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Prediction and Performance of Piles on Cemented Offshore Sediments - Two Case Studies

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ABSTRACT

The paper presents case studies for two projects. The first one is a major mining complex in the District of Sohar, Sultanate of Oman. The complex included a power station, transmission lines, roads, buildings and an offshore jetty. The jetty is the subject of the first case study. The subsoil conditions indicated a loose organic deposit overlying cemented sand and gravel interbeds interspersed with coral. The lower materials tended to be variable in density and difficult to evaluate for foundation requirements. It was recommended the jetty be supported on closed end pipe piles driven to refusal in the cemented sand and silt at about 20 metres below sea-bed level. The piles were designed conservatively to take into account the negative skin friction and the uncertainty of the capacity of cemented layers.

A pile testing program was carried out on 25 piles by using a Pile Driving Analyser. In addition, two full-scale pile load tests were carried out. The paper will compare the predicted capacities with actual ones measured in the field.

The second case study will cover the Bridgetown harbour extension. In this case, coral sands were encountered to a fairly deep depth. Steel piles were used to support the harbour component. The piles were designed, using conventional methods, to be founded in the cemented coral sands. During driving it was found that the driving destroyed the cementing effect and the piles penetrated much deeper than anticipated. A testing program was carried out where 10 piles were tested by the Pile Driving Analyser and by four full-scale load tests.

These two case studies indicate that it is very difficult to predict the capacity of piles in cemented sediments by conventional techniques. It is critical that, when founding in these materials, a comprehensive testing program be carried out.

INTRODUCTION

Cemented offshore sediments can present difficulties for foundation design which are not solved by conventional investigations and analyses. The most appropriate approach is to apply an observational and testing procedure to ensure that the foundations will perform satisfactorily.

This paper presents two case studies. The first one is a major mining complex in the District of Sohar, Sultanate of Oman. This complex included a power station, transmission lines, roads, buildings and an offshore jetty. The jetty is the subject of the first case study. The second one covers the Bridgetown harbour expansion in Barbados.

In both cases, piles were designed by conventional method and founded within the cemented deposits. A testing program was carried out on a number of piles at each site by means of a pile capacity analyser. In addition, two static load tests were carried out on each of the sites.

These papers present the project, the geotechnical investigation, subsurface conditions, foundation design, testing programs and lessons learnt from these two projects.

CASE STUDY NO. 1 - OFFSHORE JETTY IN SULTANATE OF OMAN

General

The Sultanate of Oman is located on the south-eastern portion of the Arabian peninsula. The Village of Sohar is on the north shore very close to the border of United Arabic Emirates. Large deposits of copper were rediscovered in the Sohar and surrounding regions. In 1980, the work on the major copper mining complex was started by the Oman Mining Company. The complex included a power station, transmission lines, roads, buildings and an offshore jetty. Although the writers were involved in the investigation for all phases of the complex, the jetty phase is most appropriate for this paper.

Geotechnical Investigation

The field investigation for the jetty consisted of 13 offshore borings and 3 on-shore borings. The drilling was carried out using a Pilcon Wayfarer unit using cable tools. A hydraulic powered pendant drill head was attached for core drilling purposes. Two of the thirteen boreholes were shell augered to approximately 10 metres below the sea-bed and then core drilled with an NX size double core barrel to 20 metres. Sampling was carried out by conventional split barrel sampler at 1.5 metres interval. Samples of the cemented strata were also recovered using NX size core barrel.

Subsoil Conditions

A general description of the site and the strata underlying the site is on plan and stratigraphical profile, Figure 1. The on-shore borings revealed 4 to 6 metres of loose recent beach deposits consisting of fine to medium sand with numerous shell fragments. This layer is underlain by 3 to 5 metres of loose organic shelly, silty fine sand which was also found to exist in the upper 3 to 5 metres of the sea borings. These deposits are underlain by dense, calcareous deposits of silt, sandy silt, sand and gravel to 13 to 14 metres depth (below sea-bed level). The calcareous deposits are generally weakly cemented with some hard interbeds of more strongly cemented materials ranging from silt to sand and gravel which appear conglomeritic. Below 13 to 16 metres, the subsoils are generally calcareous silts and sandy silts with occasional clayey silt zones. The deposits contain frequent cemented layers; the frequency of the cemented layers increased with depth. From approximately 21 to 30 metres, the strata was mostly continuously cemented.

be driven much further than the equivalent precast or closed-end pipe pile to develop similar capacities, hence were not considered.

Due to the presence of the cemented materials, driven piles could reach refusal at higher or lower levels than designed. The ability of a pile to penetrate the cemented layer is dependent on the pile type and pile driving system. A pile driving test to be carried out for each pile type and size was recommended to select the pile driving system to confirm both founding depth and capacity while maintaining minimum pile length.

For design purposes it was recommended that piles should be end-bearing in the cemented silt and sandy silt below approximately 20 metres below sea-bed level. This was considered more suitable than piles founded at lower depths because of increased capacity and less variable nature of the subsoils.

It was recommended that typically a 325 mm O.D. closed-end pipe pile driven to practical refusal in the cemented sand below -20 metres could

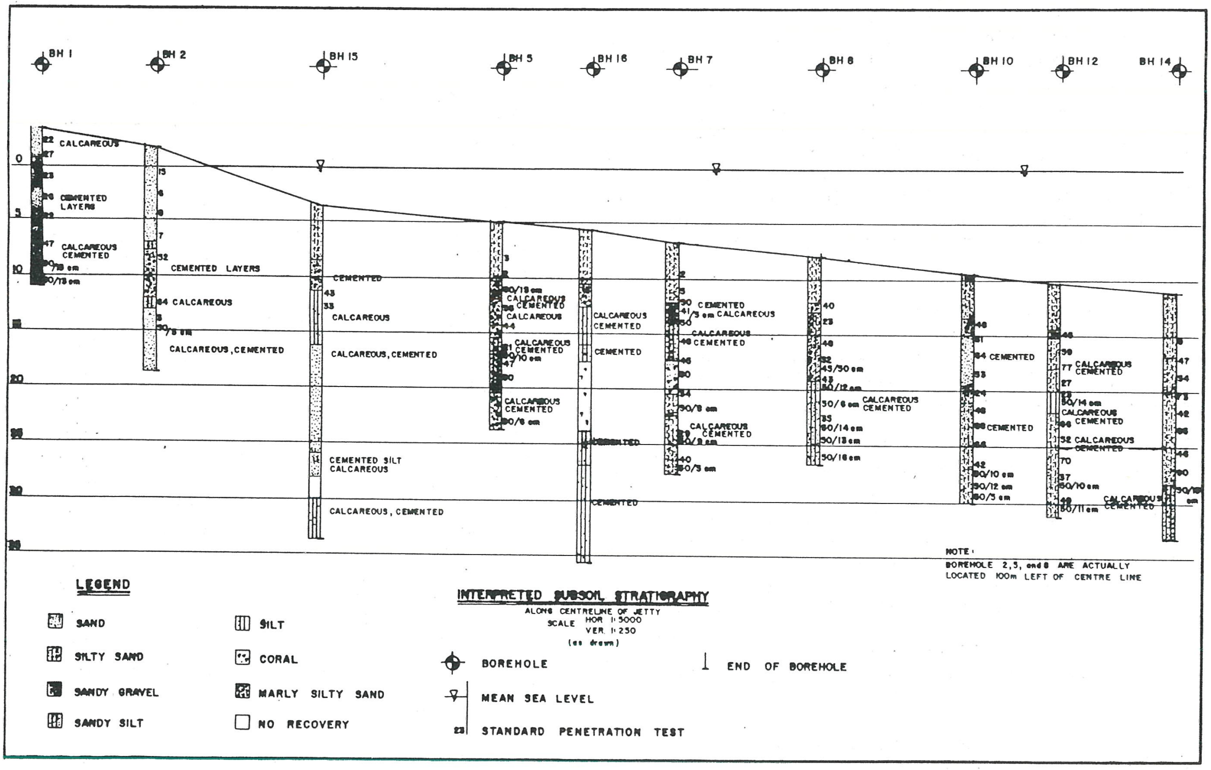

Figure 1

All samples were visually examined for the textural classifications. Typical tests are shown on Table I.

Foundation Considerations

Considering past experience and based on the geotechnical investigation, the most suitable support for the jetty was considered to be a piled foundation. Displacement type piles, i.e. driven precast concrete or concrete-filled closed-end pipe piles were considered practical for end-bearing support. Steel H-piles having a relatively small end-bearing area were expected to

develop a safe capacity of 50 tonnes (490 kN) after taking negative skin friction into account.

Due to the cemented and variable nature of these deposits, a testing program was recommended, which included two full-scale static load tests and monitoring a number of piles by means of a Pile Driving Analyser. The results of the testing program will be discussed later.

Pile Test Program

A pile test program was undertaken on-shore prior to the installation of off-shore production piles. The objective of this program was to determine the

Table 1

SELECTED GEOTECHNICAL PROPERTIES OF OFFSHORE DEPOSITS

Material	Test	Units	Range	Average	No. of Tests
Fine Sand with clay to cemented sand and with clayey zones	Liquid Limit (w_L)	(%)	32-50	36	25
	Plastic Limit (w_P)	(%)	19-24	21	25
	Plasticity Index (PI)	(%)	15-20	17	25
Sand to sand and gravel, calcareous cemented, with some clayey zones	Specific Gravity (SG)	-	2.71-2.82	2.74	46
	Unit Weights (γ)	(kg/m^3)	1800-2700	2160	43
	Modified Proctor Tests				
	Max dry density (γ_d)	(kg/m^3)	1850-2400	1980	32
	Opt. moisture content (w)	(%)	6-12	8	32
Sand to sand and gravel, calcareous cemented, with some clayey zones.	Shear Strength Parameters Cohesion (C)	(kg/m^3)	0	0	40
	Angle of Internal Friction (Ø)	(degrees)	29-37.4	32.3	40

bearing properties at design tip elevation by using the Pile Driving Analyser in conjunction with two conventional load tests.

The project, which had been designed at this stage, included the construction of a jetty trestle and jetty head which extends from the shore to approximately 1.5 kms off-shore. Approximately 604 pipe piles were to be driven at the site. The trestle piles were designed to support 99 tonnes in compression, while those at the jetty head, were designed for 120 tonnes. The majority of the trestle piles and some of the jetty head were designed to resist 35 tonnes in pull-out in addition to the compression loads. The higher design value used can be justified because of the heavier pile selected.

The purpose of the testing was:

a) to employ the Pile Driving Analyser to estimate the ultimate static bearing capacity of the piles; and to evaluate the driving system;

b) to correlate the Analyser results with load test data; and

c) to provide driving criteria to which the piles should be driven in order that they would adequately support the design loads.

A total of 25 piles were tested with the Pile Driving Analyser during the period from November 5 to December 12, 1981. After the piles were driven to approximately 9 m below the ground surface, the Analyser equipment was attached. This consisted of two re-usable strain gauges and two accelerometers securely bolted on diametrically opposite sides of the pile at 690 mm below the pile top. For each hammer blow, an electric signal from the instruments is fed into the Analyser, which converts them into force and velocity parameters to which the Case Method Wave Equation Analysis is automatically applied to determine the bearing capacity for that hammer blow. The maximum force (stress) at the pile top, the energy developed by the hammer-capblock-pile-soil system, the maximum velocity, and the dynamic resistances, are some of the parameters which may be output from the analyser. The force and velocity were continually displayed in the field on an oscilloscope and recorded on an F.M. Instrumentation Tape Recorder.

The piles were driven with a Delmag D30-13 open-end diesel hammer. The piles were 457 mm O.D. x 12.5 mm wall thickness. A 20 mm thick base plate was welded flush with the outside of the pipe, with reinforcing stiffeners projecting below the base plate.

The driving summary and analyser results are summarized in Table II. The piles were easily driven to approximately 9 m depth below which level the driving resistance gradually increased to between 5 and 14 blows per 25 mm at approximately 11 metres below the existing ground surface. A review of Table II indicates that for shallow piles (driven to a tip Elevation of about 7.5 metres below sea level) the estimated static bearing capacity ranged from 242 tonnes to 303 tonnes. Below the sandy gravel layer, the static capacity decreased to a low value of 210 tonnes at Elevation - 12 m approximately. The capacity then increased in the cemented sand and gravel layer to approximately 265 tonnes at Elevation -13.7 metres where driving was terminated.

Attempts were made to assess if there was an increase in the bearing capacity with time after driving (set-up). The results indicated that of the four piles tested, there was negligible increase in two piles, whereas, the other two showed an increase from 13 to 60 percent.

Two load tests were carried out in accordance with ASTM Standard D1143-73; i.e., to twice design load. In one test (pile 6N6) the load was recycled to three times the design load. In this case, the 470 mm x 12.5 mm wall thickness piles were tested for 240 tonnes respectively. The

Table II

RESULTS OF PILE DRIVING ANALYSER AND LOAD TESTS

Pile Designation	Pile Tip Elevation (m)	Driving Resistance Blows /25 mm	Energy Mean Metre-Tonnes	Static Bearing Capacity (Tonnes)	Capacity from Load Tests* (Tonnes)
8/S/16	-12.7	8	2.19	270	-
9/N/17	-7.6	7	2.69	257	-
9/S/18	-7.7	7	2.69	274	-
10/N/19	-8.6	10	2.35	272	-
10/N/19	-9.9	4	2.01	206	-
10/N/19	-13.6	19	2.39	251	-
2/N/2	-7.4	10	2.24	242	-
2/S/9	-7.5	8	2.45	292	-
3/N/3	-7.5	14	2.22	242	-
3/S/10	-7.4	7	3.05	297	-
4/N/4	-7.5	11	2.40	272	-
4/S/11	-7.5	9	2.78	302	-
5/N/5	-7.5	8	2.59	262	-
5/S/12	-7.5	8	2.64	265	-
6/N/6	-7.4	9	2.23	244	240
6/S/13	-7.4	7	2.88	274	-
7/N/7	-7.5	7	2.51	245	-
7/S/14	-7.6	7	2.54	264	-
8/N/15	-7.5	5	2.65	241	-
8/S/16	-7.5	6	2.72	252	-
9/N/7	-7.5	7	2.51	245	-
9/S/13	-7.4	7	2.88	274	240
9/S/4	-7.5	7	2.54	265	-
9/S/16	-7.5	6	2.72	252	-
10/S/20	-8.3	9	2.40	254	-
11/N/21	-8.3	7	2.66	306	-
11/N/21	-8.4	24	2.85	303	-
11/N/21	-13.7	32	2.04	265	-

* Notes load test only taken to twice design load

results of the load test of pile 6N6 are plotted on Figure 2. Unfortunately, the tests were not taken to ultimate failure. The results indicated that the cemented sand and gravel at relatively shallow depths had adequate bearing to support the design load. It was found that it was not necessary to drive the piles to the original design depth of 20 m below sea bed. The detailed monitoring program showed the cemented materials to be highly competent and the need to drive to depth unnecessary.

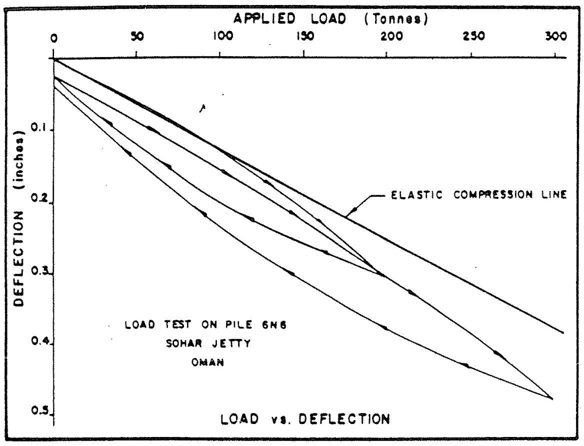

Figure 2

CASE STUDY NO. 2 - HARBOUR EXPANSION AT BRIDGETOWN, BARBADOS

General

The original deep water Harbour of Barbados was constructed at Bridgetown and completed in 1961. The Harbour Basin was formed by an artificial enclosure bounded on the east and south by reclaimed land and on the west by a breakwater. The principal marine structures within the Harbour are the Quay Wall, the Cross Berth and the Breakwater. The Harbour expansion involved an extension to an existing gravity quay wall, a shallow draft wharf, a bulk handling wharf and repairs to existing pilaster - supported wharf, together with two transit sheds and infrastructure facilities as well as navigation dredging and land reclamation.

Subsoil Conditions

Geotechnical investigation revealed that the predominant sub-surface material at the site area was coral and coral sand of fairly recent geological age. This material was fairly loose at its upper levels and becomes compact to very dense with depth. (see Figure 3) In the area of the quay wall extension, very dense coral sand was encountered between 15 and 25 metres below low water level. Most of the coral sand above these levels was loose. Closer into the shore very dense coral sand was encountered at approximately 6 metres below low water level and refusal to dynamic cone tests was encountered at 15.1 metres below low water level. Elsewhere in this vicinity refusal to dynamic cone tests was encountered at depths of up to 25 metres below low water level.

Foundation Considerations

The geotechnical data was analysed primarily on the basis of previous experience in the area. The reason for this was a lack of detailed information on the bearing capacity of coral formations and also because the investigation had revealed sharp changes in relative density both in the vertical and horizontal directions. Prestressed concrete piles, which are fairly common in Barbados, and have the advantages of being corrosion resistant and have a relatively high local versus foreign exchange component, were selected for all the marine structures.

Local experience with pile foundations in the same coral formations indicated that 400 mm square prestressed concrete piles could support design loads of 50 to 75 tons per pile. Based on this information, it was concluded that at the quay wall extension similar piles could be founded in the very dense coral encountered below -15 metres and -25 metres at similar bearing loads. It was subsequently decided to employ an assumed bearing load of 670 kN (75 tons) on piles driving to -18 metres. At other locations, i.e., at the shallow draft facility and bulk handling facilities, it was anticipated that similar loads would be achieved at -16 metres. To confirm this pile tests were carried out.

Pile Test Program

The initial program called for the contractor to drive 400 mm square piles in 25 mm thick steel plate at the tip. The piles were driven with as heavy a hammer as practical (Vulcan 101 single acting hammer). The cushion in the cap block consisted of 100 mm of pine wood. According to conventional methods, 3 blows per 25 mm would be sufficient resistance for a working load of 75 tons.

The first pile at the quay wall extension was designated at a test pile to be subjected to both bearing and pull-out tests. This pile drove at resistances of 3 to 6 blows/300 mm until driving

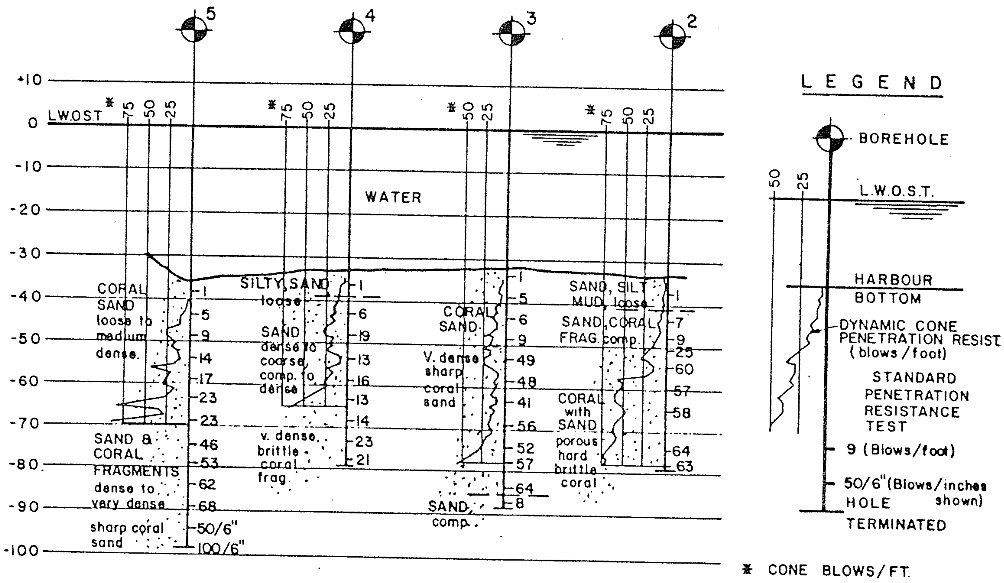

Figure 3 - Geotechnical Profile

was terminated at Elevation -30 metres. It also exhibit "soil freeze" for the first metre of redrive after splicing. The additional length was obviously uneconomical. The contractor was able to drive another test pile to -40 metres.

The tentative first conclusions drawn from the initial pile driving were:

1. Cemented coral formations in the vicinity of the test piles did not provide the type of support which might have been expected from the geotechnical information, or, from the experience with other piling in the area.

2. Piles were possibly "freezing", developing skin friction.

3. Conventional dynamic formulae were possibly not providing reasonable estimates of the bearing capacities of the piles.

To verify the second conclusion, three of the piles were redriven after 8 days. A gain in resistance was noticed. Since this was not conclusive it was decided to use the Pile Driving Analyser.

The first pile testing program included driving 3 piles at the quay wall extension with the Pile Driving Analyser attached. A static load test was also performed on one of the piles (E15). Results of the testing are shown on Figure 3 and Table II.

A second, more comprehensive program was carried out using a PDA while driving 18 piles in the quay wall extension. Some were driven with a lighter Vulcan 060 hammer and others with the Vulcan 010 used previously. Piles were first driven to an average Elevation -19 metres. Except for one pile (913a), which was selected for load testing, all other piles were redriven to -21 metres. Results are again shown on Table III and Figure 4.

Figure 4 also shows the results of a load test on pile (D27) where the base of the pile was increased by welding a 560 mm by 560 mm steel plate and driving by Vulcan 060 to -18 metres.

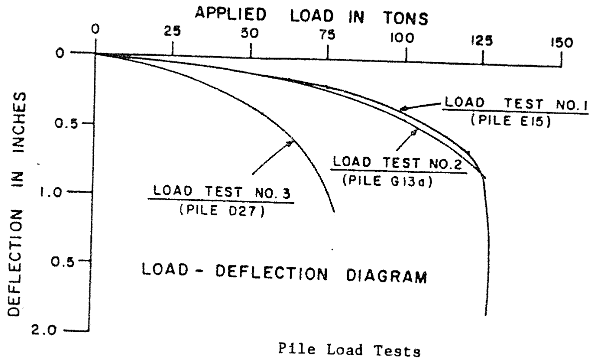

Figure 4

The results show that the pile failed at much lower value of 75 tons in compression.

Conclusions

1. Based on limited existing data for piles driven into cemented sediments, it appears that conventional means of predicting bearing capacities are not realistic.

2. In the Barbados site capacities computed by conventional mean resulted in an over estimation of capacities and much deeper piles would have to be used to obtain the desired predicted capacity.

3. During driving the cement structure was broken resulting in piles penetrating to deeper depths.

4. The piles exhibited "soil freeze" with time.

5. In the Oman site conservative values were established for design because of the above noted problems and experience. A detailed testing incorporating both load tests and Pile Driving Analyser, indicated that this conservative approach was not warranted. In

Table III

RESULTS - FIRST AND SECOND PILE TEST PROGRAMME

	INITIAL DRIVING		RESTRIKE		
Pile	Estimated(1) Bearing Capacity (Tons)	Driving(2) Resistance (Blows/Ft)	Estimated(1) Bearing Capacity (Tons)	Driving(2) Resistance (Blows/Ft)	Ultimate Bearing Capacity from Load Test (Tons)
J14	100	5	148	80	-
J13	146	10	200	60	-
H15	64	4	250	20	-
H14	24	4	200	52	-
H13	34	14	155	24	-
G13a	50	5	237	80	112.5
F15	60	2	240	53	-
F14	69	6	170	30	-
F13	57	2	130	60	-
E15	130	4	118	9	112.5
E14	105	3	70	18	-
E13	133	9	130	60	-
D14	0	6	90	20	-
D13	0	2	145	20	-
C14	25	6	100	25	-
C13	137	7	100	12	-
B14a	131	3	47	15	-
B13a	110	5	90	10	-
A14	158	6	80	27	-
A13	140	5	68	18	-

(1)Ultimate Bearing Capacity estimated from Pile Driving Analyzer results.
(2)Refer to Fig. 4 for hammer used.

this case higher capacities were available at relatively shallow depths.

6. Because of the limited historical data available, it is essential that where piles are founded on cemented sediments, that a detailed field evaluation precede the final design.

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