A STUDY OF LOAD BEARING CAPACITY OF PILES DRIVEN IN THE ZANZIBAR HARBOUR SUBMARINE SOILS

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1. ABSTRACT

The rehabilitation of the Zanzibar Port involved, among other things, driving piles into the submarine soils. Pile driving fractures the coral and as a result creates a gap filled with loose fractured coral between the unfractured coral and the pile. Grouting of the external surface of the pile is done to strengthen the fractured coral. In this paper, two grouted working piles (TP6 & TP61), randomly chosen from the pile group, were load tested to twice the design load. The test results indicate adequacy of the bearing capacity of the grouted piles.

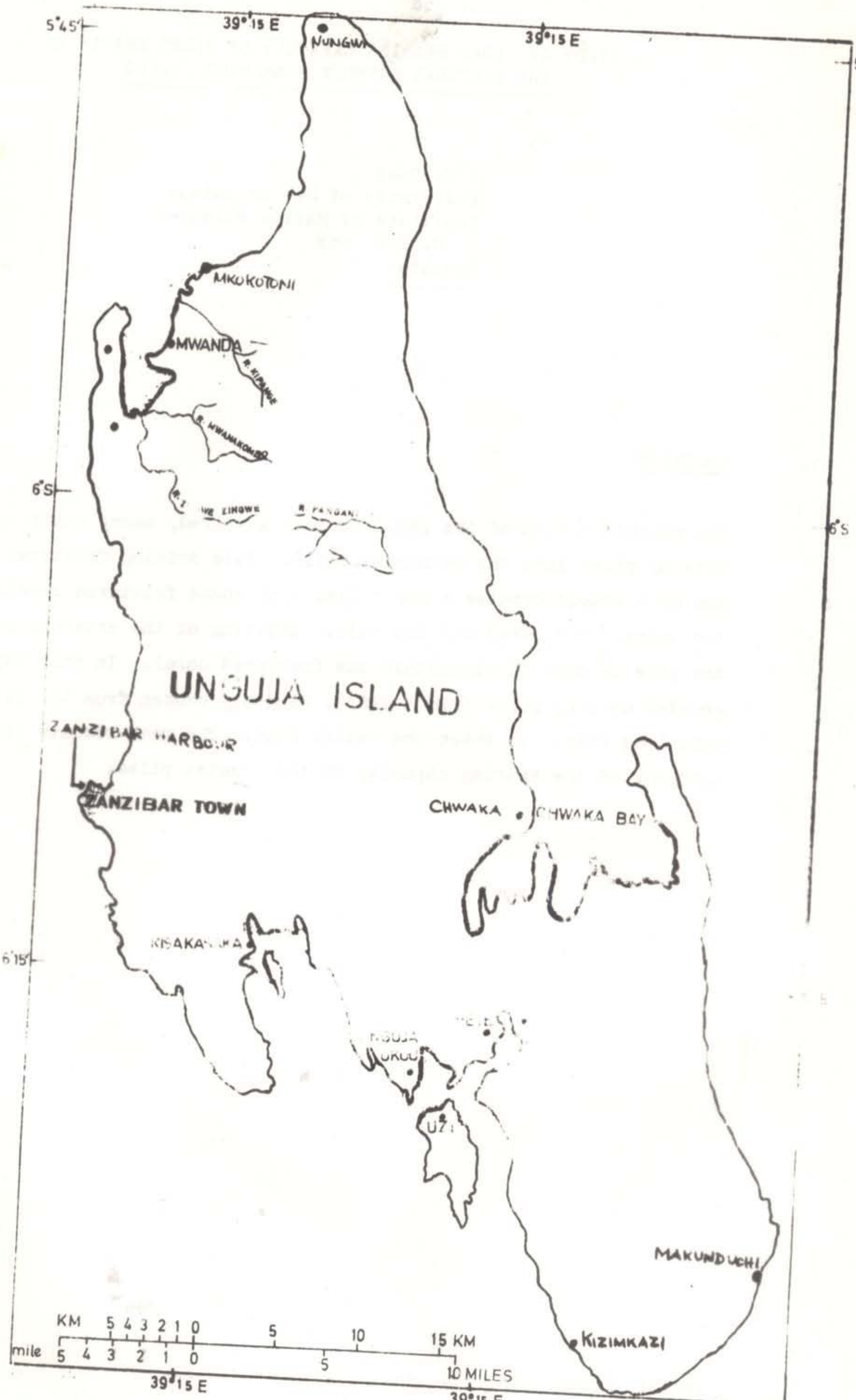


Fig 1 Location of Zanzibar Harbour

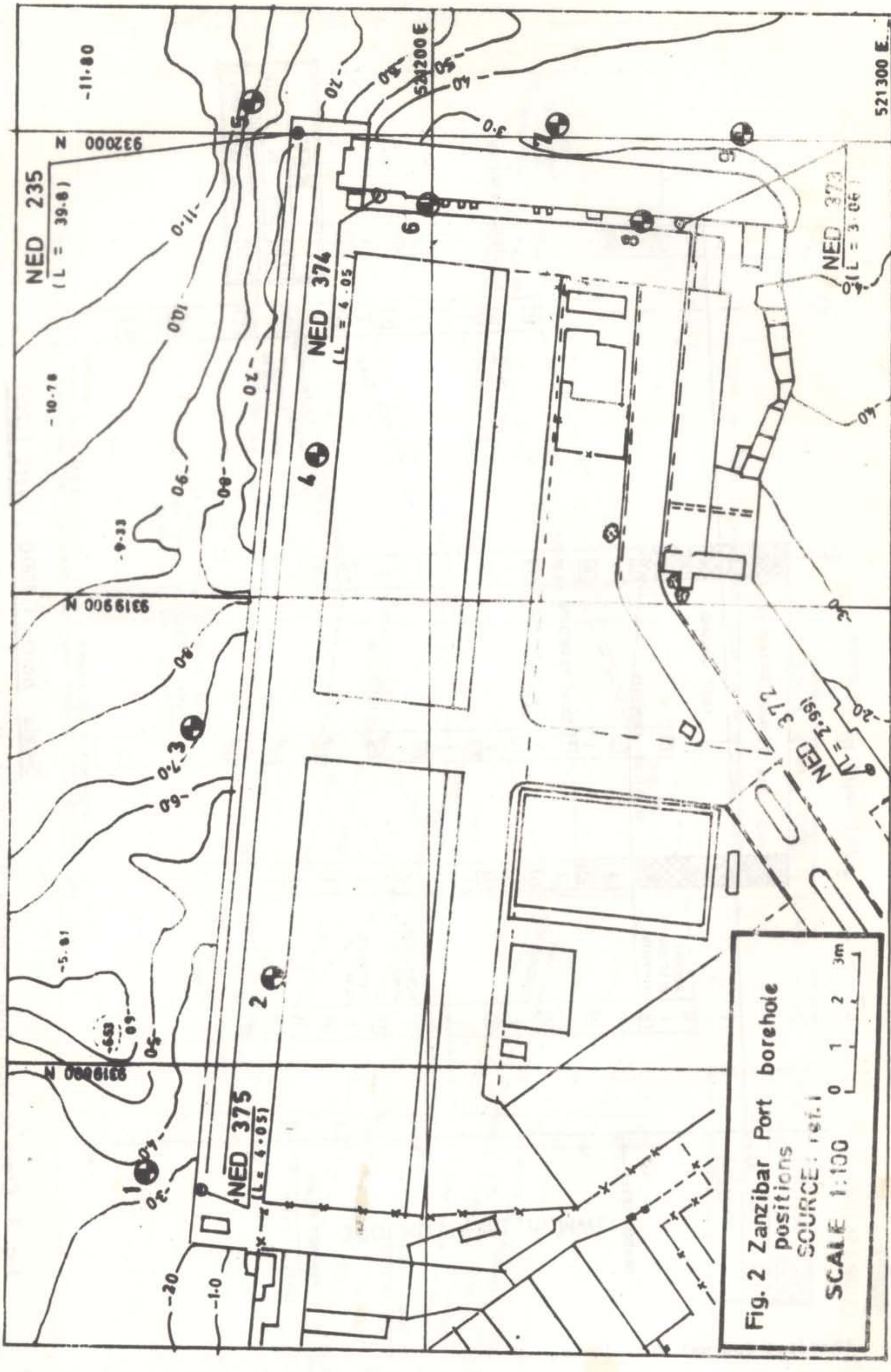
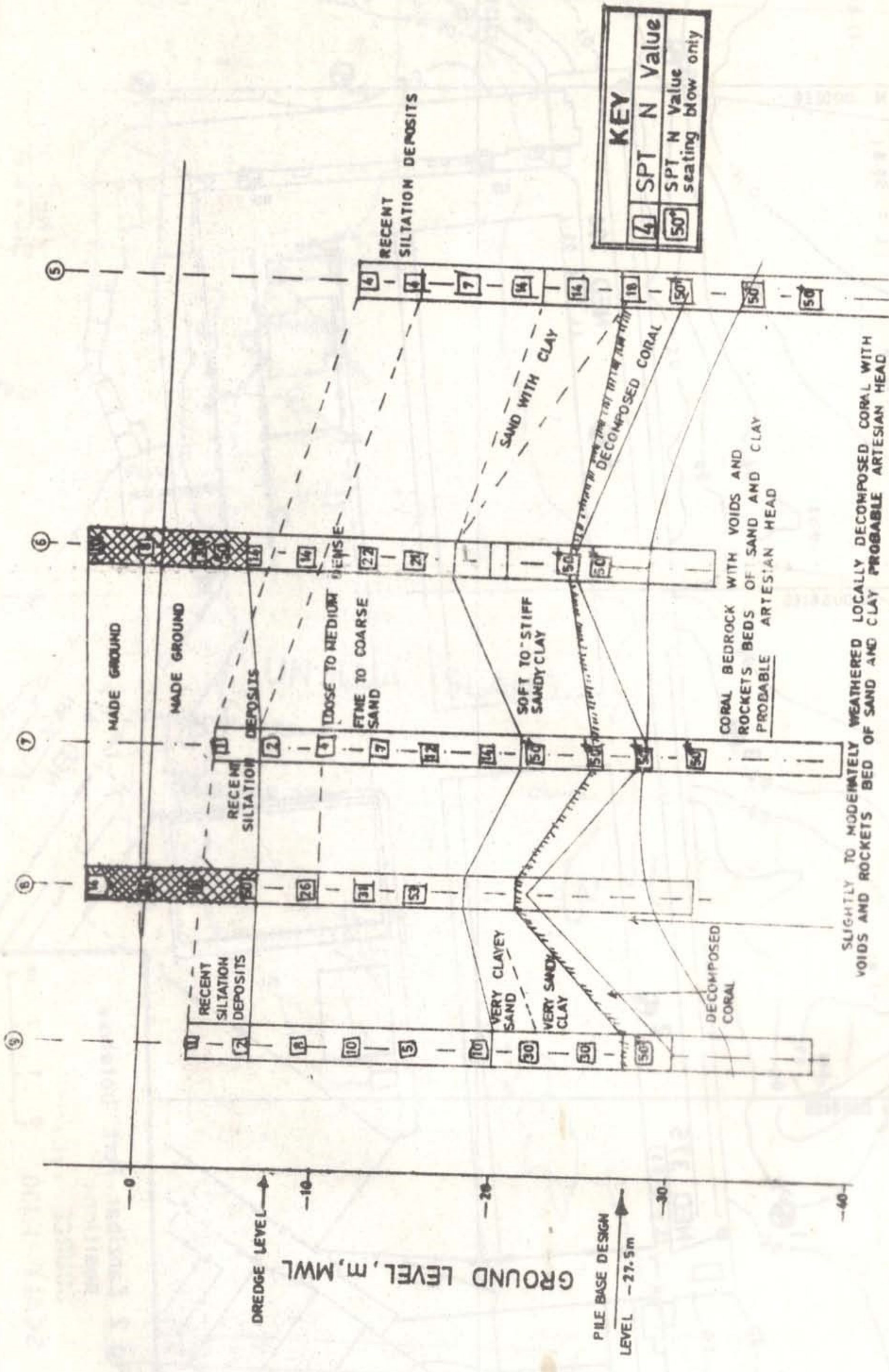


Fig. 2 Zanzibar Port borehole positions
SOURCE: ref.1
SCALE 1:100



Scale: horiz 1:1000 , vert 1:200

Fig. 3 Cross section from borehole 9 to 5 at North wharf, Zanzibar
Source: ref. 1

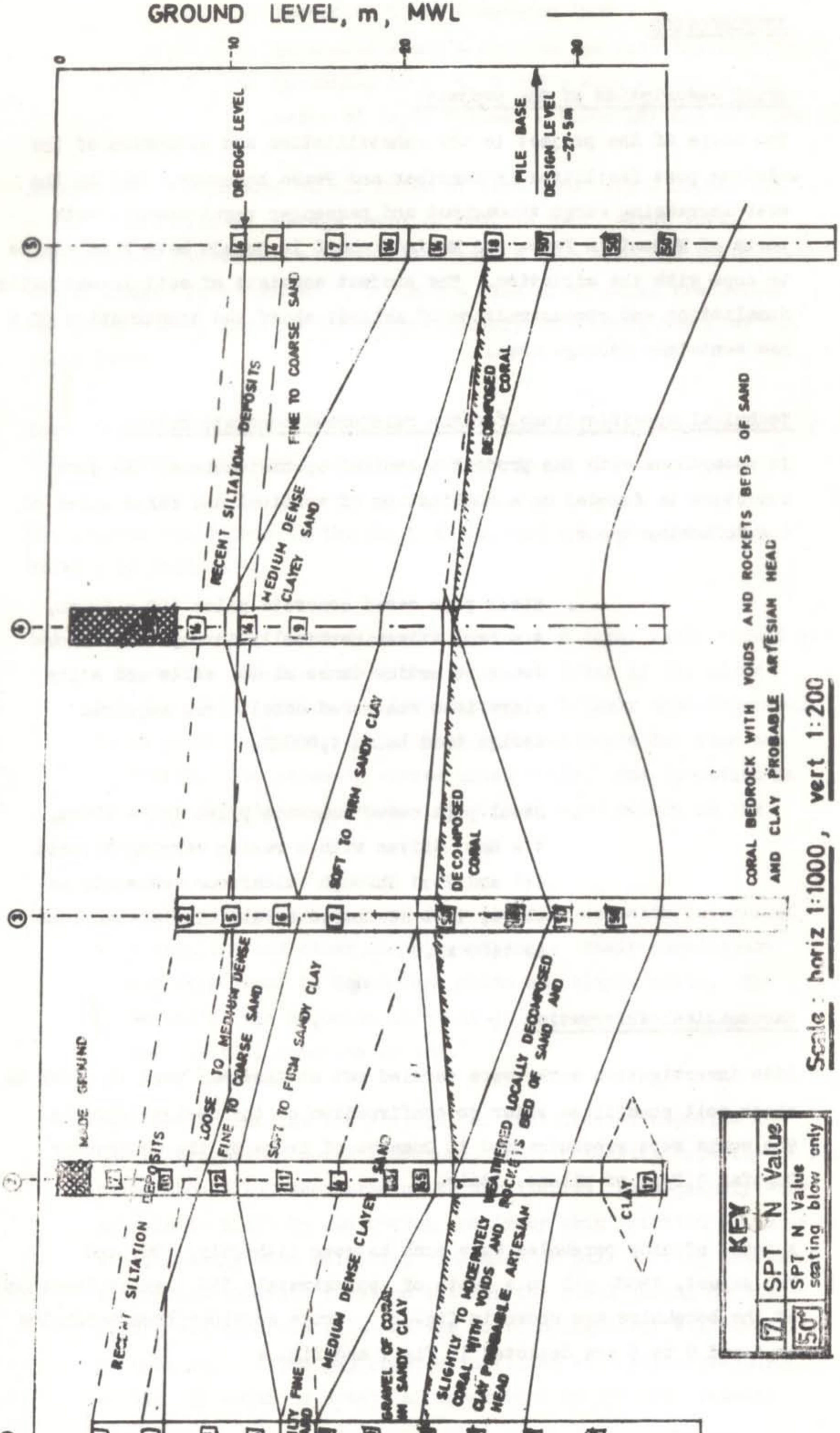


Fig. 4 Cross section from borehole 1 to 5 at West wharf, Zanzibar
Source : ref.

2. INTRODUCTION

2.1 Brief description of the project:

The scope of the project is the rehabilitation and extension of the existing port facilities in Zanzibar and Pemba harbours. Due to the ever increasing cargo throughput and passenger requirements, both ports of Mkoani in Pemba and Malindi wharf in Unguja have been unable to cope with the situation. The project consists of soil investigations, demolitions and reconstruction of Malindi wharf and construction of a new container storage area.

2.2 Technical specifications for the reinforced concrete piles:

In accordance with the project technical specifications, the port structure is founded on a combination of vertical and raked piles of the following types:

- Steel pipe cased concrete piles (OD = 600mm, t = 8mm) driven vertically through calcareous loose to medium dense sands, silts and silty clays into weathered coral. The required design load being 1,000kN.

- Steel pipe cased concrete piles (OD = 700mm, t = 8mm) driven with a raking varying between 4:1 and 15:1 through calcareous sediments as above, into weathered coral. The design load is 1400 kN.

2.3 Geotechnical information

Site investigation works were carried out at Zanzibar port in order to check soil conditions prior to confirmation of the design criteria. The works were subcontracted to Comarco of Kenya by the Contractor Cogefar S.P.A. of Milano, Italy.

A total of nine boreholes were sunk between 11th July, 1988 and 7th August, 1988, all to a depth of approximately 35m. Exact locations of the boreholes are shown in fig. 2. Cross sections from boreholes to 5 and 9 to 5 are depicted in fig 3 and fig. 4

- Boreholes 6 and 8 were on land
- Boreholes 2 and 4 were on the existing West Wharf
- Boreholes 1, 3, 5, 7 and 9 were offshore, in water of depth between - 2.86m MWL and - 10.08m MWL

Bedrock was encountered in all holes at levels between - 20.69m MWL and - 27.11m MWL with an average level of -23.71m MWL. Water level measurements fluctuated with the tide and as such were not indicative of anything. In boreholes 1, 3 and 9 a flow of artesian freshwater was noted coming from the coral, particulary BH 9 where the flow was quite large.

Laboratory tests were done by the central Testing Laboratories Ltd. of Nairobi, Kenya. All tests were carried out to BS 1377.

The general conditions in the Zanzibar harbour area are described briefly as follows:-

(a) Coral limestone consisting of older buried coral reefs formed at a time when the sea level was lower than today is the major stratum in the area. The formation is extremely nonhomogeneous. It is porous with cavities filled with carbonate detritus and sediments. Its strength varies extensively. The formation is found at elevations between - 17.00 and 20.00 meters in the boreholes.

(b) The coral limestone is covered by alluvial deposits intermixed with detritus and other marine sediments. Their consistency varies from loose to dense sand silts and clayey silts. The thickness of the deposits is about 10 meters. The strength of the alluvial deposits is low.

Bearing in mind the low strength of the alluvial deposits and the heterogeneous coral limestone, the tip bearing capacity of the piles should be considered quite unreliable. Consequently, the pile loads shall be undertaken mostly by skin friction which is statistically more reliable. Inasmuch as pile driving fractures the coral, the skin friction between the steel casing of the reinforced concrete pile and the surrounding coral will usually be very small. The gap created is filled with loose fractured coral. In order to create a higher bonding strength between the non-fractured coral and the pile, two methods could be used.

(a) to construct a concerete friction pile by either uncased boring or by cased boring and subsequent removal of the casing while concreting thus creating a direct contact between concrete and coral.

(b) to grout the external surface of the steel casing and hence eliminating the gap created by fracturing the coral while driving the pile.

According to experience (2), skin friction of loose fractured coral is typically 10 to 20 KN/m^2. In either of the two methods suggested above, an allowable skin friction of about 150 KN/m^2 can be obtained. With this value of skin friction the required minimum length (L) of pile to be embedded in coral is L=3.6m for the vertical piles (OD = 600mm) and L=4.3m for the raked piles (OD=700mm).

2.4 Pile driving, casting and grouting

Pile 61 which is typical of other vertical (600mm, close-ended tubular) pile was driven on 21st July 1989 using a 4.9T drop hammer, dropped through 1.5 meters. The blow count for the last 25cm was 50 and the supervisor agreed to casting and grouting the pile. The concrete strength was of grade A mix ($25N/mm^2$ - cube strength 28 days after mixing). Grouting was done by injecting water/cement mix through four pipes welded externally at 90^o. The total grout intake was 1100 litres. For both concreting and grouting, Sulphate resisting portland (C_3A) cement was used. Other pile data are follows:-

• Total length of casing	33.31m
• Cut-off level	+ 2.50
• Sea bed level	- 1.60
• Coral level	- 30.81
• Embedded length	29.21m
• Pile weight	268.6KN

- Soil condition: see fig 4 (near BH2)
- Steel casing of 8mm thickness throught except bottom wall of 12mm thickness, 2.5 meters high and bottom

plate of 18mm thickness.

Mean Water Level (MWL) is 2.08 metres above Admiralty chart No. 3211.

3 **LOAD TESTING**

3.1 **Load testing programme and procedure:**

The testing equipment comprised of a hydraulic jack and a load measuring device mounted on the pile head. Dial gauges were read from a position on the existing wharf about 30 metres from the kentledge stack. A scale was attached to the pile and readings were made using a level. Kentledge was created by stacking concrete blocks on the existing wharf. A schematic arrangement of kentledge is shown below. See fig. 5

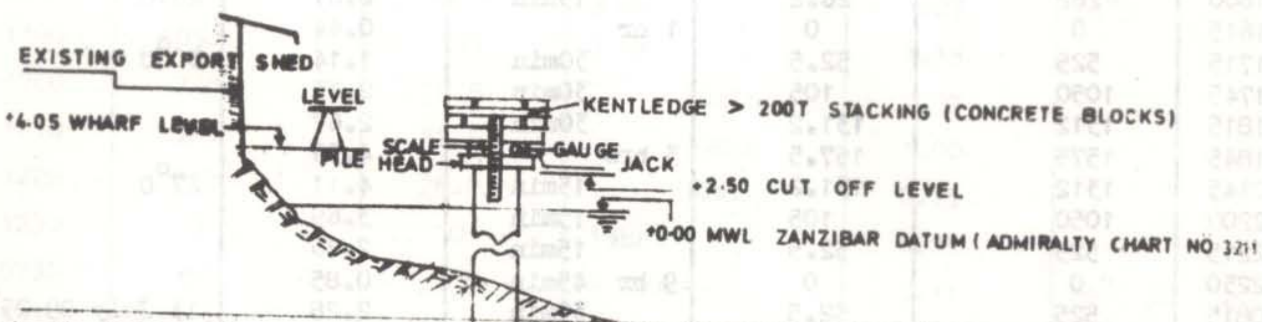


Fig 5 Schematic arrangement of Kentledge

The loading test was made on 29th August, 1989 by jacking down against the kentledge. The test procedure was pursuant to clause 11.3 (testing programme and supervision) of the Technical specifications of the Tender documents. The loading and unloading was carried out in stages as shown in tables 1a and 1b. The net settlement for TP6 was 4.13mm

Table 1a:

SITE:	MALINDI WHARF
DATE:	12 July 1989 to 13 July 1989
PILE NO:	TP6
DESIGN LOAD (DL):	1000 KN
REINFORCEMENT:	14 Ø 25
SIZE:	Ø 600 x 8mm

TIME	APPLIED LOAD (KN)	PERCENTAGE OF DL (%)	MINIMUM TIME OF HOLDING	SETTLEMENT (mm)	REMARKS
0800	262	26.2	30min	0.80	12 July 89 27°C
0830	525	52.5	30min	1.37	
0900	788	78.8	30min	1.96	
0930	1050	105	6 hrs	2.45	
1530	788	78.8	15min	1.75	
1545	525	52.5	15min	1.26	
1600	262	26.2	15min	0.61	29°C
1615	0	0	1 hr	0.44	
1715	525	52.5	30min	1.14	30°C
1745	1050	105	30min	2.13	
1815	1312	131.2	30min	2.67	
1845	1575	157.5	3 hrs	4.33	
2145	1312	131.2	15min	4.11	27°C
2200	1050	105	15min	3.69	
2215	525	52.5	15min	2.76	
2230	0	0	9 hr 45min	0.85	
0815	525	52.5	30min	2.28	13 July 89 25°C
0845	1050	105	30min	3.33	
0915	1575	157.5	30min	4.63	
0945	1837	183.7	30min	5.89	
1315	1837	183.7	15min	9.25	
1015	2100	210	3 hrs	9.64	
1330	1575	157.5	15min	8.74	
1345	1050	105	15min	7.95	
1400	525	52.5	15min	6.65	29°C
1415	0	0	3 hrs	4.13	
1715	0	0	-	4.13	

Table 1b:

SITE:	MALINDI WHARF
DATE:	29 August - 30 August 1989
PILE NO:	TP61
DESIGN LOAD (DL):	1000 KN
REINFORCEMENT:	14 Ø 25
SIZE:	Ø 600 x 8mm

TIME	APPLIED LOAD (KN)	PERCENTAGE OF DL (%)	MINIMUM TIME OF HOLDING	SETTLEMENT (mm)	REMARKS
0800	262	26.1	1 hr	0.20	25^oC 29 Aug.89
0900	525	52.5	1 hr	0.71	
1000	788	78.8	1 hr	1.27	26^oC
1100	1050	105	1 hr	1.35	
1200	788	78.8	10min	1.07	
1210	525	52.5	10min	1.00	
1220	262	26.2	10min	0.79	
1230	0	0	19hr	0.25	
0730	1050	105	6 hrs	1.71	25^oC 30 Aug.89
1330	1312	131.2	1 hr	2.06	27^oC
1430	1575	157.5	10min	3.13	28^oC
2030	1312	131.2	6 hrs	1.88	
2040	1050	105	10min	1.86	
2050	788	78.8	10min	1.43	
2100	525	52.5	10min	0.68	
2110	262	26.2	45min	0.40	
2120	0	0		0.36	
2220	0	0		0.36	

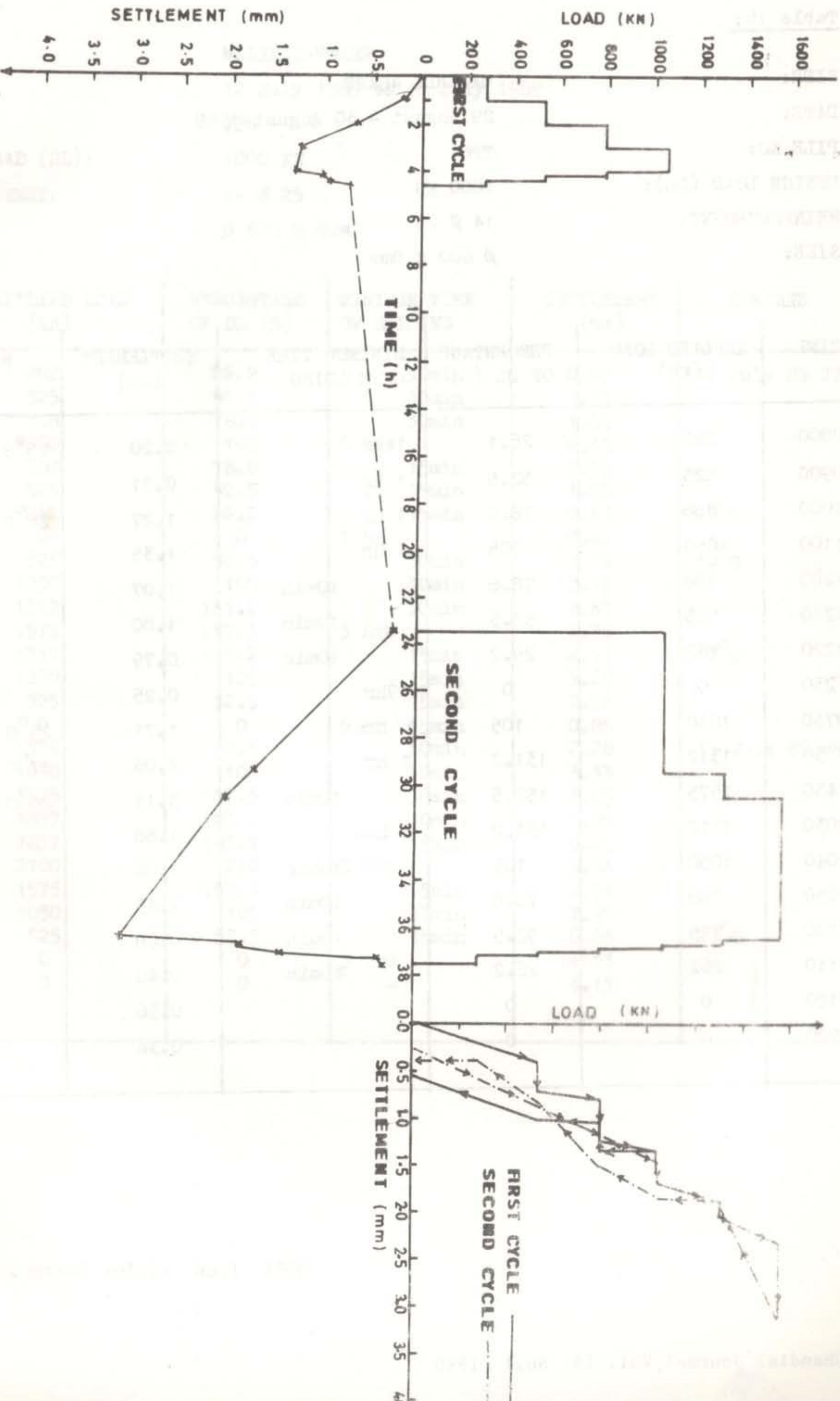


Fig. 6 Composite arithmetic plot of Load/Time and Load/Settlement. Pile 61

against a test load twice the design load. The net settlement for TP 61 was 0.36mm against a test load one and half times the design load.

3.2 Test result and design predictions

Test results have been presented in tables 1a and 1b whereby the net settlements for TP6 and TP61 were 4.13mm and 0.36mm respectively. Some of the criteria for defining failure of the pile have been stated (4)as follows:

- settlement continues to increase without any further increase of the load.
- an increase in settlement disproportionate to the increase in load.
- a plastic yielding net settlement of 6mm.

In both tests, the net settlement has been less than 6mm. and little healing effect was observed.

Brief theoretical considerations will give us a comparison of the test results with design predictions. If we assume that the bearing capacity will depend entirely on skin friction, the ultimate bearing capacity of, for example, TP61 can be found by applying Vesic's (1967) formula thus:

$$Pu = \int_o^L c\, \sigma v' \;\; Ks \tan \phi a \; dz + A_b \;\; \sigma vb \; Nq - W \qquad (1)$$

where

$C = 1.88m$ is the circumference of pile

$\sigma v' = Zc x \gamma s$ is the effective vertical stress

$Zc = 3.3m$ is critical depth $= 5.5 \times$ diameter of pile

$\gamma s = 9.8\ KN/m^3$ is density of submerged sands.

$\sigma v' = 3.3 \times 9.8 = 32.34\ KN/m^2$

$Ks \tan \phi a\ 1 = 1.1$ (test results diagrams of Vesic (1967))

$\phi a = 28^o$ is the angle of shearing resistance prior to driving derived from average SPT, N = 8-10

$A_b = 0.283m^2$ is the area of base of pile

$Nq = 60KN/m^2$ is the bearing capacity factor for base resistance

$\sigma vb = v' = 3.3 \times 9.8 = 32.34\ KN/m^2$ is the effective vertical stress at base.

W = 268 KN is the weight of pile.

Substituting the above values into equation (1) we have:

$$\int_0^{L_c} \sigma_{v'} K_s \tan\phi_a \, dz = 1.88 \left(\frac{32.34}{2} \times 3.3 + 32.34 \times (29.21-3.3) \times 1.1\right) = \underline{\underline{1,843 \text{ KN}}}$$

$$A_b \sigma_{vb} N_q = 0.283 \times 32.34 \times 60 = 549 \text{ KN}$$

$$= 268 \text{ KN}$$

$$Pu = 1843 + 549 - 268 = \underline{\underline{2,124 \text{ KN}}}$$

Other studies on calcareous sands show that such sands may show friction angles of 35^o or more but have been found to provide inferior support for driven piles than normal silica sands. Mc Clelland (1974) suggested limiting skin resistance of 19 KN/m^2 and base resistance of $4,800 KN/m^2$. In this case the expected bearing capacity (by friction only) of our piles is $P = \Pi \, d \times 29.21 \times 19 + \frac{\Pi \, d^2}{4} \times 4800 = \underline{\underline{2,403 KN}}$

4. **DISCUSSION AND CONCLUSION:**

The evaluation of ultimate bearing capacity of piles driven in the Zanzibar harbour has been based upon test results of two working piles and geotechnical data obtained from reports on site investigation. The two piles are representative of other tested piles.

Piling records tell us that pile toes have been embedded in at least 60cm of coral. Also the computation of the ultimate bearing capacity of the piles is based on the assumption that the capacity is generated by shaft and base resistances (friction) only. However, the load test results assure us of considerable contribution of the coral to the bearing capacity.

It is also important to note that the evaluation of bearing capacity has been done for single piles. Thus the safety factor of more than 2 is relative to a single pile. The construction of the port involves more than four hundred driven piles. The bearing capacity of the pile group will even be greater according to Vesic (1967) test results is which skin efficiences were reported to be high as three, whereas the point efficiences were all approximately unity. The increase in bearing capacity of a pile group is attributed by the driving of adjacent piles in a group which increases the relative density and the friction angle of the sand and the horizontal stresses. We are therefore in a position to conclude that the load bearing capacity of the piles is adequate with a minimum safety factor of 2.

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