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Pile foundation in karstic limestone terrain: a geotechnical constraint and its solution

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ABSTRACT

The karstic limestone terrain poses complex geotechnical problems, as there are many uncertainties in terms of position and dimension of cavities, sinkholes, pinnacles, and overhangs developed in it. The study deals with the 6.5 km long Ampang–Kuala Lumpur Elevated Highway over the rivers Kelang and Ampang in Malaysia. The large-diameter bored piles and cast in-place piles were constructed for this purpose.

The toll plaza area is underlain by karstic limestone. In that area, the piles were located in rows extended as columns connected by crosshead whereas they were provided in pile groups of 2, 3, 4, or 5 connected by a rigid pile cap elsewhere. An instrumented test pile was load tested using static method and calibrated with site investigation results. To design the friction piles and end bearing piles, direct methods (viz. exploratory drilling and rock probes) and indirect methods (viz. resistivity survey and transient electromagnetic survey) were adopted. Based on lithologs, the piles were designed using the safety factor of 2 and 3. The rock probe results were used for designing the rock socket length of the end bearing piles.

There was much variation between the designed and encountered rock head levels as revealed during the time of excavation. In such cases, the geotechnical design of end bearing piles was reviewed. As in most of the cases, rock probe holes were inclined (as they slid over a rock cliff, overhang, or pinnacle) giving rise to wrong information of rock head levels. To solve this problem at site, a new integrated approach was evolved considering the 3D subsurface topography of rock head level of the area, position and dimension of karstic features, rock mass classifications, and bearing capacity calculation. Based on the above parameters and other measures (underpinning, micropiling, grouting etc.), the piles were designed and load tested.

INTRODUCTION

Cavities, sinkholes, pinnacles, overhangs, and solution channels are the characteristic features of karstic limestone terrain. These features pose complex geotechnical problems, as their occurrence and extent of development are unpredictable. For the pile foundation design in karstic terrain, it is essential to obtain sufficient information about the ground in advance. Drilling and geophysical survey are the two commonly used tools to decipher these features. But, they have limitations in delineating the features, as they often go offline over these features. Hence, there is a need to develop a pile design method in case the advance ground information is inaccurate.

This case study is taken from the Ampang–Kuala Lumpur Elevated Highway (AKLEH) in Malaysia. The AKLEH is constructed over the rivers Klang and Ampang. It is 6.5 km long with 4 carriageways (having provision of 2 more carriage ways for future expansion), 11 interchanges, and one toll plaza.

There are two types of pile designed for the AKLEH: 1) large-diameter bored piles and 2) cast in-place piles. In the toll plaza area, the piles were driven in rows and extended as columns connected by a crosshead, whereas elsewhere they were provided in groups of 2, 3, 4, or 5 connected by a rigid pile cap. The piles varied in length from 20 to 65 m and as such, they are considered as long piles. The pile diameter varied from 1,000 to 1,500 mm in the toll plaza area, and it was 1,200–1,500 mm on the main line.

Geology of the Project area

Granite, quartzite, and limestone are the main rock types of the project area (Fig. 1). The granite contains an upper completely weathered zone made up of dense silty sand. The quartzite is highly to moderately fractured and occurs at a shallow depth. The limestone is intensely karstified owing to differential solution. Along the route, the bedrock is buried beneath the alluvium. At some locations, there is also a bed of very dense lower alluvium. The upper alluvium is generally less dense. At places (i.e. towards the eastern section), the alluvium has been mined and replace by slimes.

APPROACH TO THE PROBLEM

The toll plaza area consists of limestone and hence is more problematic than the granite area. Reconnaissance

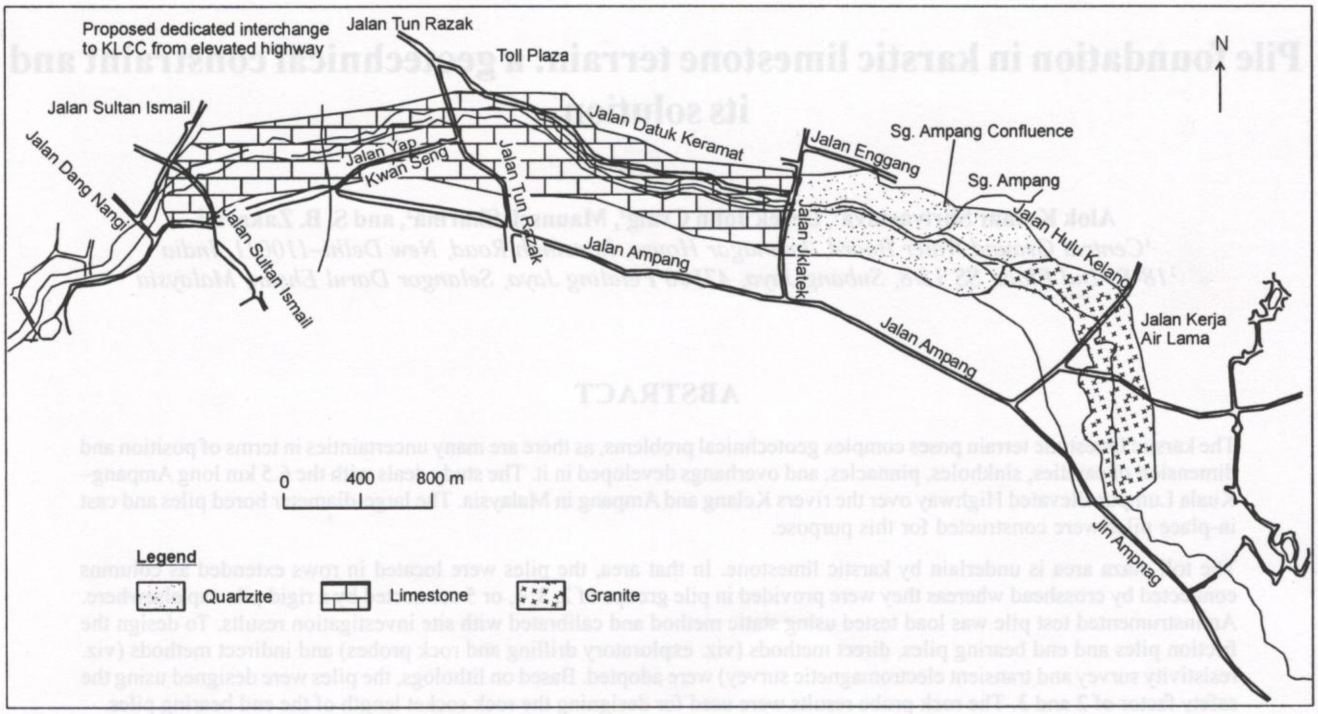

Fig. 1: Geology of the Ampang–Kuala Lumpur Elevated Highway area

boreholes were driven roughly at an interval of 200 m and showed variations in bedrock depth from 20 to more than 60 m. More boreholes were drilled during the detailed survey and they indicated similar variations but over a much shorter interval of about 25 m. As the interval of pier locations was about 45 m, it was concluded that the bedrock surface was very irregular and the ground conditions varied considerably from pier to pier.

Generally, the extent of development of the karst is shallow and does not affect the rock lying more than a few metres below the rock head. The rock head is highly irregular with pinnacles and relict ridges, and gullies are deeply incised into the surface with possible overhangs of less soluble rock. At the bedrock surface, the Rock Quality Designation (RQD) is generally low for the first two metres or so due to dissolution along the joints and bedding planes. However, the limestone is quite sound and massive at a depth of more than 2 m.

Ground conditions for piling in the limestone areas are broadly classified into three types. The first type is where thick layers of hard alluvium are found. Case histories indicate that significant shaft friction is developed in hard alluvium such that bored piles of 1 to 1.5 m diameter penetrating some 20 to 40 m into the hard alluvium are totally supported by the shaft friction. The second type of ground condition is where the limestone is deep but the hard alluvium is absent. In this case, the weak alluvium cannot support the pile by friction and end bearing on or socketing into the limestone is required. The third type is where the hard alluvium is absent and the rock head is shallow. In this case, piles get their support from end bearing on or by socketing into the limestone. These three types of ground condition lead to the selection of different types of foundation, and since the ground conditions are changed from one pier position to another, it follows that the foundations are designed to allow for these changes.

Since the pier loads are of the order of one thousand to three thousand tonnes vertically and some hundreds of tonnes horizontally at the pier head level, and since the near-surface soils are relatively weak, virtually all of the foundations are piled.

Site investigation

Site investigation was carried out by drilling a borehole at each pier location. Standard penetration tests (SPTs) were carried out in the soil whereas the rock was cored to a depth of 10 m. Where the bedrock was very deep and hard alluvium was encountered, the borehole was stopped when 12 successive SPTs exceeded 50 blows per 50 m.

Owing to the variation in ground conditions, a probe hole was drilled at each pile location. It was found that the depth of weathering of limestone was only a few metres deep and the rock head was very irregular with pinnacles and overhangs. Thus, the 10 m of coring was a practical depth for identifying these features. In addition, proof drilling at actual pile locations was carried out in order to understand more fully the rock head variation in the vicinity of the piles and to determine their stability when deriving support from the rock.

The site is made up of sand, gravel, and silty clay underlain by limestone at a depth of 8–53 m (Fig. 2). The dipole-dipole resistivity survey was carried out along five lines. The objective of the survey was to map out the soil and limestone profiles and detect any cavities in the limestone. Similarly, the transient electromagnetic survey was carried out at the site. This method located the rock head at a depth of 15–35 m in two pile-cap areas.

As the drilling rods in the karstic limestone terrain generally go offline by several metres, an attempt was made to categorise the variation between the rock probe data and the actual rock head level (Fig. 3). It was established that the variation increased with depth of the rock head. Since the indirect tools were calibrated using the site investigation or rock probe data, their results also did not match (Fig. 3).

DESIGN PROCEDURES

In the study area, the piles were designed considering the following factors.

Piles founded on rock

For the piles founded on rock, the following parameters were considered:

- Topography of limestone (sub-horizontal, inclined, sub-vertical etc);
- Karstic features (cavities, pinnacles, overhangs etc);
- Rock mass classification (CSIR, Qu, RQD; spacing, orientation, and condition of joints); and
- Bearing capacity calculation.

Based on the site investigation and rock probe data, 3D logging of pile/pile cap area was carried out to reveal the subsurface condition of limestone (Fig. 4). The basic assumptions in designing the end-bearing pile were the following:

- The stress area beneath the pile base is five times the pile diameter (5D) laterally and vertically (Fig. 5);
- The stress area can be drawn at an angle of 45°; and
- In the stress area, the rock quality should be good to excellent (CSIR classification) and devoid of any cavities.

As the piles considered are long piles, the elastic shortening due to the imposed loads could be sizable. Therefore, it is reasonable to expect that the shaft friction would support a certain component of axial load. As slip occurs between the pile and soil/rock, remaining of the axial load would be transferred to the base. The load carrying capacity of the pile thus consists of the friction component carried by the shaft and the end-bearing component provided by the rock at the base of the pile.

Friction component

From the load test carried out in similar ground conditions, the ultimate pile shaft friction was determined to correlate to

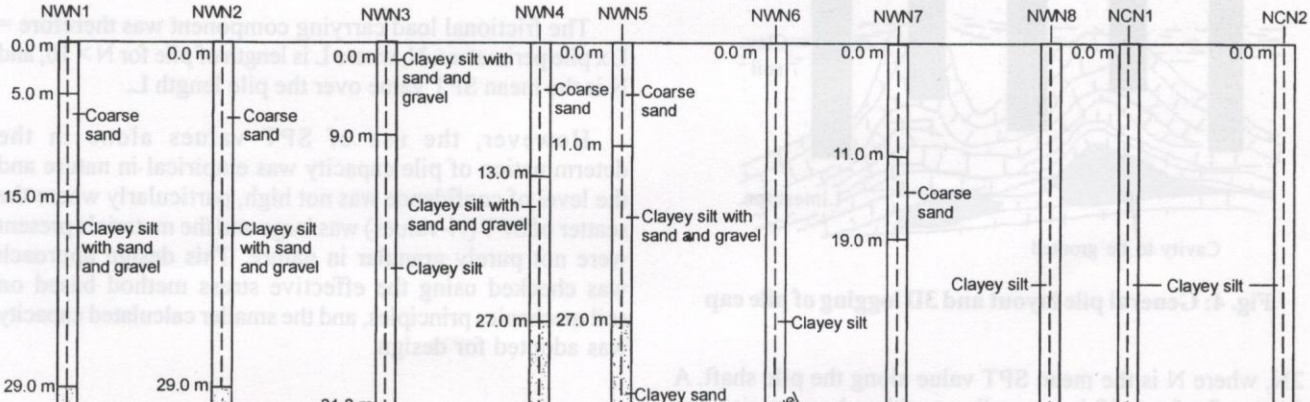

Fig. 2: Generalised soil profile of the Ampang–Kuala Lumpur Elevated Highway area

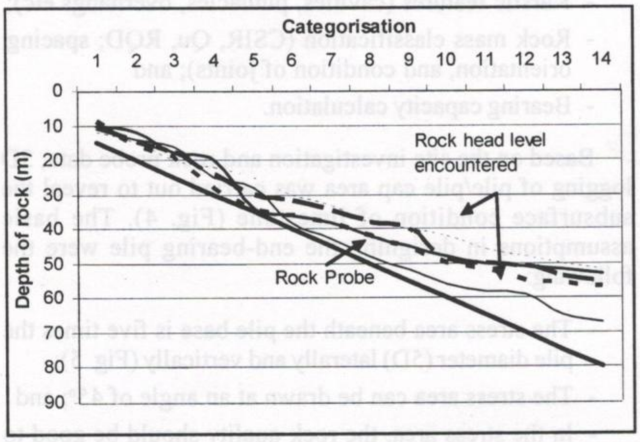

Fig. 3: Variation between rock probe results and rock head level

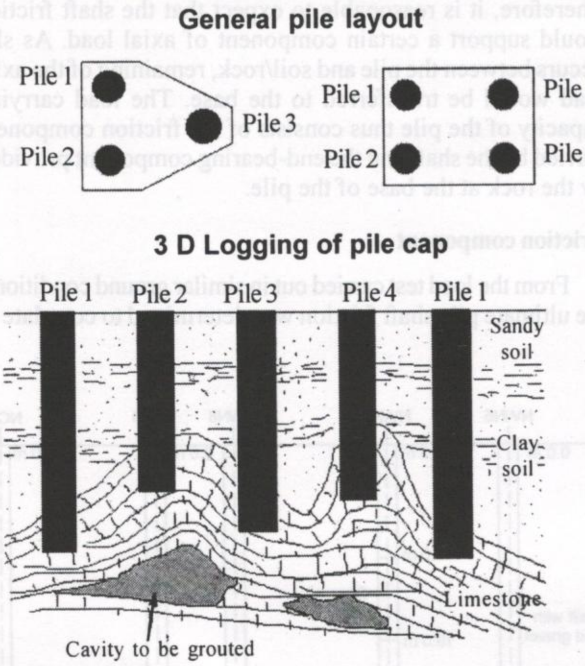

Fig. 4: General pile layout and 3D logging of pile cap

2N, where N is the mean SPT value along the pile shaft. A factor of safety of 2 is generally considered appropriate for the shaft friction. Therefore, the allowable load carrying capacity of the pile in friction can be equated to the perimeter of pile x pile length x N, where N is the uncorrected SPT value. It has been shown that the pile shaft friction is mobilised at a pile settlement of 0.5–1.0% of the pile diameter, which for a 1,500 mm diameter pile is 7.5–15 mm. This settlement is considered acceptable.

The process of boring loosens the surrounding ground resulting in the reduction of the N-value to that obtained from the site investigation. As much of the overburden was

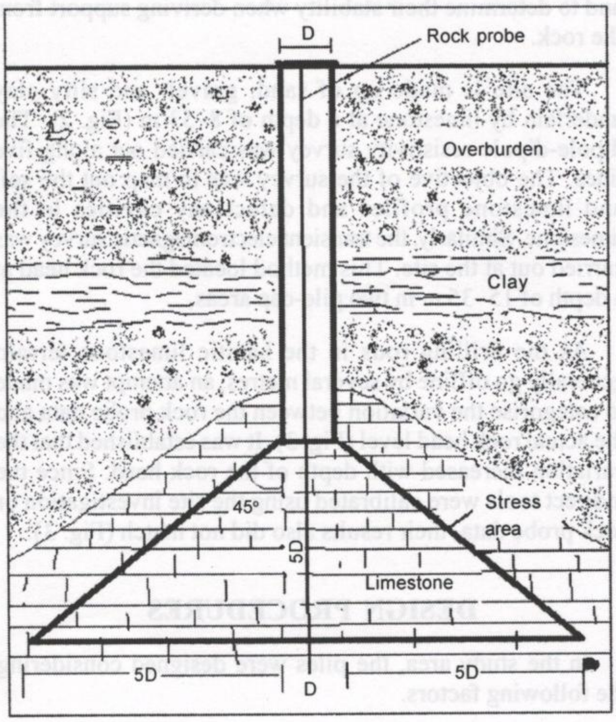

Fig. 5: Criteria for designing the end bearing pile founded on rock. D=Pile diameter

loose with SPT values less than 10, the contribution of frictional load carrying in such strata was ignored. The allowable load carrying capacity in the overburden was, therefore, limited to the strata with N-values greater than 10 and was taken as 1.0 x N.

The frictional load carrying component was therefore = L x pile perimeter x N, where L is length of pile for N > 10, and N is the mean SPT value over the pile length L.

However, the use of SPT values alone in the determination of pile capacity was empirical in nature and the level of confidence was not high, particularly where the scatter of SPT (N-values) was large and the materials present were not purely granular in nature. This design approach was checked using the effective stress method based on soil mechanics principles, and the smaller calculated capacity was adopted for design.

Rock socket

If a rock socket is provided, a significant portion of side resistance can be mobilised. However, the load carrying capacity of the rock socket is related to the RQD of the rock – the lower the RQD the less the load carried by the rock socket. The RQD values were usually very low, although some of them were due to poor drilling techniques. Because of these apparent low RQD values, the amount of the load to be carried by the rock socket in friction was carefully determined. The following two criteria were adopted simultaneously for this purpose.

1. The amount of load calculated as being carried in friction in the overburden and in the rock socket (including the amount of load transferred at the base of rock socket) was taken to be the capacity of the pile.
2. It was assumed that the load minus the load carried in friction in the overburden was carried as the end bearing on the rock (i.e. neglecting the contribution from the rock socket). Hence, the end bearing on the rock was limited to the allowable bearing pressure for the rock.

From the laboratory test, the uniaxial compressive strength of limestone has been shown to vary from 26 to 60 MPa. From Fig. 6, a uniaxial compressive strength of 30 MPa gives an ultimate shear stress of 1,200 kPa in the rock socket, and a uniaxial compressive strength of 46 MPa (mean values obtained from laboratory testing) gives an ultimate shear stress of 1,500 kPa in the rock socket. It is apparent that the shear stress developed in the rock socket is not very sensitive to the unconfined compressive strength of the rock. A value of 1,200 kPa was, therefore, taken as the ultimate value of shaft friction in the rock socket. For a factor of safety of 2 on the shaft friction, the allowable value of shear stress was 600 kPa.

Assuming that the rock socket is in the range of 2–3 m: 1) for a 1,500 mm diameter pile and a socket length of 2 m, the ratio of socket depth to pile radius is 2.67, and 2) for a 1,200 mm diameter pile and a socket length of 3 m, the ratio of socket depth to pile radius is 3.0.

Young's Modulus for grade 30 concrete is 24.6 kN/mm². The mean value of Young's Modulus for the limestone is 40 kN/mm².

Hence, Ep/Er = 24.6/40=0.6,

where Ep is Young's modulus of pile and Er is Young's modulus of rock.

From Fig. 7, for dr/r0 = 2.67, Fb/Ft = 10% (i.e. 10% of the load in the socket is carried on the base of the socket.), where dr = depth below rock surface, r0 = pile radius, Fb = load at the bottom of rock socket, and Ft = load at the top of rock socket.

Similarly, for dr/r0 = 5.00, Fb/Ft = 3%.

From the laboratory test data, the uniaxial compressive strength (Qu) of rock has been shown to be 26–64 kN/mm². As the rock was very variable and contained highly decomposed zones, the value of Qu was taken as 26 N/mm².

Allowable bearing pressure on limestone Qa = 0.2 Qu.

Qa = 0.2 x 26 = 5.2 Mpa (say, Qa = 5 Mpa).

From the above, the stress at the base of the rock socket can be taken as: 0.1 x 5,000 kPa = 500 kPa.

The depth of rock socket then was computed as:

$$\frac{\text{load on pile - load taken in friction - area of pile} \times 500}{600 \times \text{perimeter of pile}}$$

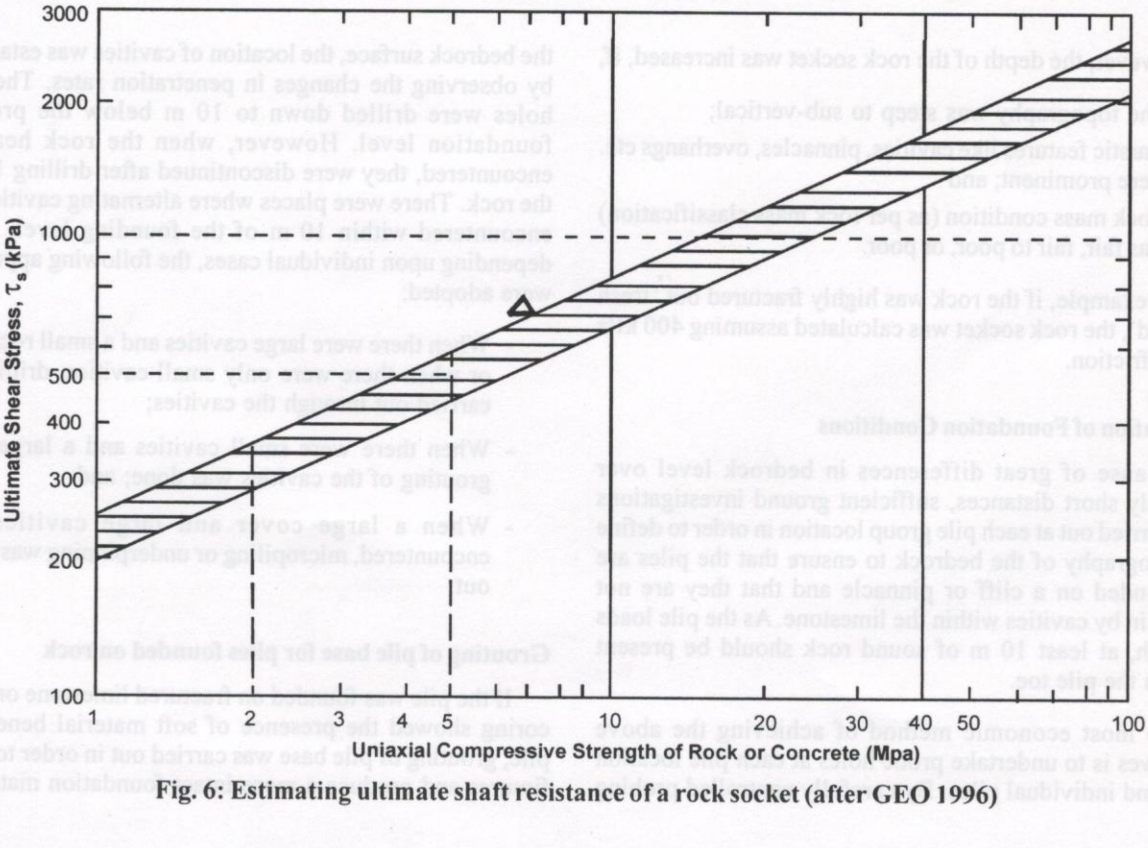

Fig. 6: Estimating ultimate shaft resistance of a rock socket (after GEO 1996)

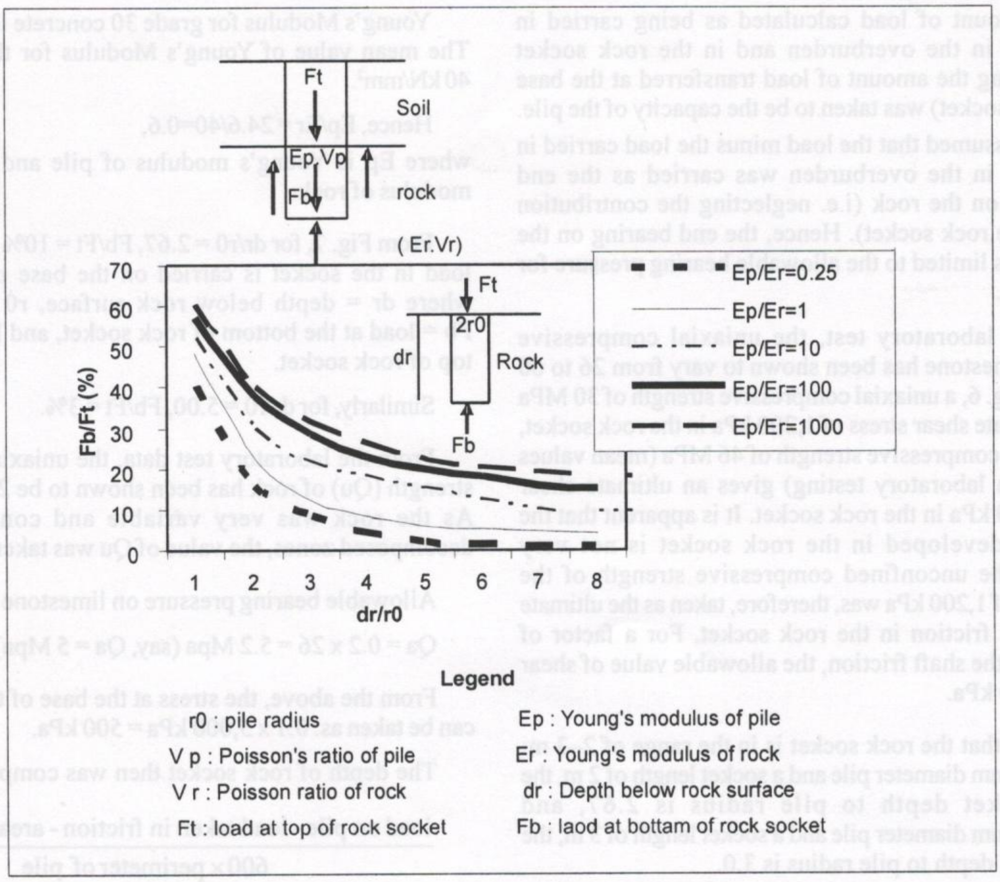

Fig. 7: Load distribution in a rock socket (after GEO 1996)

However, the depth of the rock socket was increased, if,

- The topography was steep to sub-vertical;
- Karstic features like cavities, pinnacles, overhangs etc. were prominent; and
- Rock mass condition (as per rock mass classification) was fair, fair to poor, or poor.

For example, if the rock was highly fractured but 'fresh and hard', the rock socket was calculated assuming 400 kPa of rock friction.

Verification of Foundation Conditions

Because of great differences in bedrock level over relatively short distances, sufficient ground investigations were carried out at each pile group location in order to define the topography of the bedrock to ensure that the piles are not founded on a cliff or pinnacle and that they are not underlain by cavities within the limestone. As the pile loads are high, at least 10 m of sound rock should be present beneath the pile toe.

The most economic method of achieving the above objectives is to undertake probe holes at each pile location or around individual piles. By carefully controlled probing the bedrock surface, the location of cavities was established by observing the changes in penetration rates. The probe holes were drilled down to 10 m below the proposed foundation level. However, when the rock head was encountered, they were discontinued after drilling 10 m in the rock. There were places where alternating cavities were encountered within 10 m of the founding level. Hence, depending upon individual cases, the following approaches were adopted:

- When there were large cavities and a small rock cover or when there were only small cavities, drilling was carried out through the cavities;
- When there were small cavities and a large cover, grouting of the cavities was done; and
- When a large cover and large cavities were encountered, micropiling or underpinning was carried out.

Grouting of pile base for piles founded on rock

If the pile was founded on fractured limestone or the toe coring showed the presence of soft material beneath the pile, grouting of pile base was carried out in order to fill the fissures and produce a more intact foundation material. If

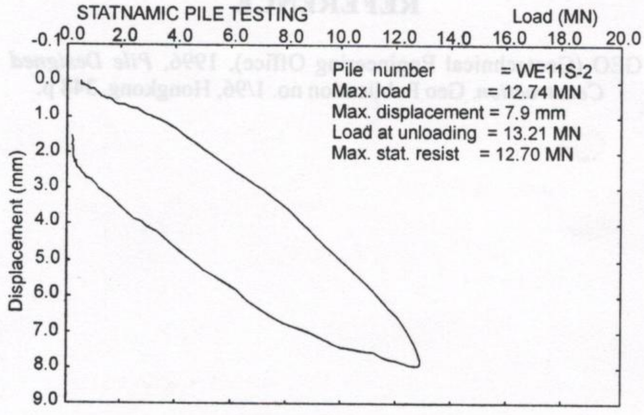

Fig. 8: Statnamic (static) load–displacement diagram

open fissures were present, the grouting reduced the foundation settlement. The grouting also permeated the rock socket and improved its load carrying capacity.

Friction piles not founded on rock

The results of pile load test confirmed that a high base pressure is mobilised within acceptable settlement limits. However, as the ground conditions along the AKLEH were very variable and the correlation of the SPT (N-values) with the test pile results was not appropriate where the ground conditions differed significantly. Hence, the end bearing in case of friction piles was not taken into account.

Owing to the unpredictable nature of the limestone terrain and deviation of drilling holes, the following approach was adopted:

- If the rock was encountered at a shallow depth, it was excavated down to 3 m, and coring/grouting and/or micropiling was provided for further investigation;
- If the rock was encountered at a deep level, the foundation was designed as overburden friction and rock friction; and
- If the rock was encountered at an intermediate depth, either of above approaches was adopted.

Pile base grouting

Because of the likelihood of significant amounts of soft material remaining at the base of the pile, the base of each pile was grouted in order to produce better contract between the base of the pile and the underlying strata, thereby reducing the possibility of a "soft toe".

Pile stiffness

For detailed pile design, the point of fixity were considered as 10 m below the ground level for both 1,200 mm and 1,500 mm diameter piles on the main line. An interactive pile–soil analysis with complete load combination was recommended for further detailed analysis.

Load test

An instrumented test pile was load tested by the "maintain load test" method in similar ground conditions for calibrating the shaft friction with site investigation results. Generally, the 10% of piles were load tested during their construction. However, if their founding levels had been reviewed due to the variation in design and actual ground conditions, all the piles were load tested. The load tests were carried out using the following methods.

Maintain load test

The "maintain load test" was carried out by one of the following ways:

- By means of a jack that obtains its reaction from kentledge heavier than the required load; and
- By means of a jack that obtains its reaction from tension piles or other suitable anchors.

Statnamic load test

A recent technique called "statnamic load test" was performed on production piles after calibration and correlation with the load test result using the "maintain load test" in similar ground conditions. All the load tests on production piles were successful as per settlement acceptance criteria (> 1.0 % of pile diameter). Fig. 8 shows the results of static and statnamic tests carried out with one of the piles.

CONCLUSIONS

The efficient use of large-diameter bored piles requires a straightforward definition of the founding level so that the machines are not held up whilst the founding levels are approved. On an irregular karst surface, the detailed knowledge of the rock surface for each pile and between piles under each pier is required. For this purpose, the level of rock head and the depth of weathering and orientation of joints were determined.

For a pier, a probe hole was carried out at each pile location. The purpose of the probe was to determine the rock head level, rate of penetration, and the depth to sound rock at each pile location. From the probe hole results, variations in bedrock levels (i.e. 3D subsurface topography) between the pile positions were drawn, features such as pinnacles and overhangs were recognised, and the target founding levels for the piles in the group were determined.

Having set up the pile boring machine, rates of penetration were observed and the boring was stopped when the desired resistance to driving was met. Then, the level reached was compared with the level determined from the probe hole. It was observed that the probe holes went offline and when the rock head was very steep, the levels encountered in the piles and by the probe holes differed by several metres.

Hence, the pile design was reviewed based on the 3D subsurface topography of rock head level of the area, position and dimension of karstic features, rock mass classifications, and bearing capacity calculation.

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