SOME RECENT DEVELOPMENTS IN DEEP FOUNDATION PRACTICE IN SOUTHEAST ASIA

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SUMMARY

In recent years active developments have taken place in Southeast Asia in various aspects of deep foundations. Some of these developments are reviewed in this Paper. These include techniques of behaviourial analysis such as Chin's Plot and Balasubramaniam's Pull-out Test. Also included are recent advances in small size piles, barrette piles, large bored piles and caisson foundations. The performance of and the experience gained in some of these foundations are highlighted.

INTRODUCTION

Southeast Asia is a strategic region which traditionally has been subjected to the influences of both the East and the West. This is true in many areas of human activities and engineering is certainly one of them. In such an environment new developments tend to thrive. Another factor which promotes new developments is the role of the Southeast Asian Geotechnical Society. This society is very active in the dissemination of new knowledge and exchange of experiences among the member countries, especially Singapore, Malaysia, Thailand, Hong Kong and Taiwan. New developments in which the above factors play an important part are in the field of barrette piles, diaphragm walls and large bored piles.

On the other hand there are also many new developments which are indigenous to Southeast Asia in the sense that they are developed locally. These include hand-dug caissons, the IFP Penetrometer, small size piles, Chin's Plot and Balasubramaniam's Pull-out Test. Chin's Plot provides a useful means of estimating the ultimate bearing capacity of a pile from the results of a routine load test which is not carried out to failure. Equally useful is Balasubramaniam's Pull-out Test in which the long term negative friction on a pile can be deduced from a short term pull-out test.

All the above new developments will be reviewed briefly in this Paper.

CHIN'S PLOT

In pile foundation practice, it is often necessary to verify the ultimate bearing capacity of individual piles on site by carrying out load tests to failure. However, apart from the high cost of ultimate load tests, it is also impractical to perform such tests in soft ground. Besides, an ultimate load test requires an additional pile to be constructed specifically for load testing and this pile does not form part of the permanent works. In view of the above, it will be useful if the results from a routine load test not carried out to failure, e.g. loaded to only 1.5 times the working load, can be extrapolated to estimate the ultimate bearing capacity.

A technique for such a purpose has been developed by Chin (1970, 1972) based on an experimental study of the shear-deformation characteristics of soils and the relationship between load (P) and deflection (Δ) of piles. It was found that in many cases a plot of Δ/P against P produces a straight line and the inverse slope of the line is equal to the ultimate bearing capacity, see. Fig. 1. Chin's Method predicts the ultimate bearing capacity well where the P-Δ relationship is hyperbolic and this is a basic assumption of the method.

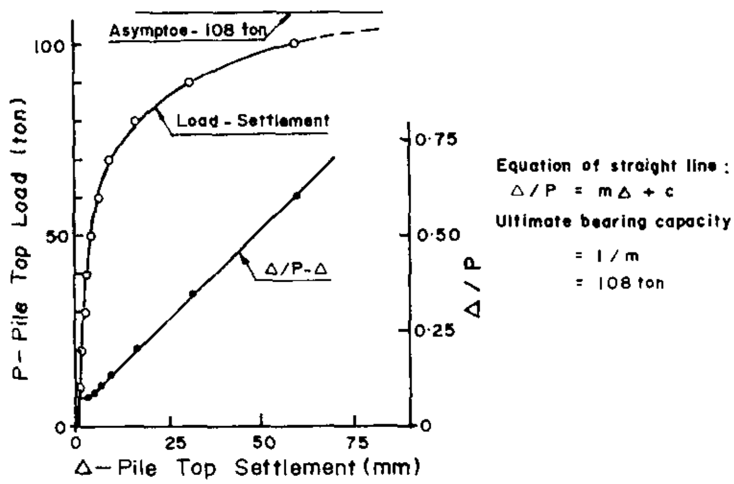

Fig. 1 Chin's Plot

Cheney et al. (1991) have considered similar plots in assessing the stability of tall structures on compressible foundations. Chin's plot in its idealised form contains an underlying assumption of a hyperbolic relationship between load and deflection resulting in a simple straight line relationship being obtained for the plot. In more complex situations successive straight lines may be obtained. This should not result in a fundamental objection to the plot as it is not inconceivable that the approach to failure may involve the development of successive hyperbolas consistent with successive mechanisms. Even though there may be room for more fundamental research into Chin's plot, the practising engineer has been provided with a simple tool which has been found by many researchers such as Fleming et al. (1985) and Hanna (1987) to provide reasonable estimates of ultimate capacities of piles as well as anchors.

BALASUBRAMANIAM'S PULL-OUT TEST

The determination of negative friction on a pile is not a simple task since this requires measurement of axial forces at various levels along the pile shaft. Furthermore, this measurement has to be made on a long term basis in view of the fact that full negative friction takes a long time to develop, usually extending over many months.

Balasubramaniam's group at AIT (Phamvan et al., 1990) have conducted full scale instrumented pile tests involving pile pull-out and long term measurements of negative friction in the Bangkok Clay. They established that for the driven piles the values of negative friction, acting on the pile shaft in the long term, may be deduced from parameters obtained in short term pull-out tests. The relevant part of the results is reproduced in Fig. 2. Although it has generally been assumed that immediate pull-out tests can provide values of undrained skin friction, the AIT group has found a relationship between pull-out test results and long term negative friction values within the soft clay layer. This is a useful finding because pull-out tests are much easier and faster to perform compared to determination of negative friction in the field.

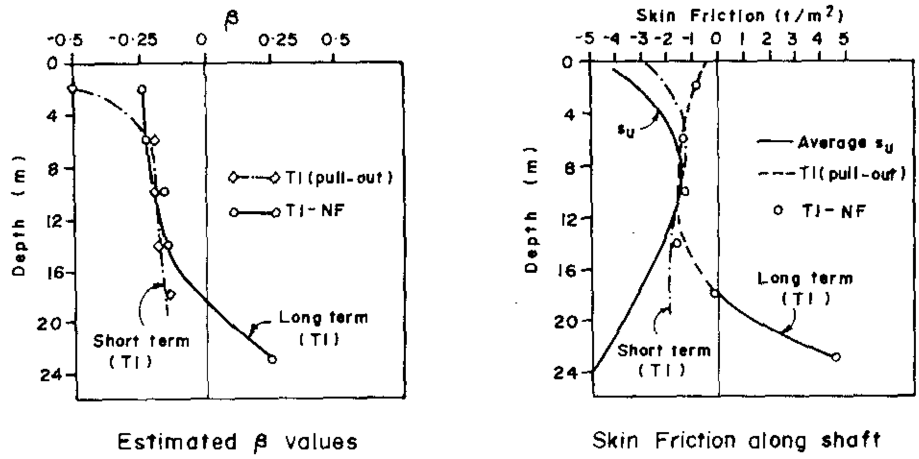

Fig. 2 Pull-out Test and Negative Friction (after Phamvan et al, 1990)

If in addition, pull-out tests on small diameter piles can be related to pull-out tests on larger piles, then a simple pull-out test of a small pile at a given site will provide us with site relevant values of negative friction which should be more realistic than estimates obtained indirectly from shear strength results. Further development of the procedure should extend its application.

SMALL SIZE PILES

In developing countries, for smaller structures on poor ground, the search continues for an appropriate foundation system. Hard woods, treated timber and other types of wood used below water table levels, were the traditional choices for piles in timber growing countries. Increasingly timber is becoming a scarce product due to widespread deforestation and subsequent forest conservation programs. The development is thus in the direction of small concrete piles because of the relatively higher cost of steel. In some parts a concrete pile as small as 82x82mm has been developed but had great difficulty in winning acceptance because of varied problems. All the problems experienced by larger piles, such as in quality control in manufacture, handling, driving and installation, exist, and in fact are accentuated for the small pile.

Some guidelines have to be set up for practising engineers as to an acceptable lower limit in size for a preformed pile. From experience of various problems encountered it is felt the limit should be set at 150mm in lateral dimension. The ACI Concrete Piles Manual (1974) recommends that the side dimension of concrete driven piles should not be less than 200mm. Smaller piles may however be safely used in groups where they cannot be used as isolated piles. Besides the question of slenderness, small piles are often extended by box joints which behave largely as pin joints with negligible bending resistance. In ground conditions where piles are subject to lateral movements and buckling the small pile extended by box joints becomes even more vulnerable. Another simple guideline, again derived from observation, is that all preformed piles must be considered as structural members capable of acting as columns subject to a certain amount of bending if they were to perform as piles. Besides having to cope with handling and installation stresses, slender piles are also subject to drifting when embedded into the ground, and therefore a limit should also be set to the length for which they are to be used. In formations susceptible to downdrag it can be easily shown that installed isolated piles smaller than 150mm diameter are unable to sustain the downdrag force even before any working load is introduced. It will be of interest to receive reports from other users of small piles.

Micropiles generally refer to small cast-in-situ piles formed by grouting. In general the piles come in two grades. The higher grade pile is formed by high pressure grouting with tube-a-manchette or other systems capable of delivering grouts at several points and effecting multiple grouting. The lower grade pile is formed by tremie grouting a drilled hole under nominal pressure. The lower grade pile experiences a high failure rate arising from collapse of borehole, arching of grout and poor bonding between pile and soil. In fact in French practice (Bustamante and Doix, 1985) separate design charts are provided for the high pressure grouted and the tremie grouted micropiles.

BARRETTE PILES

There is increasing interest in diaphragm walls in Southeast Asia as development takes place for underground facilities such as rapid transits and basements. In recent years wall elements, especially diaphragm wall elements, have been used as deep foundations in Hongkong, Singapore and Malaysia although such applications are not as common as bored piles. These wall elements are rectangular in cross-section, e.g. 1m x 2.7m, and usually carry substantial working loads of about 500 to 1500 tons per element. The rectangular elements, also known as barrettes, can be regarded as piles and are, in fact, designed in the same manner as other piles. A natural extension of barrette application is to combine two or more rectangular elements in various configurations, such as X or H, see Fig. 3 to support even heavier vertical loads acting together with substantial horizontal forces and bending moments (Xanthakos, 1979; Kienberger, 1975).

Barrette piles are generally more expensive than bored piles but the former are competitive in projects which also involve diaphragm walls since both systems use the same equipment for construction. Compared with a large diameter bored pile, a barrette provides a larger side friction area for the same volume of concrete and can reach much greater depths, well beyond the limits of conventional bored pile machines. Mitchell (1985) reported on an interesting case history involving the use of barrette piles up to 90m depth for supporting the 30 storey Pan Pacific Hotel in difficult karstic limestone in Kuala Lumpur. Barrette piles, which have been used in France for many years, have certain inherent advantages for supporting heavy loads from core walls of highrise buildings since the rectangular elements can be arranged to coincide with the wall locations. If conventional piles are used it would be necessary to provide a very thick massive pile raft.

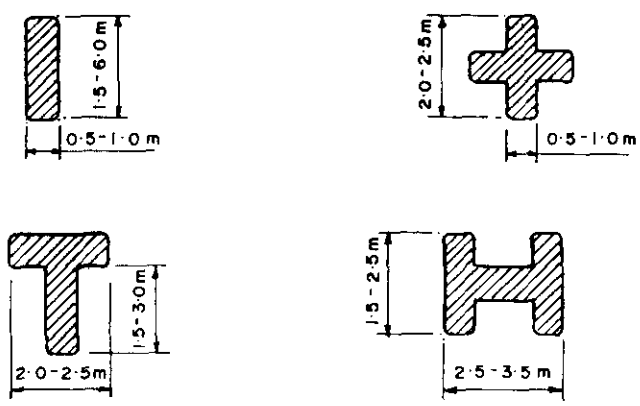

Fig. 3 Various Configurations of Barrette
(after Xanthakos, 1979)

A special case of load-bearing wall is when columns are located directly on a diaphragm wall. Here the attractive feature is that the endbearing and part of the side friction resistance are already available from the diaphragm wall acting as a retaining system. Thus, what is often required is merely to extend the depth of the diaphragm wall locally at the column positions. In practice, the panel arrangement for wall construction is also adjusted such that each column is situated at the centre of a panel. However, from the design standpoint there are some complications. For example, the horizontal deflection of the wall may affect the side friction resistance for carrying vertical loads. In view of the above interaction, the design of such walls needs extra care.

LARGE BORED PILES

Large diameter bored piles continue to be widely used to support heavy loads from bridges and highrise buildings. With advances in machinery it is now feasible to provide piles with diameters up to 2.5m or even larger, depths to 60m and working loads up to 2500 tons per pile. Fig. 4 shows a large project involving bored pile construction.

Analytical methods of pile design have developed rapidly in the last two decades. These are well described by Fleming et al. (1985) and will not be dealt with here. However, two aspects of pile design are worth noting. The first is related to working stress of bored piles. Due to stiff competition arising from the recent worldwide recession, there is a tendency to use higher working stresses on bored piles. It should be pointed out that if a higher characteristic strength of concrete is specified, it does not necessarily mean that the working stress can be increased proportionately since the concrete in bored piles is invariably not compacted, unlike structural concrete. One disadvantage of using high strength concrete in bored piles is the increased risk of shrinkage cracking of the shaft (Institution of Civil Engineers, 1988). There are other compelling reasons why working stresses in bored piles should not exceed certain reasonable limits.

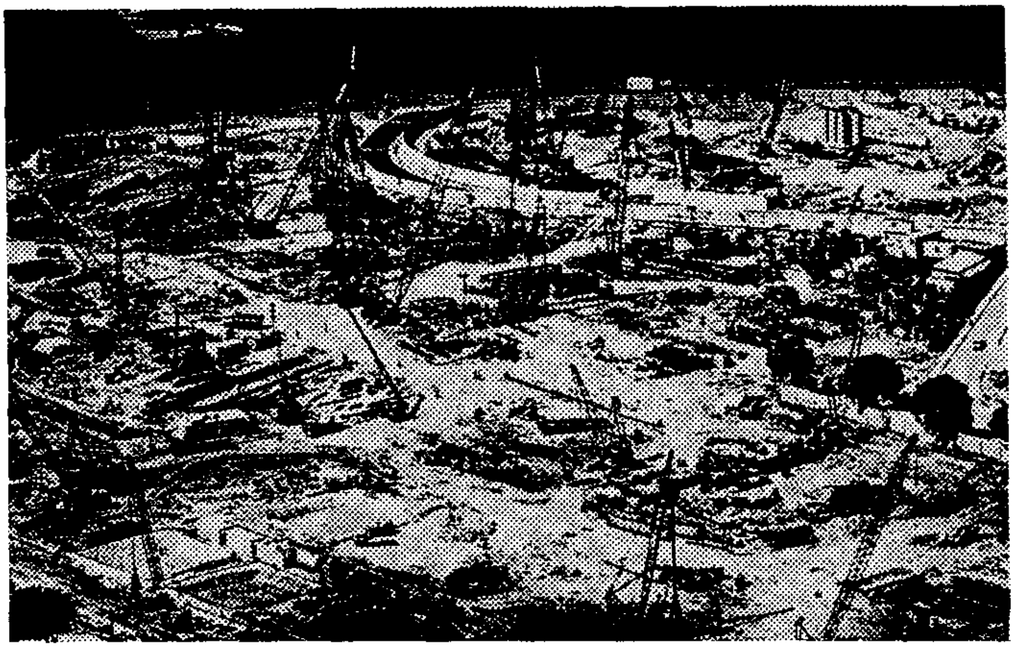

Fig. 4 Bored Piling at Suntec City, Singapore

The second aspect is related to the use of instrumented test piles as a means of verifying pile design, especially in unfamiliar ground. This practice is gaining wider acceptance in the Southeast Asian region. These piles should preferably be load tested to the ultimate state in order to derive site-specific empirical parameters for design. However, full instrumentation for monitoring settlements and axial loads at various levels along the shaft is expensive. Where cost constraint rules out full instrumentation, a useful but very inexpensive compromise is to provide a pair of tell-tale extensometers at the pile toe (base) only. Apart from providing information on elastic compression of the pile shaft and hence detect major structural defects, the extensometers will indicate at what stage of loading will endbearing resistance begin to be mobilized. This approach can also be incorporated into routine load testing of working piles (Mitchell 1985).

In the field of quality control there has been a better appreciation of the problems and defects arising from bored pile construction (CIRIA, 1977). As a result there is a wider application of non-destructive testing of piles both for structural integrity and dynamic load capacity. In dynamic testing, pounders weighing up to 25 tons, see Fig.5, have been used in Singapore (Lee et al., 1990). Useful practical guidelines on both types of test are given in a recent publication by the Institution of Civil Engineers U.K. (1988).

Construction control has traditionally lagged behind advance in analysis. An inherent problem in bored pile construction is the determination of the appropriate founding depth of each pile on site, especially in non-uniform ground. A recent development is the IFP Penetrometer (Chan, 1985) weighing 3 tons, which can be used for in-situ testing of the ground at the bottom of the pile hole as boring progresses, see Fig. 6. Since the IFP reading can be correlated to ultimate unit shaft friction and endbearing or to SPT and CPT data, the on-site determination of founding depth of every pile can be established with greater reliability.

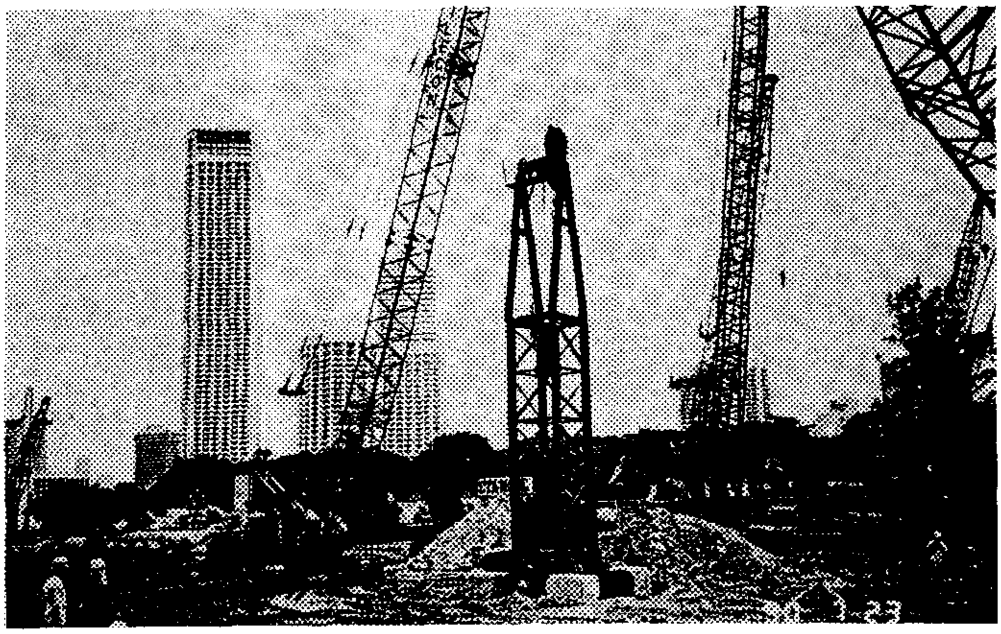

Fig. 5 Dynamic Load Testing Using a 25 Ton Pounder, Suntec City, Singapore

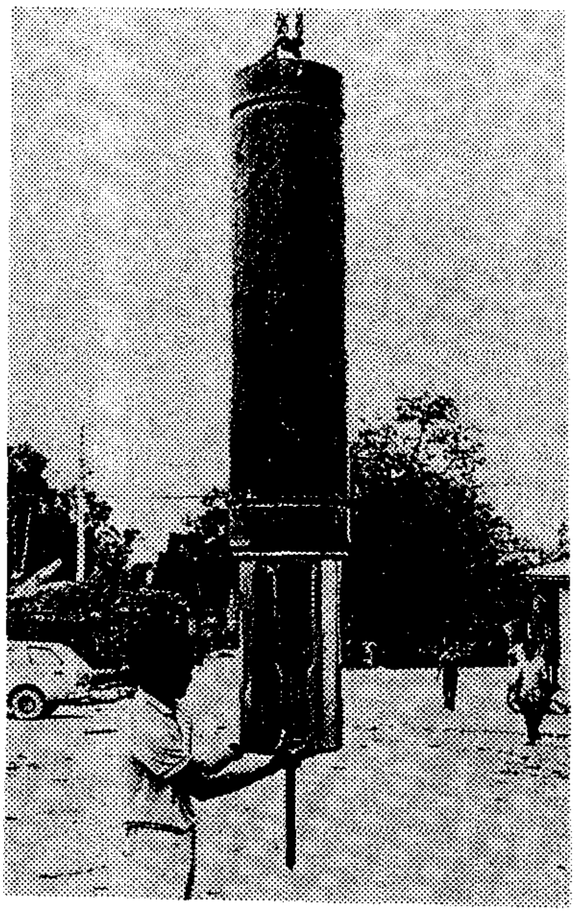

Fig. 6 The IFP Penetrometer

In a related development concerning bored piles, prefounded columns in deep basement construction are being used to a larger extent in this region. These are basement columns in the form of very heavy steel sections. They are invariably supported on large bored piles and are installed at the time of construction of the piles. The provision of prefounded columns permits the commencement of superstructure construction at the time of starting construction of the basement downward from the ground level following the "top-down" method. The other benefit is that each successive basement floor provides a very rigid support to the basement wall, e.g. diaphragm wall, compared with conventional temporary supports such as struts or anchors. As a result smaller movements in the adjacent ground and structures can be achieved. However, prefounded columns are difficult to install, since each column must be inserted accurately before the concrete of the bored pile sets. Apart from the high cost of heavy steel sections and the much smaller permissible construction tolerances, such installation requires a great deal of skill and organization. In view of the above, there is a need for improvement.

CAISSONS

Caisson foundations, also called pier foundations in the United States, have been used for many decades (Terzaghi and Peck, 1967) and continue to be used today with certain modifications reflecting advances in technology. Caissons have been used in Hong Kong, Singapore and Malaysia. Modern day caissons which are constructed following the open shaft method have a wide range of size.

Small diameter caissons are excavated by hand, hence the name hand-dug caissons. Typical sizes vary from 1.25 to 2.0m diameter with corresponding working loads of 300 to 2000 tons. These caissons are prescribed where the terrain is too steep and therefore inaccessible to heavy machinery, or where obstructions make drilling impractical. The use of hand-dug caissons is an established practice in Hong Kong (Brand, 1982), and more recently this type of foundation has been introduced in Malaysia.

In Southeast Asia, large diameter caissons have been used in Singapore and Hongkong. In Singapore, over the last 20 years such caissons have been used as deep foundations for supporting four very tall buildings of 50 to 66 storeys, see Fig. 7. A summary of the various features and

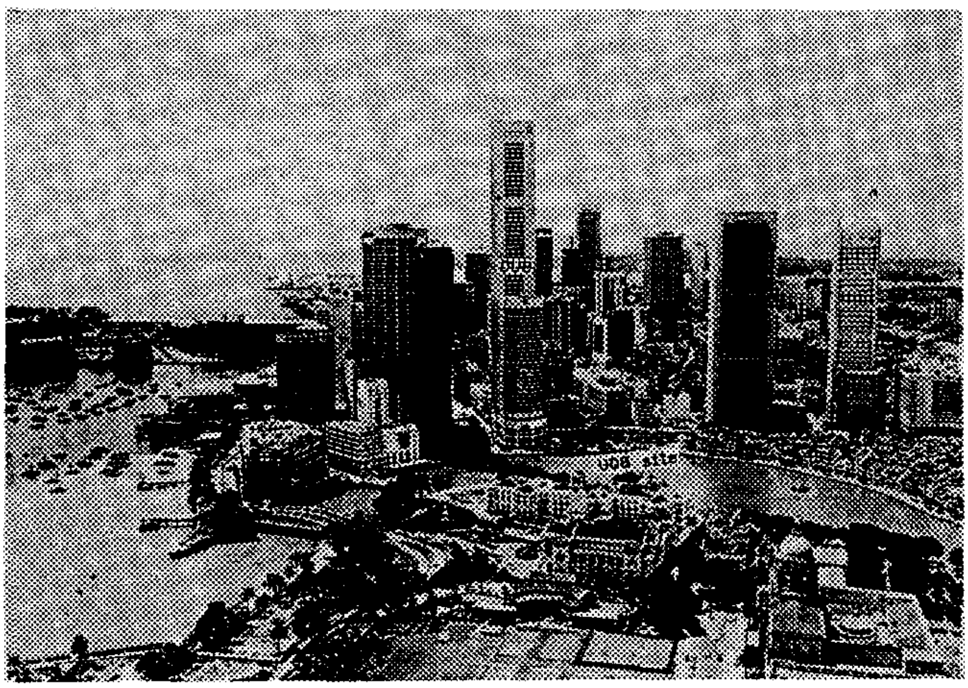

Fig. 7 Buildings Supported on Large Caissons, Singapore

performance of these caisson foundations is presented in Table 1. These caissons, with shaft diameters of 4.7 to 8.0m, carry very heavy working loads indeed, ranging from 13,000 to 38,000 tons per caisson. In view of the magnitude of the loads and the need to limit settlement, these caissons are founded in rock except in the case of the United Overseas Bank (UOB) building. The founding depth varies from 40 to 100m and depends largely on the depth and nature of bedrock.

Large caissons are used where ground conditions preclude the use of other foundation systems, as for example where there are difficulties in construction of bored piles in bouldery deposits or where the depth requirement of bored piles exceeds machine capability. The control of settlement is also a major consideration especially if the loads are concentrated in a small area.

The design philosophy varies. For the 64 storey Overseas Union Bank (OUB) building, which is supported on only 7 caissons with enlarged bases at approximately 100m depth, the caissons are designed as entirely endbearing in the siltstone and sandstone bedrock, with side friction in the overlying Bouldery Clay neglected (Kurzeme et al. 1990). It is interesting to compare the above building with the 66 storey UOB building, now under construction and located in similar ground just across the street. The UOB building is supported on 12 straight shafted caissons which are founded

Table 1. Large Caisson Foundations in Singapore

Building name	DBS	Treasury	OUB	UOB
Building height in No. of storey	50	52	64	66
Ground conditions (stratigraphy)	Marine clay Stiff clay Dense sand Mudstone bedrock	Weathered mudstone Sandstone bedrock	Fill Bouldery clay Siltstone/ sandstone/ bedrock	Marine clay Bouldery clay Siltstone/sand- stone bedrock
Founding stratum	Mudstone bedrock	Sandstone bedrock	Siltstone/ sandstone bedrock	Bouldery clay
Caisson details:-				
Total No.	4	6	7	12
Diameter (m)	7.3	8.0	5.0 - 6.0	4.7 - 6.8
Straight shaft or enlarged base	Straight Shaft	Straight Shaft	Base enlarged 6.0 to 10.5m	Straight Shaft
Depth (m)	40 - 64	50	100	60
Working load per caisson (ton)	20,000	24,000	13,400 - 37,900	8,000 - 16,000
Working stress in shaft (N/mm^2)	4.8	4.8	8.1 - 9.8	4.4 - 4.6
Settlement of building (mm)	16 - 19	not reported	9.5 - 13.0	under construction

in the Bouldery Clay with a maximum length of about 60m. These caissons are designed such that the building loads are carried entirely by side friction with a minimum factor of safety of 1.5. The endbearing resistance is regarded as a reserve contributing towards the overall factor of safety against collapse (Wallace et al. 1990).

The reported settlement performance of caisson foundations is good. In the case of the OUB building, the settlement at the top of caissons at the end of construction was 9.5 to 13.0mm whereas the corresponding settlement of the 50 storey Development Bank of Singapore (DBS) building supported on only 4 caissons was 16 to 19mm (Wong, 1984). These settlements are of the same order of magnitude as the elastic compression of the caisson shafts.

CONCLUDING REMARKS

Southeast Asia is a region where active developments in deep foundation practice are taking place. This Paper has reviewed some of these developments in various aspects of deep foundations. They include not only the design, construction and performance of various types of deep foundation but also advances in techniques of behaviourial analysis.

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