JKAU: Eng. Sci. Vol. 2, pp. 3-17 (1410 A.H. / 1990 A.D.)

A Comparative Study of Pile Foundations in Coral Formations and Calcareous Sediments

FOUAD M. GHAZALI, ZAKI A. BAGHDADI AND OSAMA A. MANSUR
Civil Engineering Department, Faculty of Engineering, King Abdulaziz University, Jeddah; Military Works – Jeddah Branch, Saudi Arabia.

ABSTRACT Great difficulties are usually encountered in the design of pile foundations in marine strata. These difficulties have been attributed mainly to the heterogenous nature and unusual behavior of these strata. Quite commonly, the marine strata have low bulk density and usually consist of weakly cemented calcareous and carbonate soils interbedded with coral rock containing cavities filled with coral debris. Geotechnical investigations show that conventional soil mechanics techniques are not successfully applicable to these formations and sediments.

This paper presents a case study of a proposed foundation design for a 380 m long quay. This quay is to be constructed near the city of Jeddah on the east coast of the Red Sea. The initial proposed design by the designer suggested 60 cm square precast concrete driven piles. This proposal was not satisfactory to the geotechnical consultant of the contractor, and instead he recommended bored and grouted piles. This recommendation was based upon field data, load testing of piles and information available in the literature. A critical evaluation of both, the proposed design as well as the alternative put forward by the contractor, is given in this paper along with recommendations envisaged by the authors.

Introduction

The basic formations of the eastern coast of Red Sea, between latitudes 30 degrees south and 30 degrees north, are calcareous deposits and cora reef rocks. Calcareous deposits are mainly composed of calcium carbonate ($CaCO_3$) which are formed by shells and skeletal remains of benthos organisms such as corals, molluscas, and calcareous algae. Coral reef rocks are formations of calcium carbonate laid down by living marine plants (coralline algae) and marine animals, corals[1]. With time, by precipitation and recrystallization, corals became more dense and rock-like and are then known as coral reef rock. Detailed mineralogical analysis by *x*-ray diffraction showed that, in recent samples of coral reef rocks, $CaCO_3$ existed entirely in the form of aragonite, whereas older samples generally contained some calcite; the greater the age, the higher was the percentage of calcite[2].

Due to unusual behaviour of coral and calcareous deposits and lack of satisfactory engineering techniques and design theories regarding such formations, designers of pile foundations in this type of strata face great difficulties and problems. These problems increase when such strata have a heterogenous nature, as in the case along the eastern coast of Red Sea.

A comparative and critical evaluation of two design methods of pile foundation in heterogeneous corals and calcareous deposits is presented in this paper, with an attempt for better understanding of the behaviour of pile foundations in calcareous and coral formations. A case history about the design of pile foundations in calcareous and coral formations and pile load tests in Jeddah area on the eastern coast of the Red Sea are presented. This pile design is for a ship repair quay 380 meters long and 14 meters wide, with a crane on top and along it, which will transport a maximum load of 60 tons. Two parties are involved in this case history in which party D (Designer) proposed 400 driven precast concrete piles while part C (Contractor) proposed an alternative method on the basis of his own analysis of site investigation and the results of the pile load tests. The views of the two parties are presented in this paper along with the authors' comments, critical analysis and recommendations of this case history.

Geological and Engineering Information

A brief description of the geological aspects of the eastern coast of Red Sea in general, and the site of the project in particular, is presented.

The eastern Red Sea coast is characterized by three main features. First are the numerous coral reefs, fringings and barriers, that stretch parallel to the coast. These formations contain cavities filled by coral debris eroded by wave action. Second feature is the sea floor between the coral heads and reefs which is covered with marine sediments. The marine sediments consist of loose to medium dense carbonate sands, silts and clays with layers of coral. The third feature is the coastal plain which contains alluvial soil from the Arabian shield (gravel, sand and silts) underlying and intermixing with the reef deposits and marine sediments.

The geology of the project area conforms to the general features outlined earlier. Two lithologic units are present in the site area. An alluvial fan consisting of mixtures of sand, clay and silt with debris of shells and corals. Underlying the alluvia, exists a coral reef formation consisting of a surficial coral reef and an older buried coral reef. Disintegration products of the reefs fill the cavities in the reefs and cover the slopes with blankets of skeletal sand and silts.

It should be pointed out that geological observation could provide essential understanding of the engineering behaviour of the soils under study. Deshmukh *et al.*[1] list some points to be observed regarding corals:

1) Origin of skeletal system: this concerns the shape and size of corallites examined and whether their walls are perforate or imperforate.

2) Age of the coral reef: older coral reefs exhibit greater lithification, higher

strength, less void ratio and less permeability.

3) Type of reef: oceanic reefs show higher strength and lower permeability than continental reefs.

4) Zonation: tests indicate that corals on exposed sides (windward) are likely to have better geotechnical properties than lagoonal or inexposed zones.

All available geotechnical data on marine formations have emphasized their heterogeneous and irregular stratification. Alluvial, calcareous and carbonate soils along with coral limestones are usually found in coastal zones. This dictates that subsurface borings should be spaced as closely as possible.

Calcareous sediments have been reported[1,3] to exhibit weak cementation, high void ratios and high moisture contents with low bulk densities. Survey of literature on marine structures has indicated that pile foundations are probably the most suitable for this kind of projects[4].

It has been found[1,3] that bearing capacities of driven piles in calcareous and coral formations calculated from conventional geotechnical procedures are significantly higher than the measured values leading to unconservative designs. Examinations of pile load tests have led to recommend limiting values of skin friction of 2t/m^2 and point bearing of 200 to 600 t/m^2[3-7]. It should also pointed out that bored and grouted piles offered 3-5 times higher bearing capacity than driven piles of the same diameter[1]. Furthermore, pile driving causes collapse of the weak cementation and destruction of coral structures leading to lower lateral pressure and point bearing[8]. Increasing penetration length or enlarging the end base of the driven pile may not appreciably improve the bearing capacity.

Standard penetration tests have been utilized to develop a pile design procedure with limiting values of friction and bearing on the basis of investigations executed along the Red Sea coast of Saudi Arabia[4,5] as shown in Fig. 1 and 2 in which good estimation of pile bearing capacity was obtained by pile load tests.

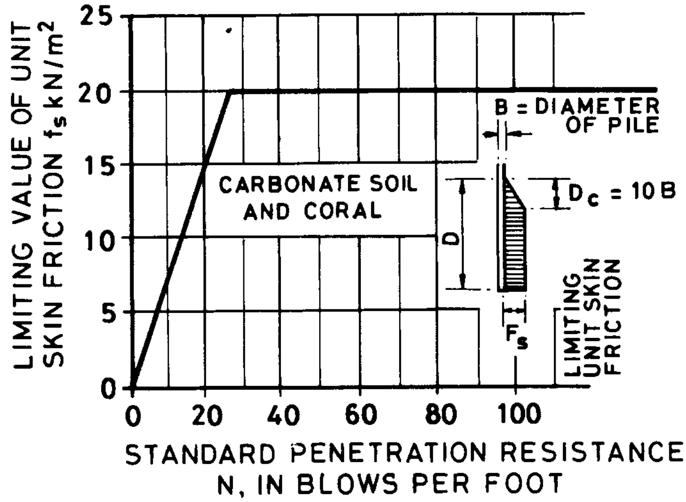

FIG. 1. Limiting unit skin friction against *N* values. Hagenaar (5).

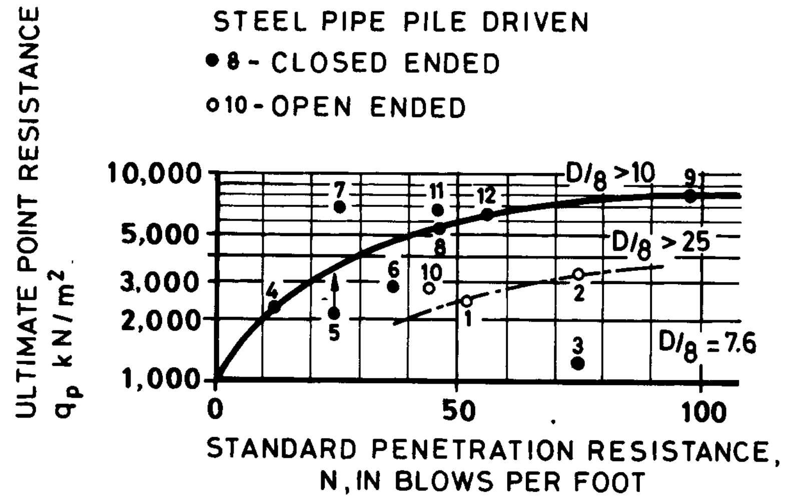

FIG. 2. Ultimate point resistance versus N values. Hagenaar (5).

At the site under consideration, party "D" conducted subsurface and geotechnical investigation that included:

1) Drilling three offshore and two onshore borings to depths rangings from 25-40m, running SPT and collecting samples.

2) Jetting 19 jet borings to determine the depth of soft sediments.

3) Performing static cone penetration tests up to 23m depth.

4) Conducting laboratory tests including soil classification on samples obtained from boreholes, direct shear box, unconfined compression tests and chemical analysis of the soils.

Examination of the bore logs indicated that the project area consists of two zones: northern and southern. The northern zone, Fig. 3, is less favourable, since no hard stratum is encountered. The southern zone, Fig. 4, is more favourable to the pile design since a hard stratum of coral limestone is encountered.

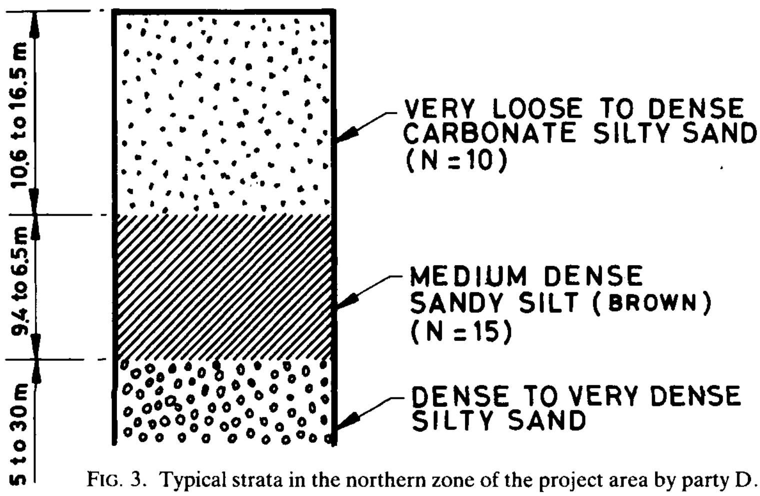

FIG. 3. Typical strata in the northern zone of the project area by party D.

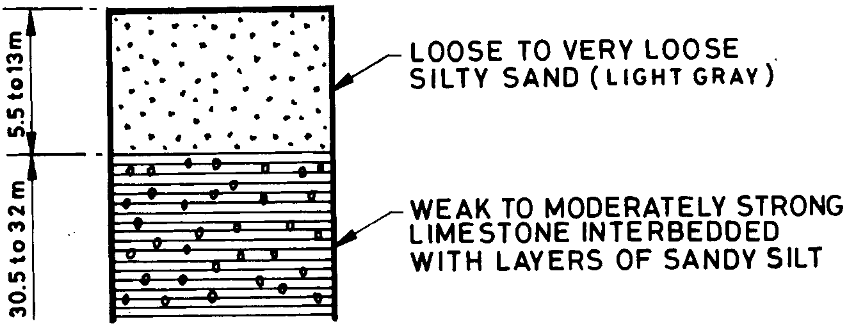

FIG. 4. Typical strata in the southern zone of the project area by party D.

Party C, conducted their own geotechnical investigation that included deep borings and laboratory tests on selected samples. On the basis of their investigations the following conclusions were put forward (Fig. 5).

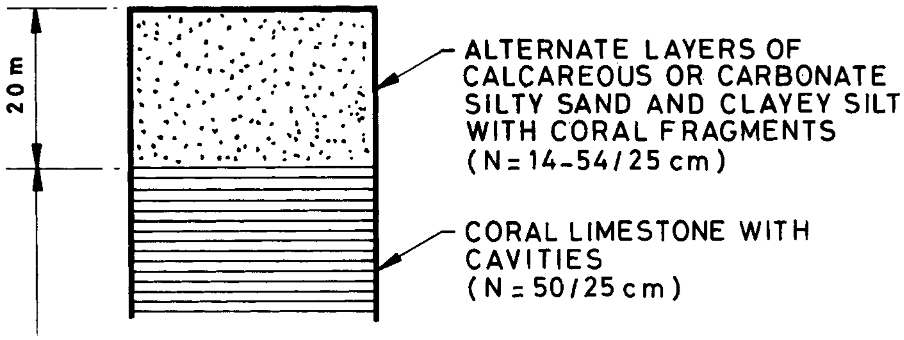

FIG. 5. Typical strata in the project area by party C.

1) The first 20 meters soft alluvial soils are encountered overlying the coral limestone.

2) Below 20 meters, penetration resistance generally increases but at greater depths it decreases from over 80 blows per 25cm down to 40 blows per 25cm thereafter. This means greater penetrations do not necessarily mean higher bearing capacity.

Selection for Pile Types for Coral and Calcareous Formations

In selecting piling for the formations explained earlier, different factors and parameters should be considered in order to choose a proper pile foundation type for a proposed structure. Among the most important factors and parameters to be considered are:

1) Subsurface conditions encountered at the project site as heterogeneous or non-heterogeneous nature, presence of cavities, their size and condition in the coral rocks and the depths and thickness of the rock layers.

2) Engineering properties and geological aspects of coral and calcareous sediments; these properties vary significantly from those of quartz formations and sediments as mentioned earlier.

3) Technique of installation of piles.

4) Ability to withstand driving in hard strata in case of driven piles.

5) Precautions against corrosive marine environments.

6) Satisfactory pile length, shape, size and embedment length.

7) Size of the structure, site and maximum number of piles permitted to be installed at site.

8) Characteristics of pile material.

9) Construction considerations for pile installation procedures.

10) Safety and economy.

According to literature review of some case records, the most suitable and economical pile types recommended for coral and carbonate formations are as follows:

1) Open-end tubular (pipe) steel piles with 30mm wall thickness by 1000mm long driving shoe fitted to the leading end. The piles are usually treated against corrosion from sea environment by cathodic protection and coating[1,4,5].

2) Driven precast prestressed concrete with an octagonal shape and a hollow core, and in the case of heterogeneous nature of subsurface conditions, special jointing systems and splice details are needed for the proposed precast piles[3,6].

3) Bored and grouted piles[2,9].

Proposed Design Method by Party "D"

The quay wall is about 380m long and is envisaged to be founded on driven precast concrete piles. The pile sizes and loads were given as:

Pile size (cm)	Vertical load (t)	
	Compression	Tension
60 × 60	230	230

Borehole information indicated that soil conditions in the southern zone permit driving the piles to a hard stratum. Piles driven to Elev. –22 (24m depth) should provide satisfactory foundation in this zone. As for the nothern zone of the project is concerned, the hard stratum has not been encountered, although a dense sand layer is encountered at Elev. –18 (20m depth). The piles in this zone should be driven down to Elev. –24 (26m depth). The platform behind the quay wall will be made from *hydraulic fill behind* a selected fill bund. The grading of the selected fill will be carefully chosen to ensure satisfactory engineering properties and permit driving of the piles through it without difficulty.

Two methods were employed to estimate pile bearing capacities: on the basis of SPT data and CPT data[10]. In the southern zone, where a limestone layer of at least 18m thick is present at Elev. –22 (16m depth), the blow count (N) was estimated to be 70 blows per 30cm. Thus, the ultimate point resistance (q_{pu}) might be taken as:

$$q_{pu} = 40N \ \text{t/m}^2$$

and thus the allowable bearing resistance (q_{pa}) using a factor of safety of 3 will be:

$$q_{pa} = \frac{40 \times 70 \times 0.36}{3} = 336 \text{ tons}$$

In the northern zone, no hard stratum was encountered but there exists a layer of dense sand with blow counts of over 50 or Dutch cone penetration resistance of 2800 t/m² between the depths of 20m and 30m from the surface. The pile should be embedded within this stratum and its allowable bearing capacity is calculated as follows:

1) Frictional resistance, $q_{su} = \frac{N}{5}$ t/m^2

Depth, m	Blow Count, N	Estimated Skin Friction, q, tons
13-16	15	$\frac{1}{5} \times 15 \times 4 \times 0.6 \times 3 = 22$
16-20	20	$\frac{1}{5} \times 20 \times 4 \times 0.6 \times 4 = 38$
20-26	40	$\frac{1}{5} \times 40 \times 4 \times 0.6 \times 6 = 115$
		Total q_{su} ,tons = 175

2) Point bearing resistance, at a depth of 26m, ($N = 40$) q_{pa} (allowable point bearing) $= \frac{40 \times 40 \times .36}{3} = 192$ tons

Thus the pile should have a total capacity of 280 tons, which is greater than the required 230 tons.

Party "D" also presented calculations of the bearing capacity by method proposed on the basis of Dutch cone tests. The results of these calculations indicated that the required 230 tons capacity is covered by pile resistance, 96 tons frictional and 134 tons end bearing.

The tension loads on the piles and 60cm² would be 30 tons, which would require an ultimate soil friction of 0.5 t/m² on the shaft to prevent a pull out. As the average blow count along the pile was at least 10, it was not anticipated that the tensile loads expected would lead to any significant pile heave.

The contractor was requested to perform pile load tests to establish the final length of piles required and the best pile driving criteria in relation to his equipment.

Settlement of the $60cm^2$ driven piles was estimated using a method proposed by Poulos. It was anticipated that pile length would be 27m. Calculations gave a settlement of 1.5cm using an ultimate load equals to maximum compression load $\times$ 2.2. Party D, thus, estimated a maximum settlement of 2cm.

Views of Party "C" and Load Tests for the Proposed Pile Design

Party "C" conducted an independent site investigation through a number of borings, laboratory tests on selected soil samples, controlled drilling using the ENPASOL recording equipment, test driving of three piles (combined with dynamic load testing using the DPLT equipment) and static loading tests of two piles. Solid precast reinforced concrete pile with 65 $\times$ 65cm section was used in performing the driving and load tests. A 15400kg hammer was used in driving the concrete piles falling from a 2.78m height.

Figure 6 shows the driving records of three pile tests performed. Piles 1 and 2 were driven in the north part of the quay site (at a distance of about 15 meters from each other), and pile 3 was driven in the south part of the quay site (at a distance of about 350 meters from the other two). The soil profile next to the driving record of pile 1 is correlated with it, since its hole was 1 meter away from pile 1, while the soil profile near the driving record of pile no. 3 is not correlated with it since the bore hole was 25 meters away from pile no. 3. This proves the heterogeneous nature of the strata.

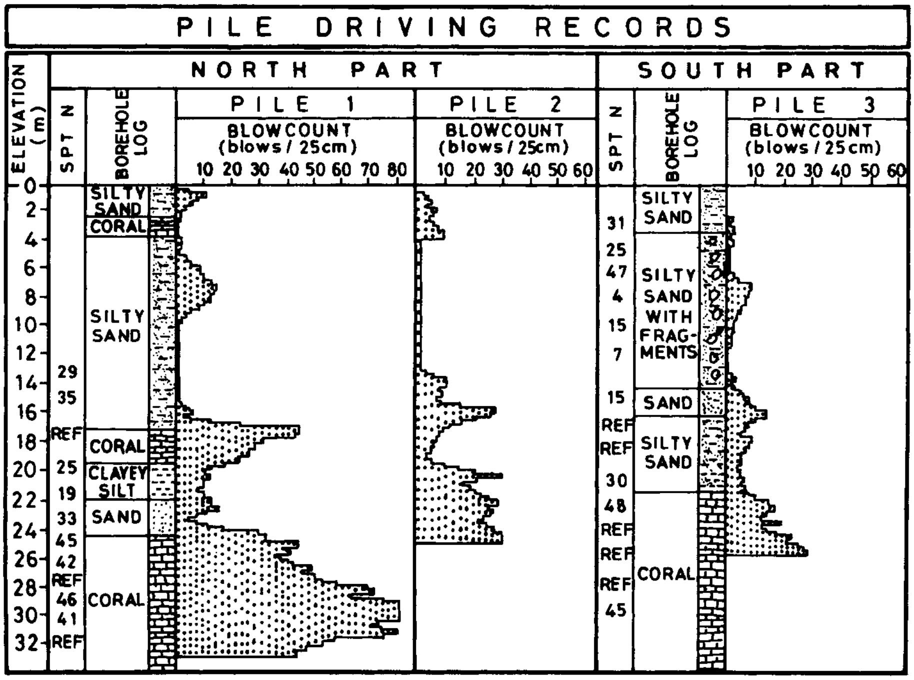

FIG. 6. Pile driving records by party C.

Figure 7 shows party "C" wave equation analysis from which the prediction of the ultimate static resistance can be obtained for various blow-counts. Figure 8 shows the load test results for piles number 2 and 3.

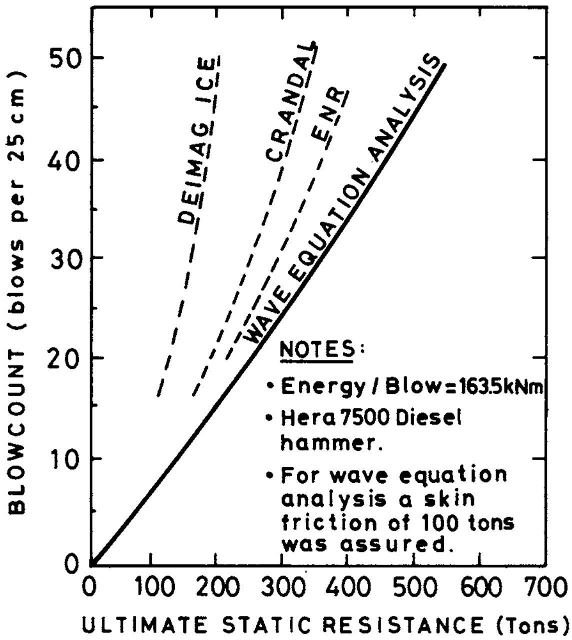

FIG. 7. Wave equation analysis results by party C.

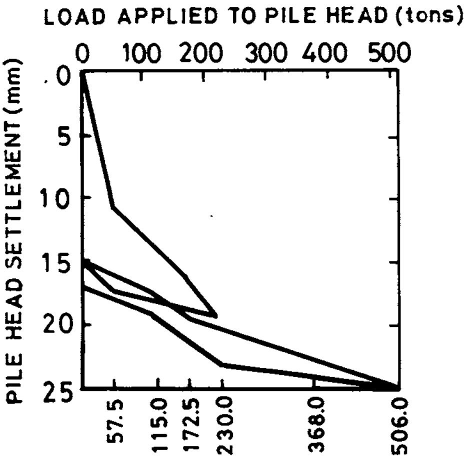

FIG. 8a. Load-settlement results test pile 2 (driven)

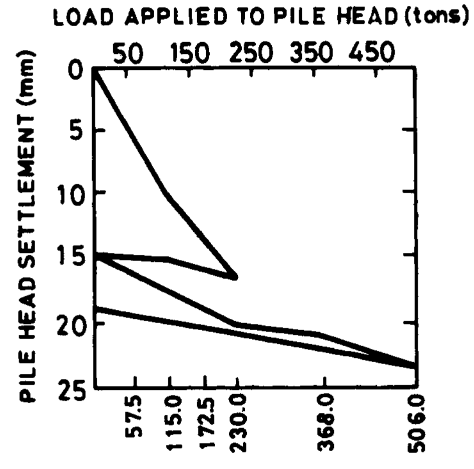

FIG. 8b. Load-settlement results test pile 3 (driven).

According to the site investigation, pile driving records and load tests, party "C" has performed its own analysis and drawn conclusions, and made recommendations therefrom. The following is a summary of this analysis, views and recommendations regarding the proposed pile design.

1) Figure 6 shows that along the first 20 meters, penetration resistance is very low due to soft alluvia through that depth. Below 20 meters, penetration resistance increases. Pile 1 gave 80 blows per 25cm penetration resistance at deeper coral layer (30 meter deep), but then again reduced to 40 blows per 25cm; piles 2 and 3 did not exceed 30 blows per 25cm along the full specified penetration length. Thus, by going to larger penetrations there is no guarantee that the bearing capacity will increase.

2) Figure 7 predicts higher bearing capacities. For a blow-count of 40, it predicts 460 tons, and for a blow-count of 50 it predicts 550 tons. The driving records in Fig. 6 have a large variability (range between blow-counts of 25 to 80 in the coral) which means that the predicted bearing capacities vary accordingly, and the proposed allowable pile design load 230 tons (ultimate 460 tons, safety factor of 2) will not be achieved, since the required blow-count should, consistently, be over 40 blows per 25cm.

3) Party "C" considered the conventional investigation and analysis used by party "D" which usually gives unconservative design values in coral and carbonate sediments and the proposed design values by party "D" of driven piles are over-estimated. Hence, party "C" recommends using skin friction of 2 tons/m^2 and tip resistance of 400 tons/m^2 as it is suggested in the literature for a similar pile installation and site conditions in coral and carbonate sediments. Using these values and a 65cm^2 concrete pile penetrating 24m into the soft alluvia and coral formation, and using a safety factor of two, an allowable design load of 147 tons/pile in compression was found. This load is significantly lower than the required design load of 230 tons pile.

4) Figure 8 shows load-settlement curves in which the proposed ultimate bearing capacity was not reached within the maximum allowed settlement. Party "C" considered these results of the pile load tests as unsatisfactory to be used in the pile design.

5) Party "C" recommended vertical large diameter bored and grouted piles as an alternative to the proposed pile design by party "D" because of low bearing capacities of driven piles in coral formations. These piles will penetrate the deep coral limestone for about seven meters and will be grouted along this length. Party "C" thinks that tip resistance should not be greatly emphasized, due to the highly heterogeneous nature of the soil formation in the site and the possibility of cavities in the disintegrated coral limestone layers. These cavities were estimated to vary in size from a few centimeters to several meters and filled with silty sand, shells and coral debris. Party "C" assumed an allowable, skin friction of 15 tons/m^2 along the grouted portion of the pile and an allowable tip resistance of 100 tons/m^2.

Comparative and Critical Evaluation

The 336 and 280 tons/pile design capacities of the driven precast concrete pile proposed by party "D" are considered to be liberal values due to the following analysis and findings:

1) The soil formations and strata at the site of the quay consist of unusual soil conditions with a heterogeneous nature. These soils are mainly made up of carbonate materials in the sand-silt range. The site conditions become weaker and poorer due to the presence of loose silty layers, coral fragments, shells, cavities within coral layers and absence of good silica sand. The soil investigation of the site performed is based upon five borings only for a rather heterogeneous subsurface condition. More boreholes could have been done to present a better a more accurate picture of the site and to minimize the possibilities of unforseen variations in sub-surface conditions which are expected between borehole locations.

2) Bearing capacity equations such as Meyerhof's and Poulos' settlement equations were used by party "D". These equations are usually used for determining bearing capacities of silica sand and quartz formations, while in carbonate soils the use of these equations leads to overestimation of the pile capacity. Conventional methods of design assume that a driven pile in quartz formation (hard particles that do not crush but displace during pile installation[1] will pack the soil tightly around it and that will build large soil-pile interface stresses, which consequently give high skin friction. In calcareous sands and carbonate deposits, however, the soil is naturally very loose and pile driving vibrations are not very effective in densifying this type of soil which is composed of many flat particles and some bulky hollow particles. A driven pile causes the soil grains to crush rather than displace literally[8]. Furthermore, soil weak cementation prevents lateral pressures from developing against the pile surface[12]. Consequently, the soil-pile interface stresses will be small, resulting in low-skin friction. A driven pile also causes a breakage in the structure and cementation of the coral rock[6], which results in low skin friction. It seems that these facts were not taken into account by party "D".

3) The calculated end bearing capacities of the piles driven in the north and south

part of the site were 192 and 336 tons, respectively. These two values were calculated with heavy emphasis on the end bearing, while soil investigations in these two regions indicated that at the required depths, there existed either medium dense to dense carbonate silts and sands with coral fragments or coral rocks with cavities.

The driving records in Fig. 6 show that approximately along the first 20 meters, penetration resistance is very low due to the existence of soft alluvia. This leads to the suggestions of soil improvement for the soft alluvia or its replacement with a selected silica fill. This should increase the lateral support of the upper soil layers of 20 meters and consequently the skin friction of the pile. These possibilities were not taken into account in the design by both parties.

The load-settlement curves, Fig. 8 do not follow the typical pattern usually found from pile load tests. They also show that the ultimate bearing capacity of the tested pile has not been reached within the permissible total deformation of 20mm as calculated and given in the specifications. The unusual shape of load settlement curves at low load levels cannot be attributed to the unique nature of coral formations. Probably due to the crushing of particles and breakage of cementitious bonds between them. The loading was probably conducted almost immediately after driving. Typical curve shapes have been found from pile load tests in corals[3,11].

It is also observed from Fig. 6 that the two load tests of piles no. 2 and 3 may not give their maximum capacity due to the following reasons:

1. Both piles have been driven through a moderately weak coral layer (blow count less than 30), while it might be expected that this count will increase at deeper depths.
2. The period between driving and test loading is short since piles driven in calcareous soils show very low skin friction during and shortly after driving. After driving, the skin friction increases with time due to soil freeze phenomenon around the pile[3]. In order to allow the soil freeze to take place, the pile load tests should start after a reasonable period has elapsed.

Based on the results presented and if the specifications requirements are unconditional, it could be concluded that either the design method should be improved or a change of pile type may be unavoidable.

The proposed pile design method by party "C" was bored and grouted piles. Party "C" was very conservative in using a low tip resistance of the pile in the total net bearing capacity. Also, the skin friction component of the bearing capacity used by party "C" was a liberal value.

According to the comparative evaluation of the two proposed pile types, the authors suggested that full scale bored and grouted pile load tests should be performed by party C.

Hence, three bored and grouted piles were tested under a vertical load of 506 tons[13]. This load is equivalent to 2.2 times the design load which is the ultimate bearing capacity as started in the specification and the design of the foundation of the ship repair quay. To limit the cost of testing, it was decided to limit the loading of this value rather than increasing it to the failure condition. Bored and grouted piles are rather expensive. The construction of such type of piles exclusively for pile load testing, without using them in the main foundation, does not seem to be a wise economical choice. In addition, the locations of these piles coincide with those of the piles to be used later as a part of the proposed foundation structure. Hence, continuous loading of these piles till failure was not considered as the main objective of the pile load test. Instead, settlements of piles were monitored so that these should not exceed 20mm under the ultimate bearing capacity as stated in the specifications of the project.

The results of the tests of the three tested piles showed settlements of 1.6mm, 1.6mm and 3.6mm, respectively, as shown in Fig. 9. This clearly indicated that the tested bored and grouted piles satisfied the specification requirements of the foundation.

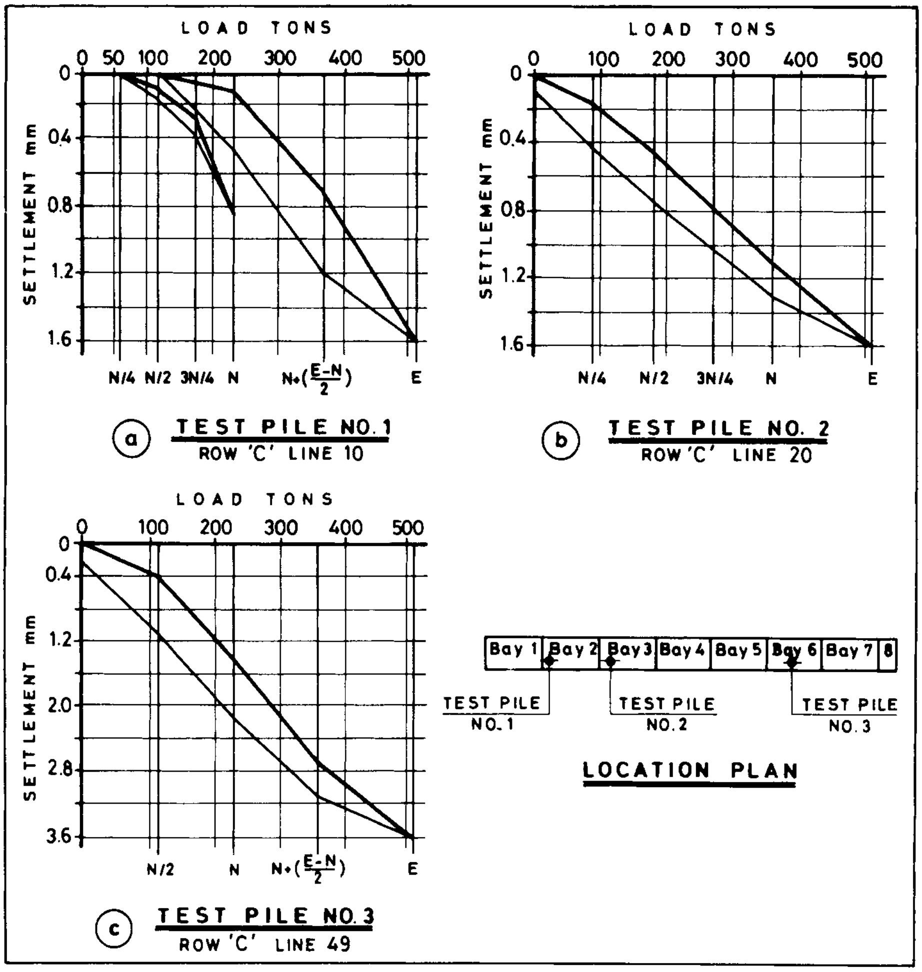

FIG. 9. Results of pile load tests (bored and grouted).

Conclusion

The importance of understanding the geotechnical problems encountered in the design of foundations in marine sediments are highlighted in this paper by presenting an actual case study. This case study presents two design approaches: precast concrete driven piles, and bored and grouted piles. According to the pile load tests of these two pile types proposed, bored and grouted cast in place concrete piles were found to be the most suitable type for coral formation and carbonate sediments of the east coast of the Red Sea. This type of pile foundation is capable of carrying safely the allowable vertical load of the proposed ship/repair quay with minimal settlement values; while in case of driven precast concrete piles, the observed allowable settlements were high and exceeded the values stated in the specification even before reaching the working load.

Acknowledgement

The authors are grateful to the General Directorate of Military Works, Ministry of Defence and Aviation, Saudi Arabia for their assistance and permission to publish this paper.

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دراسة مقارنة على أساسات الخوازيق في المكوَّنات المرجانية والرواسب الكلسية

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وفرع الأشغال العسكرية بجـدة ، المديرية العامة للأشغال العسكرية ، وزارة الدفاع والطيران ،
جـدة ، المملكة العربية السعودية

تقابل عادة مصممي أساسات الخوازيق في الطبقات البحرية مشاكل وصعاب جمة . وتُعزى تلك الصعاب بشكل أساسي إلى الطبيعة غير المتجانسة والسلوك غير العادي لتلك الطبقات . ومن المتعارف عليه أن تلك الطبقات البحرية ذات قيم كثافة صغيرة ، ومكوناتها مواد كربونية وكلسية متماسكة ، تتخللها صخور مرجانية تحتوى على تجويفات وفراغات ممتلئة بحُتات الصخور المرجانية . وقد أظهرت الاستقصاءات والبحوث في مجال التقنية الأرضية أن تكنيك ميكانيكا التربة التقليدي لا يُمكن استخدامه بنجاح لهذه المكونات والرواسب .

وتُقدم هذه الورقة دراسة حالة خاصة لمقترح تصميمي لأساسات رصيف بحري بطول ٣٨٠ متراً سيُنشأ بالقرب من مدينة جدة على الساحل الشرقي للبحر الأحمر . وقد اقترح المصمم الأساسي للمشروع استخدام خوازيق خرسانية مسبقة الصنع ، مدفوعة في التربة ، وبمقطع مربع طول ضلعه ٦٠ سم . ولم يجز هذا التصميم على موافقة الاستشاري الخاص بالتقنية الأرضية لمقاول المشروع ، وبدلاً عن ذلك أوصى بتبني خوازيق خرسانية مصبوبة بالموقع مع الحقن . وقد استند استشاري المقاول عند وضعه لهذه التوصية على النتائج والمعلومات الحقلية وعلى تجربة تحميل الخازوق الحقلي والموضوع في التربة ، إضافة إلى المعلومات المتوافرة عن أدبيات الموضوع . كما تحوى هذه الورقة تقويماً نقدياً لكل من التصميم الأساسي والبديل المقترح المقدم من استشاري المقاول مع توصيات الباحثين .