FOUNDATION DESIGN AND CONSTRUCTION PRACTICE IN LIMESTONE AREA IN MALAYSIA

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Abstract: The design and construction of foundations in limestone areas have posed various problems to geotechnical engineers due to the karstic features of limestone such as steeply inclined bedrock, cavities, floaters, etc. The design of foundations in such highly irregular ground conditions requires careful planning and execution of the works from preliminary to detailed subsurface investigation, analysis and design, and up to the construction stage where continuous feedback is essential for the satisfactory performance of the foundations. This paper presents some aspects of foundation design and construction practice in limestone areas in Malaysia with particular emphasis on the design of piled foundations such as driven piles, jacked-in piles, bored piles and micropiles.

INTRODUCTION

The design and construction of foundations in limestone areas have posed various problems to geotechnical engineers due to the karstic features of limestone such as steeply inclined bedrock, cavities, floaters, etc. Karst refers to a characteristic topographic feature or landscape which can be developed by rock undergoing dissolution by percolating meteoric water (Jakucs 1977). In Peninsular Malaysia, under tropical humid conditions, calcite and dolomite limestones or their metamorphised equivalents develop tropical features which show spectacular tall steep-sided hills (Jennings 1982) and solution features such as pinnacles, sinkholes and cavities. The treacherous and almost unpredictable karstic bedrock associated with extremely variable overburden soil properties is a typical feature of limestone (Yeap 1985), which leads to a variety of geotechnical problems and hazards. Some of these common engineering problems are discussed below (Gue 1999) and Figure 1 shows some typical piling problems in limestone areas.

Pinnacles

Pinnacles are columns or cones of limestone or marble left by dissolution of the surrounding rock. Pinnacles with soft or loose overburden immediately above them pose tremendous challenges to geotechnical engineers to ensure proper seating of piles on the rock, particularly for driven concrete piles, as well as the difficulty of rock socketing for bored piles.

Subsidence

Subsidence is described as several related localized or already widespread phenomena associated with the sinking of the ground. The vertical movements ranged from sudden collapse to slow settlement. Generally, subsidence can be caused by the removal of subsurface fluids, drainage or oxidation of organic soil and surface collapse into natural or excavated subsurface cavities had taken place. The formation of subsidence in the Kuala Lumpur and Ipoh areas is often associated with the occurrence of soft mining slime in ex-mining areas

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upon which housing and roadwork projects are carried out. This is due to the consolidation of the underlying slime/clay upon loading and proceeding at a gradual or slow rate.

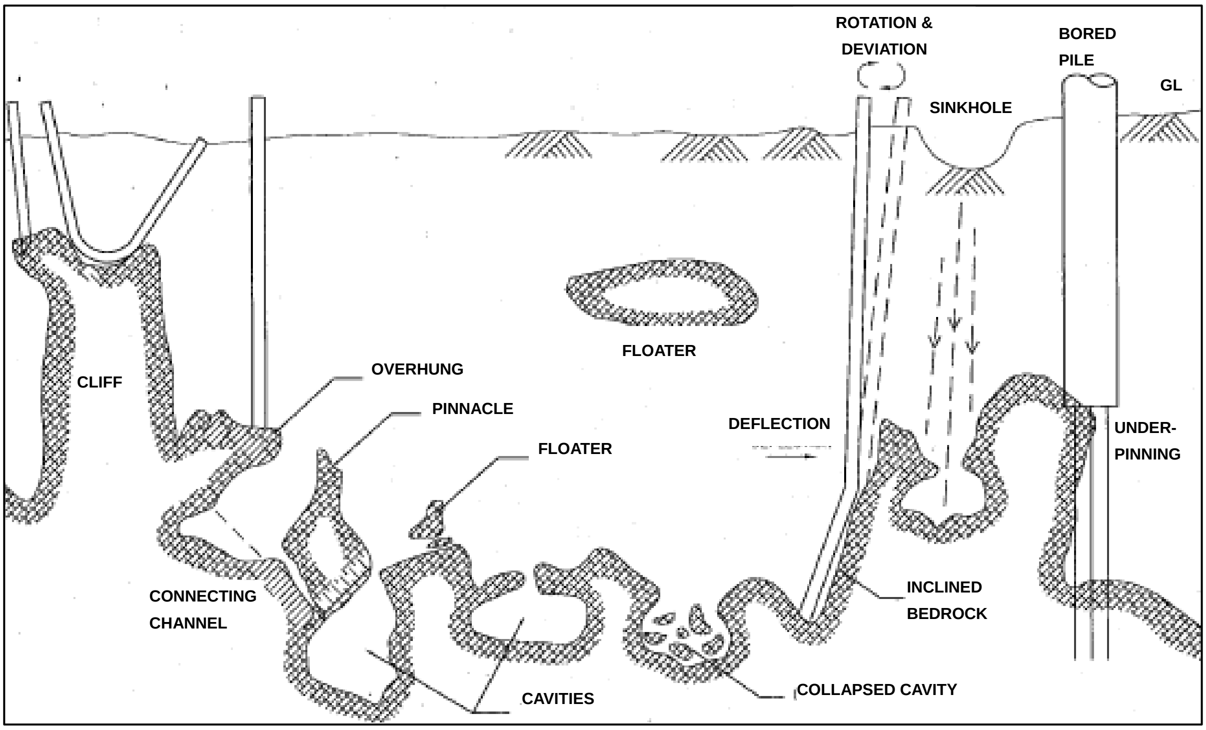

Figure 1: Some piling problems in limestone areas (from Neoh 1998).

Sinkhole

This is a common phenomenon in karst areas, especially areas with loose and non-cohesive sands over limestone bedrock. It is commonly known that limestone can be dissolved by acidic solution from rain or polluted groundwater. After a certain period of time, flow or penetration of groundwater through weak zones in the limestone will develop channels and voids. These channels will act as passages for water together with the loose sand to flow into voids and cavities in the limestone. The movement of sand into existing cavities or voids will then develop empty spaces in the sand layer where soil arching occurred. This continuous process will reach a critical stage where the roof of the space will no longer support the weight of the overburden. This will result in the occurrence of sinkholes or cave-ins as illustrated in Figure 2 and Figure 3. Sometimes, the delicate balance of the arching mechanism that supports the overburden soil can also be disrupted by lowering of groundwater level due to construction activities (e.g. excavation dewatering) or by dynamic loadings from sources such as earthquakes and railway lines. Sinkholes are hazards to both shallow and deep foundations as their emergence is sudden and catastrophic and certainly pose tremendous challenges to the geotechnical engineering profession (Figure 4).

Slump zones

Zones of weakness often occur immediately above the bedrock of limestone. The slump zone above limestone formation is usually identified by the very low SPT-N values or low cone resistance where SPT-N values of zero are often detected. The formation is either due to subsurface erosion as a result of overburden slumping into cavities in the limestone or residual weathering of ancient karst features (Tan & Ch’ng 1986).

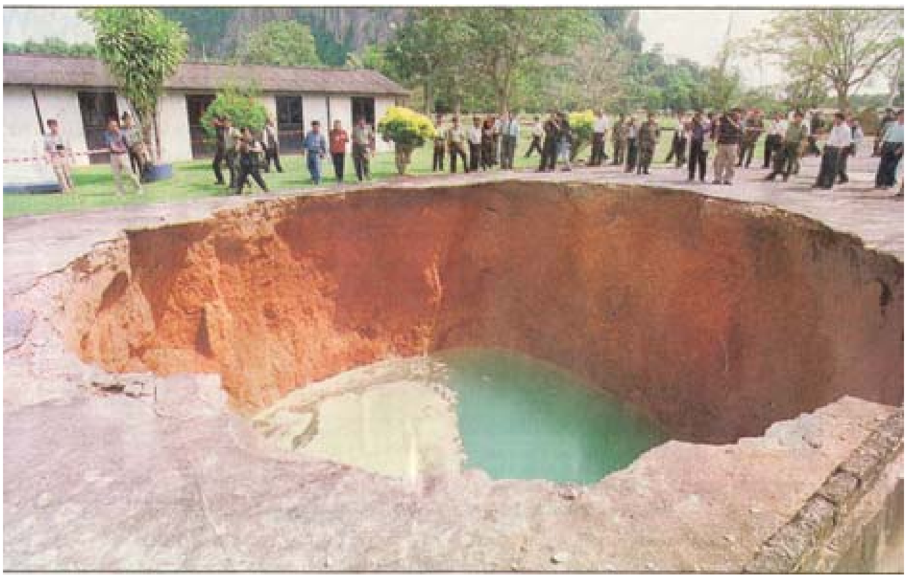

Figure 2: Sinkhole in Mata Ayer, Perlis.

Figure 3: Sinkhole near Ipoh, Perak.

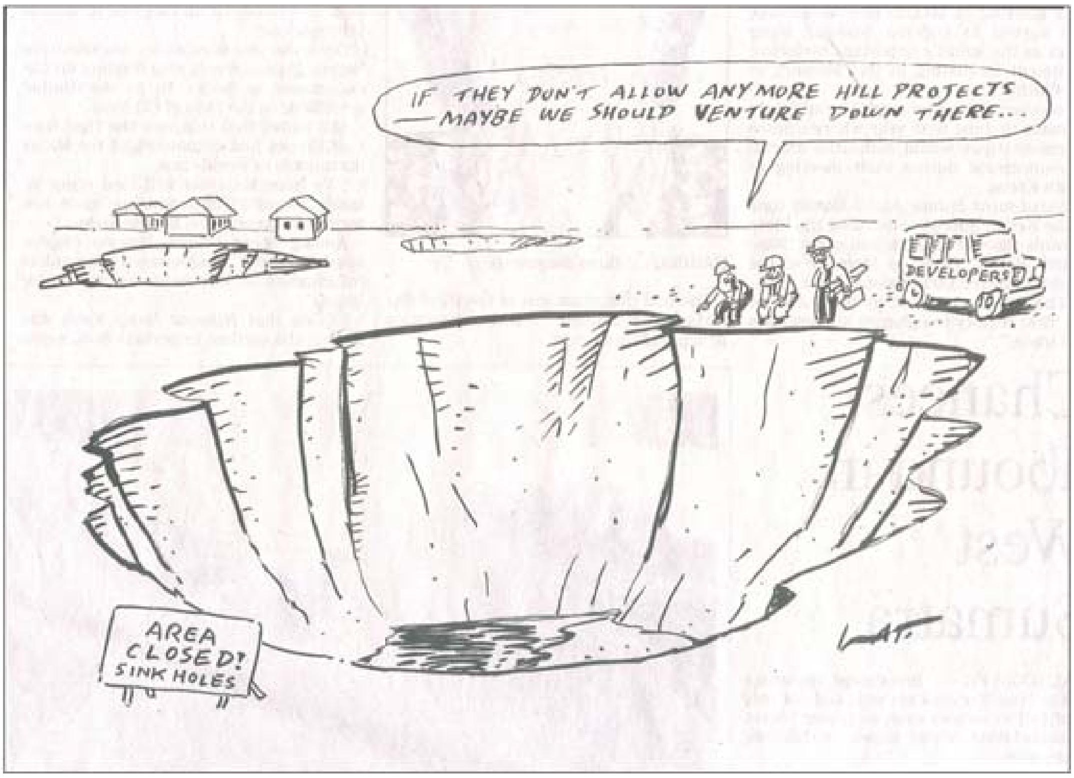

Figure 4: A lighter note on the challenges faced by geotechnical engineers (from LAT).

Caverns/Cavities

Cavities are voids formed by dissolution of the rock in the limestone which will pose problems if the roof of the cavities is not of sufficient strength to support the foundation resting on them, especially for empty or partially filled cavities (Figure 5).

Figure 5: Cavern/cavity exposed after excavation.

Steeply inclined bedrock surfaces

Steeply inclined bedrock surfaces in limestone posed significant difficulties for piled foundations such as inadequate end-bearing resistance and pile breakage during installation of driven piles, especially for bedrock with adverse geological features such as vertical joints as shown in Figure 6 and Figure 7 which are not uncommon.

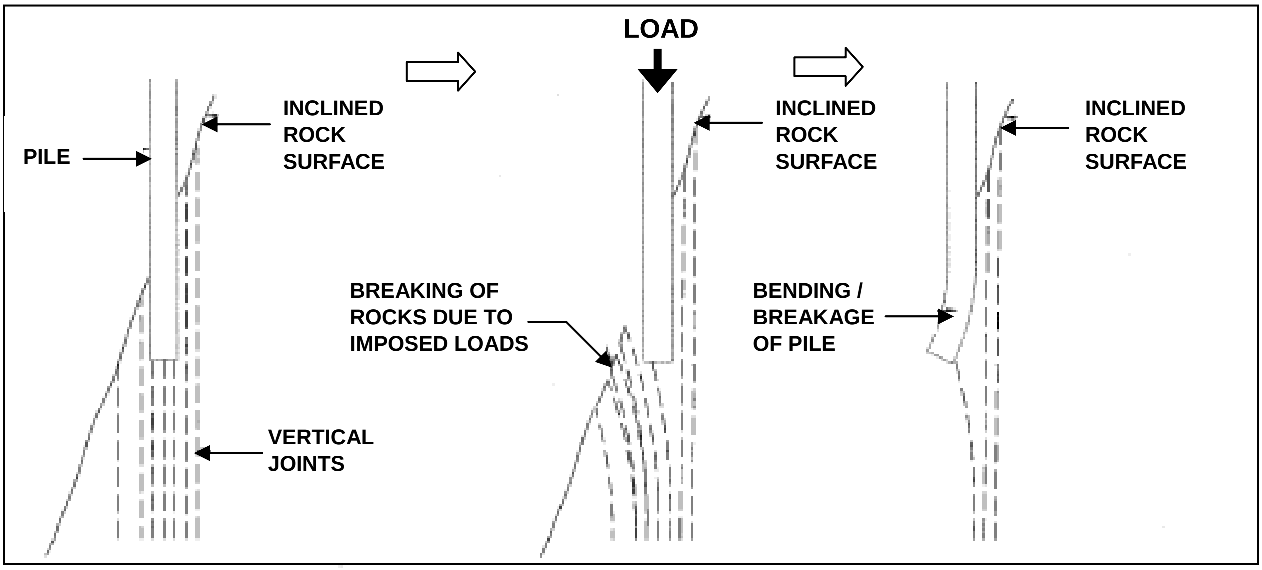

Figure 6: Pile breakage in steeply inclined bedrock surface with adverse geological features.

Figure 7: Vertical joints in exposed limestone in the Kuala Lumpur area.

SUBSURFACE INVESTIGATION

Subsurface investigation (SI) in limestone areas for the design of foundation is quite similar to other formations except it needs to be more extensive. The investigations should concentrate on the rock profile and its properties, especially the extent of weathering near the contact zone; and karstic features especially cavities.

The subsurface investigation in limestone would ideally be carried out in three stages. The preliminary stage normally includes conventional boreholes with geophysical survey such as seismic survey, microgravity, resistivity, Transient Electromagnetic Method (TEM), Site Uniformity Borehole Seismic (SUBS) and Seismic Tomographic Imaging (STI) unless the development site is very small where boreholes can be economically implemented to provide the required information with sufficient confidence. The advantages of using the geophysical survey method include (Poulos 2003):

a) Better defining bedrock level
b) Identifying weak or soft layers within the zone of interest
c) Estimating the stiffness of the various geotechnical materials

The use of geophysical surveys however, requires experienced specialist interpretation in order to produce reliable results. The use of geophysical methods is also limited in urban areas due to noise disturbance which affects seismic procedures. Underground utilities/services, on the other hand, affects methods based on electrical/electromagnetic forces.

Detailed subsurface investigation is usually carried out in areas of high loading intensity and areas with anomalies as detected by the geophysical survey and preliminary boreholes. The drilling of boreholes in limestone areas is commonly terminated upon 10m of continuous solid coring into rock to ensure that the interpretation of bedrock level is correct. A shorter termination criterion is not recommended to prevent misinterpreting floaters, overhang, etc. as the founding bedrock level in the highly irregular karstic features of limestone.

Cavity probing before or in the early phase of construction is usually needed depending on the type of foundation selected. Cavity probing aims at the doubtful areas to collect additional information on the size of cavities and the properties of the in-fill materials. If micropiles (which are relatively costly compared to other foundation systems) are adopted as foundation systems and proper logging is carried out during installation, the extent of cavity probing can be reduced significantly. Information on the cavity is needed for the design and modification to suit the soil and rock condition of the site.

Hence, input and design review from the design team are very important during construction to ensure that the expected subsoil model interpreted during the design stage is in reasonable agreement with the actual site conditions, otherwise modification to the design or additional measures is needed to ensure safety of the foundation to support the imposed loads. Inadequate site supervision by the Engineer during construction is often dangerous and will lead to potential cost overrun.

FOUNDATION DESIGN

Bored piles and barrettes are generally used for high-rise buildings while driven/jacked-in piles are adopted for low-rise buildings. However, with the introduction of higher capacity jacked-in piling systems, jacked-in piles can also be adopted for high-rise buildings. The size of bored piles ranges between 600mm to 1500mm diameter although 3000mm diameter has also been installed in Malaysia. Barrettes are sometimes used in thick overburden (e.g. more than 60m) where bored pile machines have difficulty reaching the required depth. Hand-dug caisson foundations have also been designed and constructed in Malaysia. Yee & Yap (1998) reported the design and construction of hand-dug caisson foundations of up to 3.0m diameter for a 33-storey tower in Kuala Lumpur together with other measures such as cavity detection, replacement grouting and verification of cavity grouting to ensure satisfactory foundation performance in the highly unpredictable limestone formation.

Driven and Jacked-In Piles

In this section, some general design considerations associated with driven and jacked-in piles in limestone area are discussed. Driven piles have been installed in limestone formations in Malaysia with varying degree of success. Various problems such as excessive tilting/deflection, rotation, distortion, buckling/bending, cracking, shattering, etc leading to high percentage of damaged piles could be expected if inadequate attention is given during the design and construction of the piles in limestone foundations. Recently, high capacity jacked-in piles have also been installed in Malaysia. Pile capacities of up to 1800kN have been installed in Malaysia where the piles are jacked to at least two times of their capacity, i.e. 3600kN. Figure 8 shows a typical hydraulic jacked-in piling system used in Malaysia where jacking force of up to 6000kN can be achieved.

Figure 8: A typical hydraulic jacked-in piling system used in Malaysia.

Some design aspects related to driven and jacked-in piles in limestone areas are as follows:

a) Sloping bedrock/steeply inclined bedrock surface
b) Floating boulder
c) Cavity

Poulos (2003) has carried out theoretical studies on the two aspects above with relation to foundation performance and the findings are summarized as follows:

a) Studies of a hypothetical 9-pile group which is assumed to be founded on bedrock which has a surface inclined at an angle, α to the horizontal has been carried out. Two cases are analysed, i.e. end-bearing group and floating group as shown in Figure 9. The findings have concluded that the settlement and load distribution of pile groups at normal working loads is unlikely to be affected significantly by a sloping bedrock surface, but rotation of the group may develop, both for end-bearing and floating groups. This will in turn, induce bending moments in the piles as shown in Figure 10 and Figure 11.

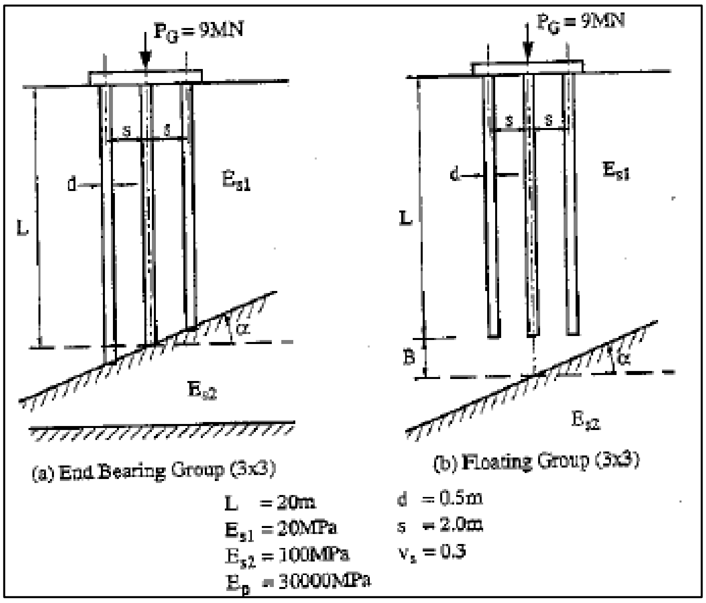

Figure 9: Pile groups analysed for effect of sloping bedrock (from Poulos 2003).

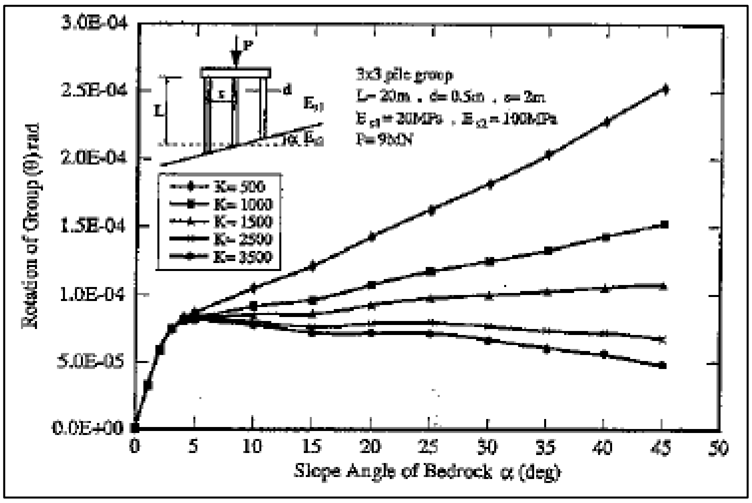

Figure 10: Effect of sloping bedrock on rotation of end-bearing pile group (from Poulos 2003).

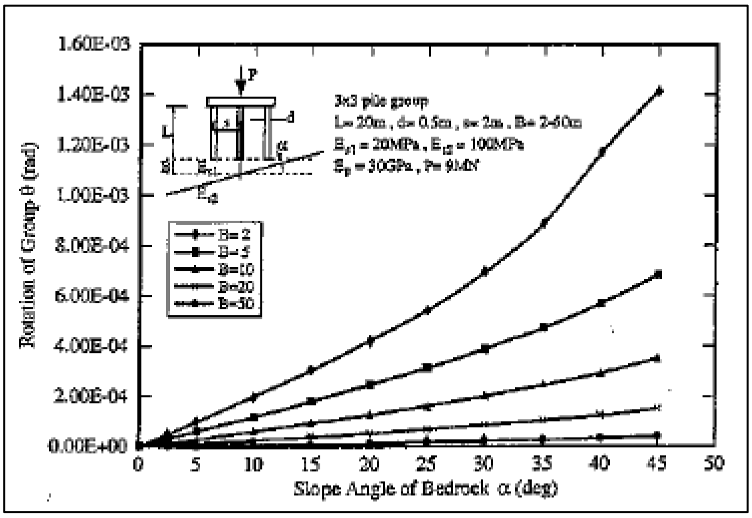

Figure 11: Effect of sloping bedrock on rotation of floating pile group (from Poulos 2003).

It must be noted that the above studies are only carried out for sloping bedrock with the angle of inclination of the bedrock up to 45°. In limestone areas, much steeper bedrock is possible and as such, the rotation, and hence bending moments induced in the piles, would be greater. This is illustrated in Figure 12 where the interpreted bedrock profile for a project site in Kuala Lumpur, based on subsurface investigation information and piling records, shows a highly erratic bedrock profile with regular occurrences of boulders/floaters associated with limestone formations. With proper planning of SI and cavity probing and treatment, jacked-in piles have been successfully installed in this site for a 21-storey high low cost apartment.

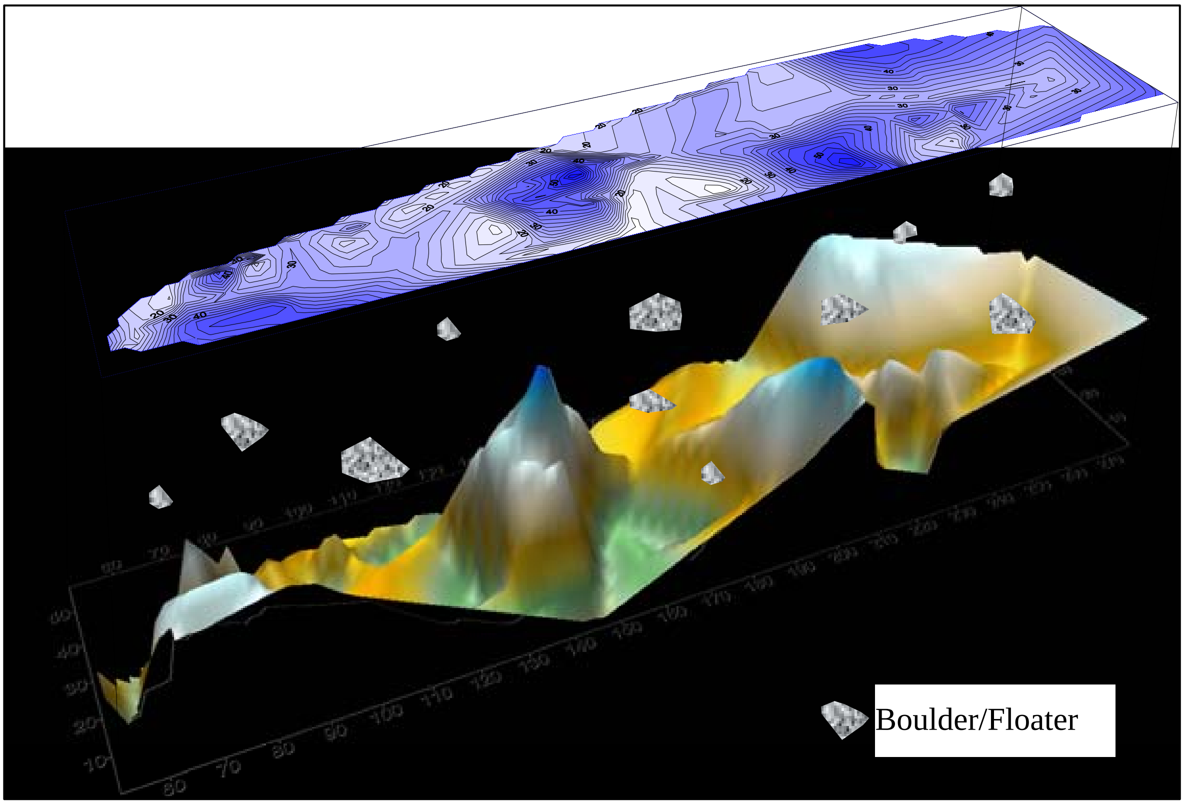

Figure 12: Highly erratic simplified bedrock profile for a project site in Ampang, Kuala Lumpur.

In Malaysia, the design of pile foundations to cater for highly erratic bedrock profiles and sloping bedrock involves the following:

i) Provision of compensation piles within the pile group (if necessary) to ensure that the induced rotation within the group is within tolerable limits, i.e. within the bending moment capacity of the pile and pilecap-column connection and no piles within the group are overstressed. This also applies in situations where significant differences in pile length are observed within the same pile group due to the highly irregular bedrock profile of limestone areas. Such large differences in pile length will induce bending moments and also uneven distribution of loads within the pile group due to different magnitudes of elastic shortening of the piles. Therefore, provisions for higher percentages of compensation piles are usually required for driven/jacked-in piles foundation in limestone area due to the complex geological settings of limestone formation. In addition, the risk of pile damage during driving is also higher in limestone area. This, however, can be minimised with competent site supervision and experienced contractors.

ii) Provision of Oslo-point rock shoes (Bjerrum 1957) in areas where the overburden soils are soft or loose to prevent pile deflection during installation and to ensure the pile toe is properly secured to the rock as illustrated in Figure 13. The hardness of the hardened steel used for Oslo-point rock shoes should be larger than 300 (Brinell hardness) and the yield strength of the rock shoe should not be less than 760 MPa. The rock shoe should be designed to take the full required load at the contact and extra care should be taken during construction to prevent altering its properties, in particular, by welding. Typical details of Oslo-point pile shoes are shown in Figure 14.

iii) Adjustment of rock sockets based on the actual bedrock surface encountered during construction to ensure sufficient socket capacity for rock socketed bored piles or micropiles, especially in steeply inclined bedrock areas with adverse geological features or in pinnacle/cliff areas as illustrated in Figure 15. This illustrates the importance of input during construction for the successful design and construction of foundations in limestone areas. The information obtained during SI can be greatly enhanced by input during construction as the bedrock level can be continuously updated as piling works progress. In addition, the inclination of the bedrock surface can be deduced based on the bedrock level as encountered during pile construction/installation, and refinement to design can then be continuously carried out as construction progresses.

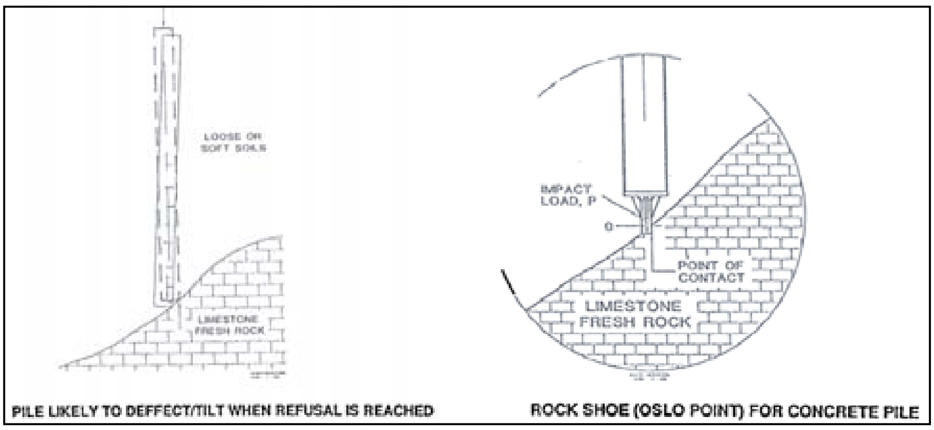

Figure 13: Mechanism of Oslo-point.

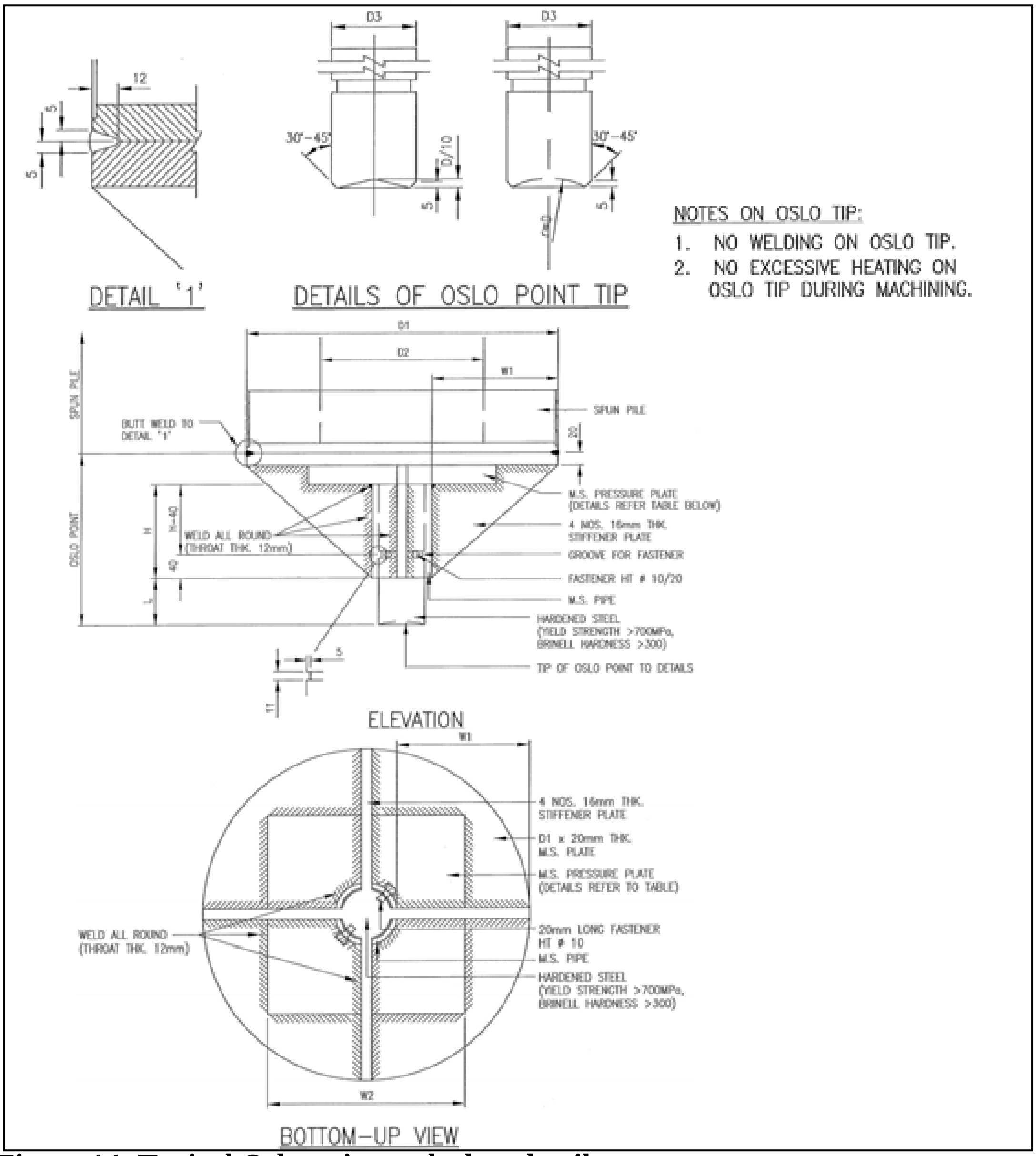

Figure 14: Typical Oslo-point rock shoe details.

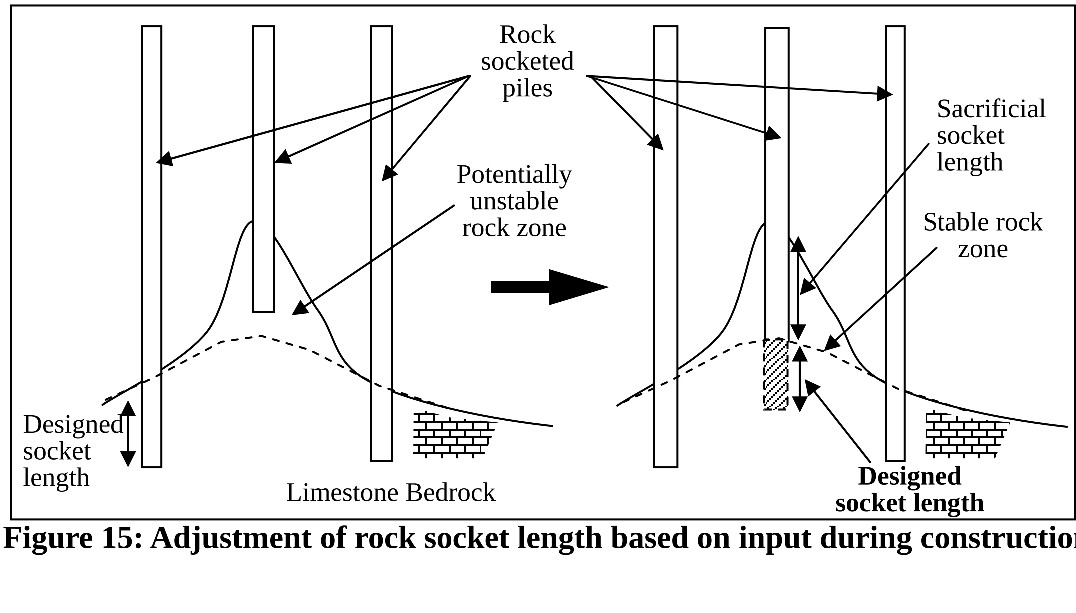

Figure 15: Adjustment of rock socket length based on input during construction.

b) The effect of founding the pile foundations on floating boulder as idealised in Figure 16 is that the axial load in the piles founded on the intended founding layer increases, while the axial load in the piles on the floater decreases. This will induce uneven settlement in the pile group and hence, rotation is developed which will induce bending moments in the piles at the pile heads and also causes potential overstressing of piles founded on the intended founding layer.

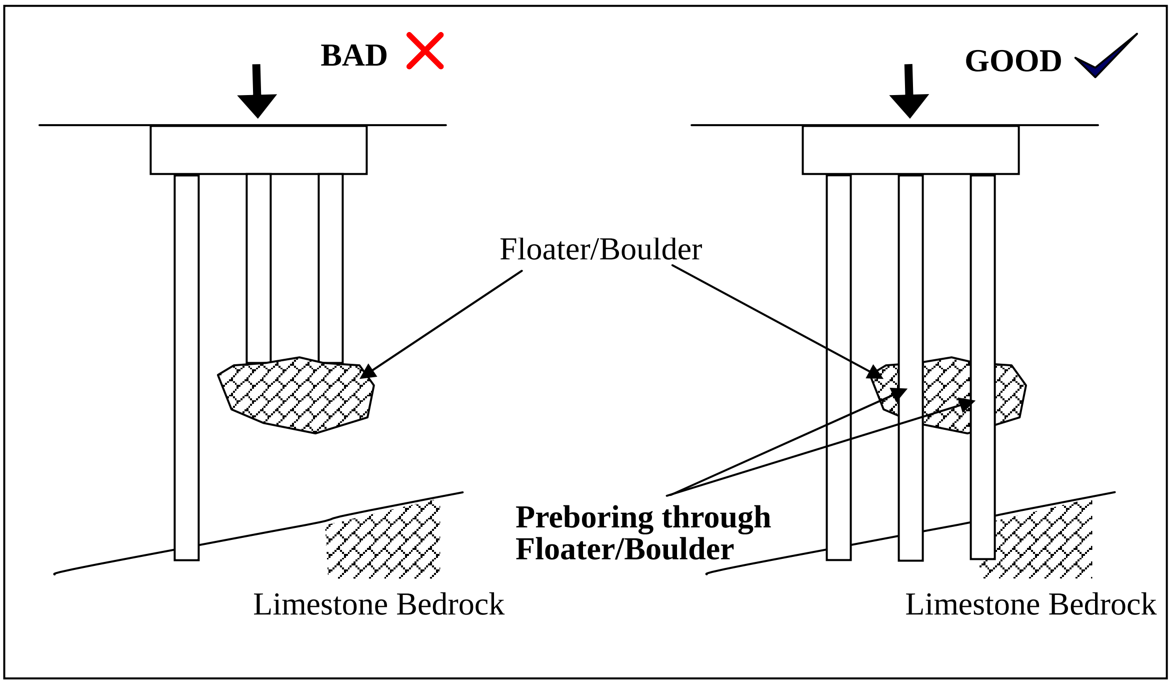

Figure 16: Pile foundation on floating boulders.

Therefore, to cater for such situations, the following is normally carried out:

i) Preboring through the floater prior to installation of driven/jacked-in piles. This is to ensure that the piles reach the intended founding layer.

ii) In the event that the driven/jacked-in piles terminate prematurely on the floater, provision of compensation piles should take into consideration the reduced capacity of the piles founded on the floater and the pilecap/tiebeam should be sufficiently stiff and adequately designed to span across the piles founded on the floater.

c) If the extent of the cavity is limited, treatment of cavity using grout can be carried out. The essential steps for successful treatment of cavity and slumpzone involve:

i) Cavity and slumpzone probing

ii) Injection of grout/mortar

iii) Verification of cavity grouting

Cavity and slumpzone probing should be carried out using a suitable drilling machine to a minimum depth of 10m into solid limestone if no cavity is encountered or 10m below the last cavity encountered. Based on the results of cavity and slumpzone probing, the required treatment should be carried out using grout/mortar according to the following sequence:

i) If there is more than one drill hole for treatment, generally mortar injection should commence around the perimeter of the treatment zone and then proceeding toward the centre. Each hole should be drilled and grouted before moving to the next hole.

ii) In the case of multiple cavities or multiple limestone layers in any drill hole, treatment should proceed from the lowest cavity and completed for that cavity before proceeding to the next higher cavity.

iii) If required, packer(s) are to be adopted to prevent flow out of the grout/mortar before achieving the required criteria of acceptance or pressure specified. Each drill hole for grout treatment may be accompanied by at least one vent hole or pressure release hole of similar depth and size.

Acceptance criteria for cavity treatment using grout/mortar are commonly based on the following criteria:

i) For soils within the treatment zone, the individual SPT-N values at any point are not less than 20 and the average SPT-N value is not less than 25

ii) No void is encountered

iii) Unconfined compressive strengths of the cores (if required) are in excess of 2 N/mm2 or other strength requirements as per design

Figure 17 shows typical drilling machine and pump for cavity treatment using grout/mortar.

Figure 17: Typical drilling machine and pump for cavity treatment using grout/mortar.

Good construction practice is also very important to ensure successful installation of driven/jacked-in pile foundations in limestone areas especially for sites where steeply inclined bedrock and floaters are expected. Some good construction practices are summarized below:

a) Based on available boreholes and cavity probing points, interpretation of the bedrock profile is carried out. The interpreted bedrock profile will serve as reference during pile driving where the hammer height is reduced when approaching the interpreted bedrock profile to prevent slip-off of pile point. However, the Engineer should be aware that the interpreted bedrock profile is only a rough guide as the limestone is usually highly irregular in depth and therefore, good engineering judgement must be exercised. When the pile point has come into contact with the rock surface which normally can be recognized by a sudden change in the response of the hammer, pile driving is then continued with very small heights of drop of the ram (typically about 100mm to 200mm). After the pile has been subjected to a series of blows until the penetration of the pile is negligible, the fall is increased to double the height. The steps are repeated until the required termination criterion is achieved. This procedure is intended to socket the pile into competent bedrock and to prevent sliding of the pile point at the contact with the rock surface.
b) High strain dynamic pile test should be used to calibrate the permissible drop height to prevent damage to piles during installation of driven piles. It also serves as a useful tool for quality control during pile installation and detection of damaged piles. For preliminary estimation of pile driving criteria, methods based on wave equations, e.g. using software such as GRLWEAP, should be used to determine the permissible drop height and set criteria. The use of dynamic formulas (e.g. Hiley) is strongly discouraged as this is fundamentally incorrect.
c) In the event of premature pile termination due to the existence of intermediate hard lenses (high SPT-N value) or small boulders, the problem can be overcome by applying a higher jack-in force or increasing the driving energy (a heavier hammer is preferred to higher drop heights to reduce potential pile damage). Again, the use of high strain dynamic pile tests is recommended to monitor pile stresses during installation of driven pile in such situations to ensure the compressive and tensile stresses induced in the pile are within tolerable limits. Similarly, software such as GRLWEAP is also useful to study the pile driving characteristics in such situations prior to implementation at site.

Bored Piles

In Malaysia, bored pile design in limestone is heavily dependent on semi-empirical methods. Generally, the design rock socket friction is a function of the surface roughness of rock sockets, the unconfined compressive strength of intact rock, the confining stiffness around the rock socket in relation to fractures of rock mass and socket diameter, and the geometry ratio of socket length-to-diameter. Roughness is an important factor in rock socket pile design as it has significant effects on the normal contact stress at the socket interface during shearing. The normal contact stress increases due to dilation, resulting in increased socket friction. The degree of dilation is mostly governed by the socket roughness. The second factor on the intact rock strength governs the ability of the irregular asperity of the socket interface transferring the shear force, otherwise shearing through the irregular asperity will occur due to highly concentrated shear forces from the socket. The third factor will govern the overall performance of strength and stiffness of the rock socket in jointed or fractured rock mass and the last factor is controlled by the profile of socket friction distribution. It is very complicated to quantify all of these aspects in rock socket pile design. Therefore, based on local experience, some conservative semi-empirical methods have evolved to facilitate quick and simple rock socket design taking into considerations the factors discussed above. In most cases, roughness of socket is only qualitatively assessed due to lack of systematic and reliable methods of assessment. The other three factors can be quantified through strength tests on the rock cores and point load tests on the recovered fragments, RQD values of the core samples and some analytical method of assessing the socket distribution. It is also customary and important to perform preliminary and working load tests to verify the rock socket design using such semi-empirical methods. A safety factor of two is a common requirement for rock socket pile design. Table 1 summarises typical design/working socket friction values for limestone formations in Malaysia:

Table 1: Summary of Rock Socket Friction Design Values for Limestone Formations in Malaysia (Neoh 1998).

Working Rock Socket Friction	**Remarks**
300kPa for RQD < 25% 600kPa for RQD = 25 – 70% 1000kPa for RQD > 70%	The working/design values given are subject to 0.05 x minimum of (q_{uc}, f_{cu}), whichever is smaller. q_{uc} = unconfined compressive strength of intact rock f_{cu} = compressive strength of concrete/grout for piles

Another more systematic approach developed by Rosenberg & Journeaux (1976), Horvath (1978) and Williams & Pells (1981) is also referred to in Malaysia. The following simple expression is used to compute the rock socket friction, f_s with consideration of the strength of intact rock and the rock mass effect due to discontinuities:

$$f_s = \alpha * \beta * q_{uc}$$

where

q_{uc} = unconfined compressive strength of intact rock

α = reduction factor with respect to q_{uc} (Figure 18)

β = reduction factor with respect to rock mass effect (Figure 19)

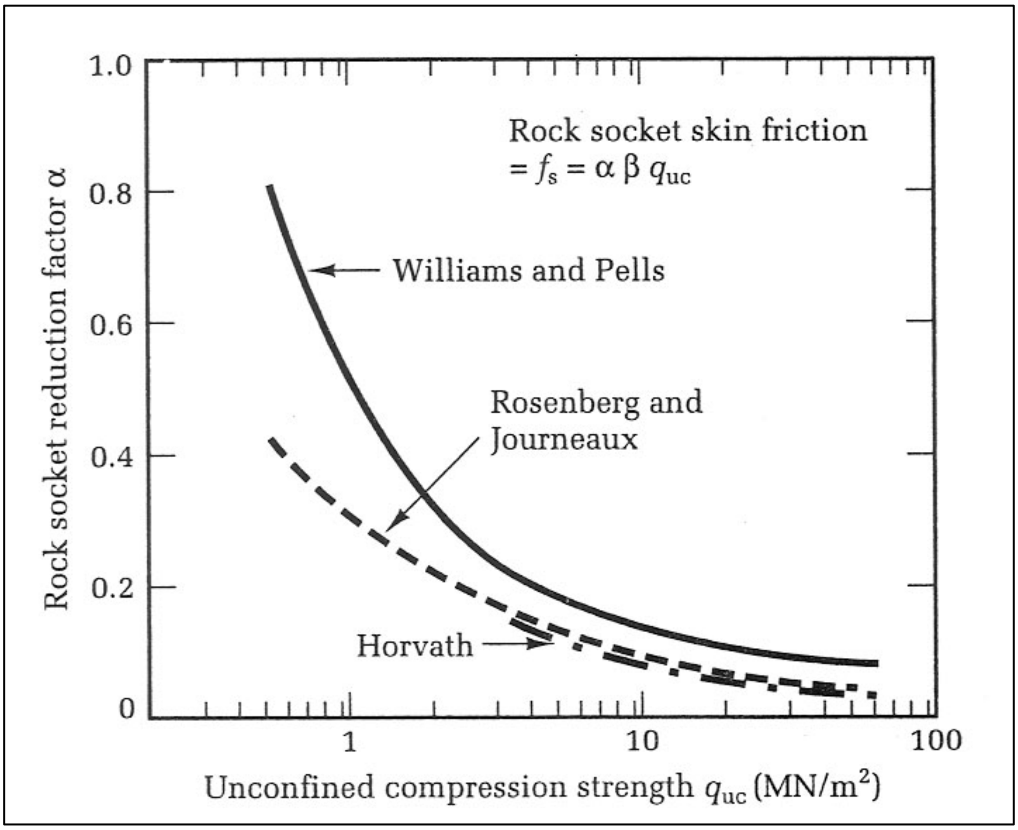

Figure 18: Rock socket reduction factor, α (from Tomlinson 1995).

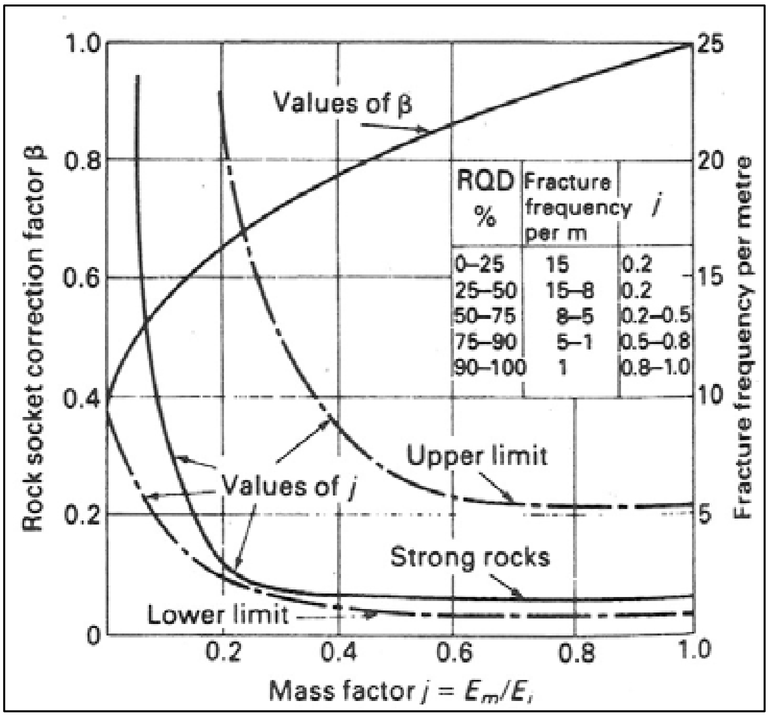

Figure 19: Rock socket reduction factor, β (from Tomlinson 1995).

During borehole exploration, statistics of q_{uc} can be compiled for different weathering grades of bedrock and the rock fracture can be assessed through the Rock Quality Designation, RQD on the rock core recovered or by interpretation of pressuremeter modulus in the rock mass against the elastic modulus of intact rock, which is equivalent to the mass factor, j which is the ratio of elastic modulus of rock mass to that of intact rock. In some cases, at very small cost, the point load test is used to assess and verify rock strength on recovered rock fragments during bored piling after proper calibration with borehole results.

In general, the contribution of base resistance in bored piles should be ignored due to difficulty in proper cleaning of base especially for wet hole construction (with drilling fluid). The contribution of base resistance can only be used if it is constructed in dry holes, if proper inspection of the base can be carried out, or if base grouting is implemented. Tan et al. (1998) reported low values of mobilised base resistance for bored piles in tropical residual soils where K_{bu} values of between 7 and 10 were obtained. K_{bu} is the ultimate base resistance factor for the semi-empirical correlations of base resistance with N-values from Standard Penetration Tests (SPT-N values) given in the equation below:

$$\text{Ultimate base resistance, } f_{bu} = K_{bu} \times \text{SPT-N (in kPa)}$$

The relatively low K_{bu} values are most probably due to the soft toe effect which is very much dependent on workmanship and pile geometry. This even more pronounced in long piles.

In view of the difficulty of proper base cleaning, the authors strongly recommend ignoring the base contribution in bored pile design unless proper base cleaning can be assured and verified.

The construction method is also an important consideration in the design of bored piles in limestone areas. In Malaysia, the two most common methods of forming rock sockets are rock coring with rock cutting bits, and chiselling by mechanical impact. Both methods have their own merits and need skillful operators to form a proper rock socket. In general, the rock coring method will form a smoother, but intact socket surface while chiselling will form relatively rougher sockets, but these rock sockets could be more fractured due to dynamic disturbance to existing discontinuities in the bedrock. Therefore, chiselling is usually not recommended in highly fractured limestone formations to prevent the risk of further fracturing the rock mass.

In addition, construction of bored piles in limestone areas often requires good collaboration between the design engineer and the contractor. This is due to the highly variable ground conditions which require significant input from site personnel and in addition to good geotechnical design, it is recommended that the “observational approach” be adopted for bored pile construction in limestone areas. Such an arrangement will enable any unexpected geological formations and uncertainties to be detected and changes to the design can be made immediately to ensure safe and cost effective design. In order for the successful implementation of the observational approach, the designer should anticipate and identify the potential difficulties and measures that need to be carried out due to “unexpected” geological formation, such as criteria for compensation piles due to large differences in pile length caused by irregular bedrock profiles, etc. which should be in place during the design stage. Therefore, foundation construction in limestone areas is expected to involve significantly more input from the designer during the construction stage as compared to other less complicated geological formations.

In Malaysia, construction method for bored piles in limestone areas is also modified to ensure proper formation of the piles. Figure 20 shows a modified rock coring tool used for bored pile construction in limestone areas. Such a tool enables the casing to penetrate (be reamed) into to the required rock socket length and thus prevents problems such as the collapse of loose soil (slime) surrounding the bored hole normally associated with the construction of rock socketed piles as illustrated in Figure 21.

Figure 20: Modified rock coring tool for bored pile construction in limestone areas.

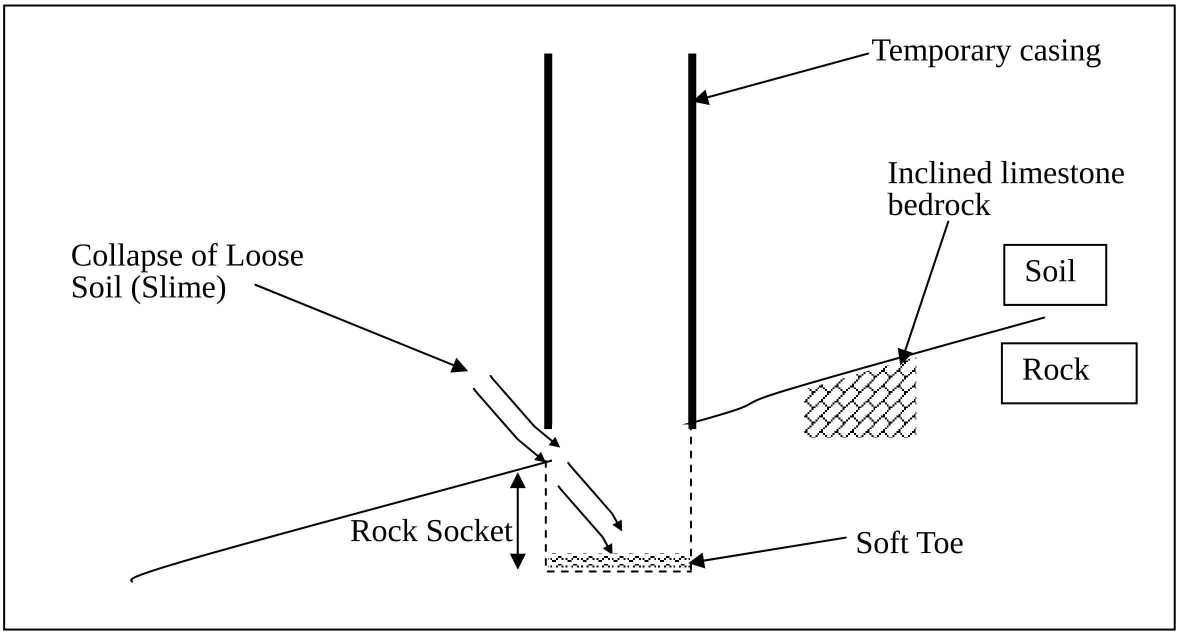

Figure 21: Collapse of loose soil (slime) surrounding the bored hole.

Figure 22 illustrates the performance of the modified coring tool in preventing the above problem at the interface between rock and soil by coring through to the required socket depth together with the casing. Conventional method of construction, where the temporary casing is installed using a vibro-hammer, is unable to penetrate into the rock layer and thus causes situations such as those shown in Figure 21 and also loss of concrete during concreting of the pile.

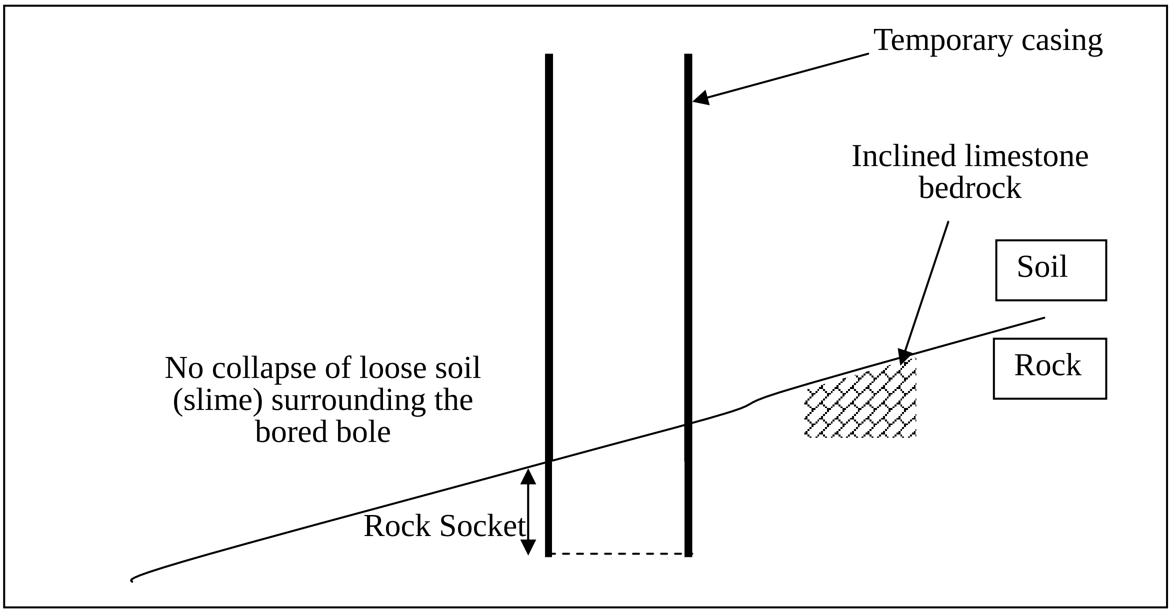

Figure 22: Performance of modified coring tool.

Micropiles

Micropiles in limestone areas are usually designed as rock socketed piles in the limestone bedrock to carry either compression load or tension load. All micropiles are designed to transfer load through the shaft friction, and end bearing at the pile tip is generally negligible due to its small base area. In Malaysia, the design of micropiles is usually based on British Standards such as BS449, BS8081, BS8110 and BS8004 as there is no specific design standard for micropiles. References are also made to other publications such as the Federal Highway Administration, FHWA manual titled "Micropile Design and Construction Guidelines" (FHWA 2000), Bruce et al. (1997) and Juran et al. (1999).

The design rock socket friction can be estimated following the procedures outlined above for bored piles. Alternatively, preliminary estimates can also be made with reference to Table 24, BS8081: 1989 where summaries of rock/grout bond values which have been employed in practice for ground anchors are presented.

In this paper, only specific design aspects related to micropiles in limestone area are discussed. For general design aspects of micropiles, reference can be made to publications cited earlier and also by Gue & Liew (1998) for Malaysian practice.

One aspect of micropile design which is often overlooked is the strain compatibility between the unconfined grout and the reinforcements. In view of the relatively high design axial stress (50% of the yield stress of the reinforcement) usually adopted for the reinforcement, the primary load carrying element in micropiles is the reinforcement instead of grout. In the load transfer stratum, the grout in the annulus between the reinforcement and founding stratum, as a bonding medium, plays an important role in transferring axial load from the reinforcement to the founding stratum. Therefore, the grout must be in good integrity and be intact to transfer the load effectively. If the grout fails in crushing due to excessive compressive stress before the reinforcement reaches the design axial stress, progressive debonding at the grout/reinforcement interface is then expected, hence increasing the elastic deformation at the debonded pile segment and reducing the load transfer efficiency at the grout/soil interface. This is particularly critical for micropiles with bar reinforcement under compressive load as the bar reinforcement will buckle due to insufficient confinement by the crushed grout.

Although the compressive strain limit for concrete is well recognised to range from 2.0 x 10^{-3} to 3.5 x 10^{-3} (e.g. AASHTO section 8.16.2.3 limits the maximum usable concrete compression strain to 3.0 x 10^{-3} and BS8110 adopts a failure strain of 3.5 x 10^{-3}), it is believed that yielding of grout at lower compressive strain may occur. Two failure mechanisms can be expected if the strain of the reinforcement reaches the yielding strain limit of the grout. First is the crushing of grout body under excessive compression. Second is the yielding at reinforcement/grout interface. Gue & Liew (1998) are of the opinion that the second failure mechanism will be likely to happen, because the adhesion of most normal material is always lower than its cohesion, which is governed by its grout strength. Once the overstressing occurs, the yielding of the grout/reinforcement interface will propagate to a deeper depth until the stress level in the grout under lateral confinement drops below the limit. The yielding of the interface is expected to be less significant for micropiles socketed into sound rock. This is because the confinement provided by the sound rock and the axial strain in the micropile attenuates very rapidly with depth at the rock socket. The design implications of interface yielding are elastic shortening, reduction of effective composite sections and unsatisfactory load transfer at the yielding portion of the pile to the ground. In the design of friction piles in soil, care has to be taken to minimise such yielding. Similar concepts are applicable to tension piles. Therefore, it is recommended to limit the strain to prevent yielding of grout. For preliminary design purposes, a strain limit of 1.0 x 10^{-3} is recommended for unconfined grout. This strain limit corresponds to the yield limit specified in BS8110: Part 1 which is given by the following equation:

$$\varepsilon_{yield} = 2.4 \times 10^{-4} (f_{cu}/\gamma_m)^{0.5}$$

where
f_{cu} = characteristic strength of grout
γ_m = partial safety factor (= 1.5)

Therefore, for typical range of grout strength of 25N/mm^2 to 40N/mm^2, the above equation would give values of ε_{yield} between 1.0 x 10^{-3} to 1.2 x 10^{-3}.

An example calculation of the strain compatibility problem for micropile design is demonstrated below:

<u>Assuming:</u>
Yield strength of steel reinforcement (API), f_y = 552,000 kPa
Young's modulus of steel reinforcement, E_s = 210 x 10^6 kPa
Characteristic strength of grout, f_{cu} = 30,000 kPa

At allowable working stress of steel reinforcement (50% of yield stress), the elastic strain, ε_s, on the reinforcement will be as follows:

$$\varepsilon_s = \sigma_s / E_s = (0.5 \times 552000) / (210 \times 10^6) = 1.314 \times 10^{-3}$$

For strain compatibility, the grout should have the same strain as the reinforcement and the calculated values exceed ε_{yield} of the grout of 1.0 x 10^{-3} and therefore, yielding of the interface is expected.

The solutions to this problem for piles under compression are as follows:

a) Reduce the pile axial stress to an acceptable strain limit of grout by downgrading the pile capacity or increasing the reinforcement.
b) Use grout with higher characteristic strength and stiffness and therefore, a higher yield strain limit.
c) Provide permanent steel casing to confine the grout as higher strength and stiffness are experienced in full confinement of the material. The confined state of the grout inside the casing section also allows the grout to support higher strain values without fracturing (FHWA 2000). Based on FHWA (2000), typical value of Young's modulus for grout, E_{grout} is 23,000 MPa for unconfined grout and increases to 31,000 MPa for grout confined in a cased length. Such confined grout would be able to support higher strain values and therefore, strain limit of 1.3×10^{-3} is recommended for confined grout.

For micropile design in limestone area, if empty cavity or very soft slime zone is encountered, the buckling load should be considered for necessary downgrading of pile capacity in compression. The Euler formula shown below can be used to calculate the buckling load depending on the end constraints:

$$P_{cr} = \pi^2 E_p I_p / (KL)^2$$

where

P_{cr} = Buckling load (kN)
E_p = Young's modulus of equivalent pile section (kN/m^2)
I_p = Moment of inertia of equivalent pile section (m^4)
L = Length of pile column without lateral support (m)
K = 1.0 for pinned ends
0.25 for fixed ends (for the cases of cavity or slime zone – Cases A and B)
0.7 for one fixed end and one pinned end (for the case of soft clay – Case C)

Figure 23 shows the possible end constraints for buckling piles in different cases.

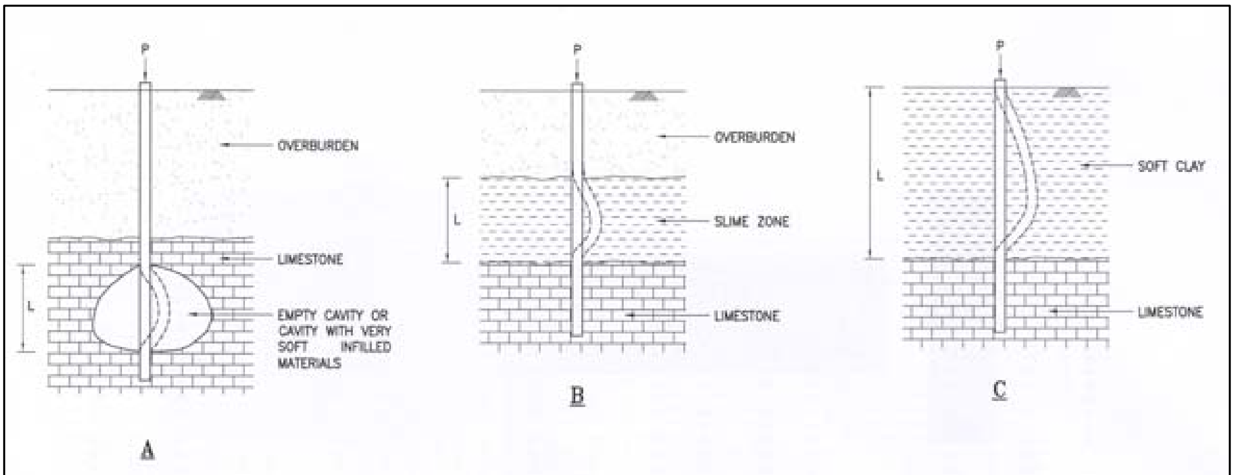

Figure 23: Buckling modes of micropiles (from Gue & Liew 1998).

Similar to other pile foundations, the success of micropiles in limestone area also relies on the quality of installation and therefore some construction control guidelines to ensure successful pile installation are given as follows (Gue & Liew 1998):

a) As it is very difficult to determine the rock conditions for every pile, visual inspection of the rock chipping by experienced supervising personnel is useful in determining the degree of weathering, indicative rock strength, rock mass structures and/or karst features. Records of the socket penetration rate, calibrated to the borehole information and the hydraulic pressure applied on the drill shafts can provide indication of rock quality. Changes of water level or stabilising fluid may indicate the existence of cavities, solution channels and permeable layers where excessive grout loss is anticipated. Change of hydraulic pressure or a sudden drop of drill shaft may also indicate karst features.

d) Measures should be taken to avoid drillhole collapse by means of temporary protection casing and/or stabilising fluid.
e) Grouting should be carried out immediately after cleaning the drillhole by flushing the drillhole with clean water.
d) Permanent casing can be used to minimise excessive grout loss. Alternatively, the use of rapid hardening grout or compaction grout to seal the flow channel can be considered.
e) Proper connection ensuring both ends of the pipes in full contact for coupler and threaded joints and sufficient lapping of reinforcement bars is important to ensure efficient load transfer between the reinforcement. At coupling or reinforcement lapping, it is recommended to stagger the coupling or lapping to avoid weak sections.
f) Centralisers of reinforcements are important elements to ensure adequate grout cover for the bonding of interfaces.
g) Excessive welding on high yield steel reinforcement should be avoided as heat can alter the chemical and physical properties of the material.
h) Grease or coating on reinforcement should be removed to ensure good bonding. However, cleaning of the debonding material at the inner surface of the pipes is very difficult.
i) Provision of holes should be allowed at the tip of API pipe to facilitate grouting between the drillhole and the API pipe.

SUMMARY

The design and construction of foundations in limestone areas poses tremendous challenges to geotechnical engineers due to the highly irregular karstic features of limestone areas. Understanding of the potential difficulties arising from these karstic features is essential in order to provide safe and cost effective foundation solutions.

Some typical piling problems associated with limestone formations were presented and design and construction recommendations for driven piles, jacked-in piles, bored piles and micropiles were also discussed. The design and construction of foundations in limestone areas requires careful planning from the design stage up to the construction stage where continuous input from the construction team and the design team is very important to ensure successful construction and satisfactory performance of foundations in limestone areas.

REFERENCES

Bjerrum, L. (1957). "Norwegian experiences with steel piles to rock". *Geotechnique*, Vol. 7, pp. 73-96.

Bruce, D.A., DiMillio, A.F. & Juran, I. (1997). "Micropiles: the state of practice. Part 1: Characteristics, definitions and classifications". *Ground Improvement*, 1, pp. 25-35.

FHWA. (2000). *Micropile design and construction guidelines*. Federal Highway Administration, US Department of Transportation, USA.

Gue, S.S. & Liew, S.S. (1998). "Design, installation and performance of micropiles in Kuala Lumpur area, Malaysia". Proc. 13th SEAGC, Taipei, Taiwan, ROC, pp. 503-509.

Gue, S.S. (1999). "Foundations in limestone areas of peninsular Malaysia". Civil and Environmental Engineering Conference (C&EEC) New Frontiers & Challenges, Bangkok, Thailand.

Horvath, R.G. (1978). "Field load test data on concrete to rock bond strength". University of Toronto, Publication No. 78-07.

Jakucs, L.D. (1977). *Morphogenetics of karst regions*. Translated by Balkay, Adam Hilger, Bristol, pp. 283.

Jennings, J.N. (1982). *Principles and problems in reconstructing karst history*. Helicite 20(2), pp. 37-51.

Juran, I., Bruce, D.A., DiMillio, A. & Benslimane, A. (1999). "Micropiles: the state of practice. Part II: design of single micropiles and groups and networks of micropiles". *Ground Improvement*, 3, pp. 89-110.

Neoh, C.A. (1998). "Design & construction of pile foundations in limestone formation". *Journal – Institution of Engineers, Malaysia*. Vol. 59, No. 1, pp. 23 – 38.

Poulos, H.G. (2003). "The significance of ground characterisation for foundation deformation prediction". Chin Fung Kee Lecture, *Geotechnical Engineering*, Eds. Ho & Li, Swets & Zeitlinger, Lisse.

Rosenberg, P. & Journeaux, N.L. (1976). "Friction and end bearing tests on bedrock for high capacity socket design". *Canadian Geotechnical Journal*, 13, pp. 324-333.

Tan, B.K. & Ch'ng, S.C. (1986). "Weak soil above limestone bedrock". *Newsletter of the Geological Society of Malaysia*, No. 2, Vol. 12.

Tan, Y.C., Chen, C.S. & Liew, S.S. (1998). "Load transfer behaviour of cast-in-place bored piles in tropical residual soils". Proc. 13th SEAGC, Taipei, Taiwan, ROC, pp. 563-571.

Tomlinson, M.J. (1995). *Foundation design and construction*. 6th edn. Longman.