\overline{qv}$ is the average hydrostatic pressure of the uncured grout column, Ks is half the possible lateral earth pressure coefficient, Kp, and ∅ is the average internal friction angle of the soil obtained using the correlation diagram given by Meyerhof's (1976, p.200) Figure 2. Due to possible lateral stress relief and soil creep with time it is suggested that Ks should not exceed the proposed value. For determination of pile end bearing capacity, it is recommended that Coyle and Castellos (1981, p.979) Figure 6 be used to obtain the Nq bearing capacity factor. Meyerhof's (op.cit.) Figure 2 may also be used to estimate ∅ at the pile toe. Since MV pile toe diameters typically lie within the range 0,3 to 0,5m, Nq does not need to be adjusted by a reduction factor.

## 7.5 Summary

1 The shaft friction capacity of the model pile has been investigated. It was found that high lateral earth pressures were developed which were close to passive lateral earth pressures at failure in a soil mass.

2 A possible explanation for the phenomenon of "arching" near the pile toe reducing the skin friction capacity in this region, was obtained. The "arching" may be due to interference of stress fields originating against the pile shaft and at the pile toe.

3 The slip-line field around the toe of the model pile does not resemble any of the conventional slip-line fields developed from limiting equilibrium principles. It is approximately similar, however, to the slip-line field developed around an expanding cavity in an elastic solid.

4 Slip on the surface of the pile may not occur on a plane of maximum stress obliquity.

5 Preliminary design recommendations for MV piles in relatively uniform cohesionless soils were made.

# 8 SUMMARY AND RECOMMENDATIONS FOR FURTHER WORK

## 8.1 Summary

An overview of piling in general with brief comments on pile design approaches has been presented along with notes on the history and details of the MV pile type. Test equipment for the installation and loading of a model MV pile in the laboratory has been described. A major feature of this equipment was the use of a semi-circular half pile installed against a glass panel which formed one side of a box containing the test sand. The equipment was used to provide a visual record of the displacement of sand particles around the model on an approximate plane of symmetry, as well as information on the response of the pile to both static and dynamic loading. Results of one of the tests have ben reported in detail and discussed.

In this test, the model pile was installed to a depth with a penetration ratio of about L/d = 12,5, and into a sand with an average dry density of 1673 kg/m$^3$ and a friction angle of $\phi = 38^{\circ}$. Four types of load test were carried out, most of the testing being done in submerged sand. These included: (a) Measurement of impact loads during pile installation; (b) measurement of static toe load immediately following installation; (c) measurement of load distribution in the pile during loading in submerged sand, and (d) a repeat of test (c) after draining the water from the sand box. Due to frictional contact between the half pile and the glass side of the sand box, loads measured in the pile shaft had to be corrected to obtain an estimate of the loads due to pile/soil interaction only. Draining the sand box of water prior to conducting the final load test apparently resulted in significant stress relief. Loads measured during this test are therefore thought to have little practical value.

Soil displacements were obtained using a photogrammetric technique. Measurements of the horizontal and vertical displacement components at discrete penetration increments of the pile, of a large number of individual sand grains located near the pile toe, were presented in the form of contour diagrams. The accuracy of the displacement measurements was limited to about 25 microns. Contour diagrams of various soil strain components, calculated from the displacement data, have also been presented. The data suggests that there may be loss of axial symmetry conditions near the point of the pile shoe.

Analysis of the model test data leads to the following observations:

1 The "wave equation" could be used to predict pile "set", but not shaft stress or the scale of the model pile.

2 The lateral radius of the zone of soil disturbed by pile movement during both pile installation and static load testing was about 3,6 pile diameters. Disturbance below the pile toe occurred to a depth of about 4 pile diameters during pile installation, but to about 2,5 pile diameters during static loading.

3 Contours of principal strains indicated a correlation with the geometry of the pile shoe. The volumetric strain diagrams showed that in general 3 zones of similar volumetric changes can be identified around the pile toe. These include an outer compressive zone, a central dilatant zone and an inner zone of complex volume changes.

4 Unit shaft friction stresses were found to follow a triangular distribution pattern along the pile shaft. The estimated stresses indicated that high lateral earth pressures were developed against the pile shaft. The calculated lateral earth pressure coefficients approached the value of the passive lateral earth pressure coefficient. It was

suggested that the preservation of high lateral stresses was probably due to the method of pile manufacture.

5 A diagram of a possible slip-line field around the lower region of the pile was presented using calculated principal strain direction data. The proposed shear pattern differed considerably from "conventional" slip-line field patterns assumed in the determination of the Nq bearing capacity factor, but approximately resembled the pattern developed around an expanding cavity in an elastic solid. Interference of slip-lines originating at the pile shaft with slip-lines originating at the pile toe was observed. Slip on the pile surface may not occur on a plane of maximum stress obliquity.

6 A preliminary design method for MV piles was proposed on the basis of the model test results and selected field test data.

## 8.2 Recommendations for Further Work

Despite the known limitations of model tests, it is believed that further investigations of a similar type to that described here would prove worthwhile. A minimum supplement to the work reported here would be to install the same pile into the same sand at a depth given by an embedment ratio of about L/d = 20. Further, in order to obtain more accurate data on the load distribution down the pile shaft, it is suggested that for each half pile test a complementary and duplicate circular pile test be conducted in the centre of a proportionately larger sand box. It has been shown that when the water was drained from the sand box stress relief occurred in the soil influencing the pile load capacity. This method of attempting to investigate the pile capacity at greater confining pressures is therefore to be discouraged. A more efficient method of measuring sand grain displacements needs to be developed in order to improve the resolution of displacement measurements in areas of high displacement gradients.

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# APPENDIX A

## A.1 Measurement of Grain Displacements

The principles of the method used for measuring the displacements of the sand particles around the model MV pile have been given in section 4.7. This appendix provides further details of the method.

The reference grid drawn onto the external face of the glass panel of the sandbox (see Figures 4.2 and 4.9), was 270mm square with grid lines at 30mm intervals dividing the region around the pile toe into 81 squares. Letraset members were placed at the intersection of the grid lines. When viewed in the stereoplotter, certain positions on these numbers were selected as stationary control points to which the displaced sand could be referred (see Figure A.2). The camera used to record the sand patterns around the model pile was a Super Cambo III with a Voigtlander Braunschweig Apu-Skopa 1:9/21cm lens. An extension was added to the rear of the camera to enable it to accept 230 X 240mm sheets of Kodak Plus X film. The model pile and surrounding soil, were illuminated during exposure of the film, by two flash units placed either side of the camera lens and inclined at 45° to the camera axis. The entire area lying between the horizontal bracing supporting the glass panel of the sand box and between the pile axis and the side of the sand box (see Figure 4.2), some 340mm x 282mm, was recorded on the plate film.

The Butterfield et al (1970) stereophotogrammetric method for measuring planar displacement fields, requires stereo superposition of photo-pairs to express the displacements as height in the stereo image. Where the trajectory of the soil displacement has components of movement along both the x and y axes of the stereo plotter, corrections discussed by Luker (1984) may have to be applied if removal of, for example, the y parallax, interferes with measurement of the x parallxx.

The method used here, however, does not rely on stereo superposition of the photo-pairs. Instead the Wild A7 Autograph (Wild Heerburgg [1957]) was used simply as a rather sophisticated optical micrometer to measure the co-ordinates of the control points and selected sand grains in each photograph. (The technique was originally applied by Roscoe et al [1963a] who used an "Orthogonal displacement meter" to obtain displacement components on X-ray plates of discrete lead shot markers embedded in the test soil.) Accordingly both left and right plotter cameras of the Wild A7 Autograph were given the same bz, omega and phi settings which remained fixed throughout the series of measurements (see Figure A.1). For convenience the settings selected were the so-called "normal" settings of 100mm, 100 grads and

100 grads respectively. In this position, the picture carriers lie in the same orientation and elevation above, and parallel to, the plane of the x and y carriages. The z axis was nominally set at 300mm, and the focal lengths of the plotting cameras, at 200mm. These settings also remained constant during the measurements, as did the settings of the X and Y carriage counters.

The first exposure in the sequence of photographs (e.g. Plate 1 Dyn) was placed emulsion side down in the left hand plotting camera with the toe of the pile pointing towards the observer so that the axis of the pile was parallel to the axis of the y carriage. The next photograph was placed in a similar position in the right hand plotting camera. Coarse misalignments between the orientation of the photographs as placed in the left and right plotting cameras were removed by applying b'x, b'y and kappa adjustments. Then using the X and Y carriages only and working square by square through the reference grid, the co-ordinates of the 4 control points defining each square were measured a minimum of 3 times, while between 15 and 36 selected sand grains were co-ordinated, once each in both photographs. Control points common to adjacent squares in the reference grid were co-ordinated up to 12 times in this way and about 25 co-ordinate pairs were typically obtained for each square measured on each photograph. All data was