Design and Construction of Foundation of High Rise Building

Livio Locatelli
Technical Department, RODIO S.p.A. - Milan - Italy

Pham Hoang Nhan
Technical Department, RODIO Vietnam Branch - HCMCity - S.R. Vietnam

SYNOPSIS

The soil of HCMCity is firstly described, then some works executed by RODIO company are presented. Bored Piles of various diameters down to 60m have been performed to carry heavy loads of columns of high buildings in HCMC. Results of some loading test are outlined. The use of Diaphragm Wall allows the construction of deeper underground storeys supporting the lateral earth and water pressure. A problem of excavation nearby an existing building has been technically solved by underpinning its shallow foundation. For the future developments of projects in HCMC, the possible introduction of anchorage system is presented as alternative solution to the bracing system, describing its advantages. The use of grouting techniques is exposed to solve the problem of excavation under water table avoiding the costs related to a continuous dewatering system. The proposal to create underground parking is then depicted referring to works executed and in progress in European towns.

PHAÀN TOÙM TAÉT

Ñaàu tieân, chuùng toâi trình baøy sô löôïc veà tình hình ñòa chaát cuûa khu vöïc thaønh phoá Hoà Chí Minh cuøng vôùi moät soá coâng trình maø RODIO ñaõ vaø ñang thi coâng. Caùc coïc khoan nhoài vôùiù caùc loaïi ñöôøng kính khaùc nhau saâu ñeán 60m ñaõ ñöôïc thi coâng ñeå chòu taûi troïng coät raát lôùn cho caùc coâng træình nhaø cao taàng. Vaøi keát quaû thöû taûi ñöôïc trình baøy beân döôùi ñeå minh hoïa chi tieát hôn khaû naêng chòu taûi cuûa coïc. Vieäc öùng duïng caùc keát caáu töôøng chaén ñaát baèng beâtoâng coát theùp ñeå chòu aùp löïc ngang cuûa ñaát vaø nöôùc ngaàm. Caùc vaán ñeà gia coá moùng noâng cuûa caùc coâng trình hieän höõu xung quanh baèng kyõ thuaät khoan coïc choáng trong ñaát ñeå thi coâng caùc haàm saâu cuûa coâng trình môùi.Söï phaùt trieån trong töông lai veà kyõ thuaät xaây döïng caùc keát caáu ngaàm trong ñaát ôû thaønh phoá Hoà Chí Minh. Söï giôùi thieäu heä thoáng neo trong ñaát nhaèm thay theá heä giaèng choáng truyeàn thoáng vaø vieäc so saùnh caùc öu khuyeát dieåm cuûa noù. Vieäc öùng duïng kyõ thuaät phun vöõa coù aùp löïc nhaèm thi coâng taàng haàm döôùiù möïc nöôùc ngaàm vaø giaûm thieåu caùc chi phí veà heä thoáng haï möïc nöôùc ngaàm. Vaø cuoái cuøng laø vieäc moâ taû caùc kyõ thuaät tieân tieán trong thi coâng caùc baõi ñaäu xe ngaàm döôùi loøng ñaát ôû Chaâu AÂu coù theå öùng duïng ôû Vieät Nam.

INTRODUCTION

For the construction of high-rise building, the attention of the designer is usually addressed to the structural elements of the upper part of the building, in order to respect its foreseen, and sometimes complex, architectures. Once the loads (live and dead loads) acting on the base columns of the building are defined, the design of the foundation has to be analyzed by the geotechnical engineer. His duty is to study how to transfer such loads to the soil in order to guarantee an adequate safety to the building itself.

The initial problem is to identify the soil stratigraphy, and from a correct number of laboratory tests ascertain the mechanical characteristics of each layer. The soil-structure interaction is governed by a non-linear behaviour and by the non-uniformity of the soil itself. Conversely, the uncertainties of the analysis of the upper structure are usually limited to unusual geometry, or anomalous loading conditions.

The assessment of a proper foundation solution should be investigated to be cheap, long-lasting but overall, safe, either for settlement and stress of the building structures.

Regarding to foundations and works related to the soil, the present paper intends to touch the following points:

- HCMCity typical stratigraphic profile;
- Specialized foundation techniques such as Bored piles and Diaphragm wall;
- Underpinning as method to maintain safe conditions for existing buildings next to excavation works;
- Anchorage system as support of underground excavation;
- Application of new technologies in problems related to excavation below water table, dewatering, civil constructions, etc.;
- Case histories of underground car parking as applied in European towns.

Every point describing the technology is herewith treated as general outlook. For a better knowledge of each single matter, the study of specific technical literatures is recommended.

1. THE SOIL OF HCMCITY

The stratigraphic sequence of Ho Chi Minh City's soil, laying on the course of Saigon river Delta, is mainly composed of alluviums whose horizontal strata have been deposited during the centuries.

Fig. 1 Typical stratigraphy

The ground level varies from +7.0 ÷ +10.0m ASL (above sea level) in the zone of the Notre Dame Cathedral, and steps down to approx. +2.0m in vicinity of Saigon River. The water level is mainly affected by the seasons (rainy or dry) and by the tides. Its average level is +0.60 ÷ +1.50 m ASL.

The stratigraphic sequence, whose typical section is drawn in Fig. 1, is characterized by the following geological strata:

- FILL Generally composed of bricks of pre-existing foundations;
- CL Inorganic clays of low or medium plasticity, gravely clays, sand clays, silty clays, clean clays
- SC Clayey sands, sand-clay mixtures;
- SM Silty sands, sand-silt mixtures;
- CH Inorganic clays of high plasticity, fat clays.

Tab. I collects an average summary of the geotechnical data of the various layers obtained from the of HCMCity's sites where RODIO company has operated.

The upper layers (until a level of approx. 0.0m ASL are composed of clay and of organic silt (CL1 and SC1) and include possible pre-existent foundations and filling.

Such type of soil is quite impervious to the water seepage and allows the excavation of soil without relevant support. In some zones, the presence of an intrusion of hard clay (CH1) can be also encountered.

The layer SM from 0.0m down to -31.0m ÷ -34.0m is composed in its upper part (approx. 12m) of silty sand (SM1a) with low and medium fineness, while in the deeper part such material is more dense (SM1b). The number of NSPT is quite small, values ranging between 4 and 35. The friction angle can be assumed to vary on 24°÷32° range.

Tab. I - Average Characteristics of HCMCity's soil [1]

Layer	Level from - to	Thickness m	NSPT	γ kN/m^3	c' kPa	ϕ' deg
CH1-CL1	+8.0 ÷ +3.0	2 ÷ 8	4 - 10	15 - 18	20	7 - 9
SC1	+3.0 ÷ -3.0	2 ÷ 5-	3 - 8	17.5 - 19	11	12 - 17
SM1a	-3.0 ÷ -16.0	16	6 - 17	20	10 - 15	24 - 28
SM1b	-16.0 ÷ -34.0	14	15 - 25	20	5 - 10	28 - 32
CH2 - CL2	-34.0 ÷ -50.0	7 ÷ 14	30 - 90	20	50 - 100	15 - 18
SM2	-50.0 ÷ -60.0	2 ÷ 8	30 - 60	20	0 - 2	30 - 35

[1] RODIO is not responsible for the use of the present data for design purposes or applications.

The silty sand layer (SM) is sometimes interrupted by intrusions of clay (CH) of clayey sand (SC) of limited thickness (1÷2m).

From pumping test the permeability range of SM layer can be identified between $5 \cdot 10^{-5}$ m/s to $2 \cdot 10^{-6}$ m/s.

The layer composed of hard clay (CH) is for all the investigated sites from -34 to -50m ASL with lightly variable thickness. This layer is usually properly adopted as base for the piling foundation being its NSPT between 40 ÷ 100.

A further layer of silty clay (CL2) of 4m thickness is sometime encountered just below the hard clay layer.

From this level down to the normal depth reached by the boreholes, the type of soil is again silty sand (SM2), medium to fine sand dense to very dense with alternation of thin clay vein, NSPT ranging from 32 to 60.

The soil of HCMCity is therefore quite uniformly distributed with horizontal layers, whose characteristics allows the use of shallow foundation for building up to 4-5 floors, while for higher buildings the shallow foundation method does not provide a safe base for the footings and it is necessary to transfer the vertical loads to deeper layers.

Until '93 the use of driven piles allowed to put the bases on the deep hard layers, since such layers provide a proper base for the deepest piles that requires higher resistance load.

Due to the vibration of their installation system, the driven piles have been abandoned in HCM urban area.

Therefore such technology has been substituted by deep foundation composed by bored piles or barrettes.

2. SPECIALIZED FOUNDATION WORKS

RODIO is specialized in many fields regarding the soil and foundation works around the world.

Two main activities are related to the bored piles and diaphragm wall techniques that Rodio already implemented in Vietnam from 1993.

As above expressed the soil of HCMCity is suitable to support the load of high buildings only by using deep foundation systems.

2.1 Bored piles

Bored cast in situ piles are formed by boring and subsequently filling the hole with concrete. The normal diameter range is between 250mm and 1500mm, distinguishing in small diameter for $\phi<600$mm and large diameter for $\phi>600$mm.

Various depths can be adopted depending on the soil characteristics and the applied load. Piles of 60m depth can be considered as deepest. Bored pile's construction sequence is shown in Fig. 2.

On a crawler crane a rotary equipment is mounted which rotates a telescopic rod fitted with a drilling tool (auger or bucket). The penetration of tools into ground is caused by a downward thrust imparted to the telescopic rod by two hydraulic jacks. The verticality of the pile is assured by the telescopic rods that counteracts possible deviations of the drilling tool.

The boring tool is fitted with reaming blades that form an interstice between the hole and the tool.

In order to guarantee the stability of the borehole and prevent inflow of soil, the boring takes place through bentonite slurry suspension.

The formation by the slurry of an impervious film (filter cake) on the walls prevents the loss of the fluid. The bentonite mud gives a hydrostatic force against the vertical walls in excess of the ground water pressure, and increases the strength of the subsoil by the intrusion of the mud and subsequent gelling (thixotropy).

Quality of slurry fluid during excavation is constantly monitored by in-situ test; in order to guarantee that the slurry characteristics are within the limit ranges. For instance the mud density and viscosity control avoids the risks of collapse of the hole, while the sand contents check before concreting assure that the concrete will not be mixed with the mud.

On completion of desanding operations, the installation of the cages can be proceeded. Then the concreting operation can start only if percentage of sand in bentonite slurry is less than 2%.

Concrete is poured through a tremie pipe, equipped with a funnel on its upper end, keeping the edge of the tremie pipe always immerse into the concrete in order to avoid contamination.

By this method, named "Contractor", the concrete fills the hole from bottom upwards so that the top layer of concrete polluted by the mud can be removed when it reaches ground level (or foreseen cut off level).

Fig. 2 Bored piles - Construction sequence

The design of a bored piles foundation is governed by rules that have demonstrated their reliability on the basis of a great number of cases where they been successfully applied.

The pile resistance is composed of the contribution of the end base capacity and the shaft resistance (ultimate skin friction):

$$Q_p = Q_b + Q_s \qquad (1)$$

For both the addends it is necessary to distinguish if the pile is in cohesive or uncoherent soil.

The classic formula for calculating the resistance of piles in cohesionless soils is:

$$Q_p = N_q \sigma'_{vo} A_b + \frac{1}{2} K_S \sigma'_{vo} A_S \tan\delta \qquad (2)$$

where:

N_q is the bearing capacity factor (function of the friction angle of soil);

σ'_{vo} is the effective overburden pressure at pile base level;

A_b is the base area of the pile;

K_s is the coefficient of the horizontal soil stress that for bored piles varies from 0.7 to 1.0;

A_s is the shaft lateral area;

δ is the angle of friction between soil and pile.

Fig. 3 Bored piles site (Ocean Place)

In case of cohesive soil the two addends are:

$$Q_b = N_c c_b A_b \quad (3)$$

and

$$Q_S = \alpha \bar{c}_u A_S \quad (4)$$

where:

N_c is the bearing capacity factor ;

c_b is the undisturbed undrained cohesion at the pile toe;

A_b is the base area of the pile;

α is an adhesion factor varying beetween 0.2 to 1.0;

$\bar{c}_u$ is the average undisturbed undrained cohesion of the soil sourrounding the pile shaft;

A_s is the shaft lateral area.

The base resistance is highly effected by the overburden pressure in case of cohesionless soil, while, in case of pile's toe embedded in cohesive layer, the end bearing capacity is independent from the depth.

Care shall be taken in choosing the correct parameters in the above equations, since they are effecting the estimation of the ultimate capacity consistently

2.1.1. Loading tests

In order to verify the design's assumptions, a preliminary loading test is always carried out before starting with the production of the working piles. By this way the assessment of the propriety (or not) of the design allows to revise it for the subsequent piles.

Then further loading test on working piles are usually performed to check if their construction accomplish the expected requirements.

Since piles are usually built to support compressive loads transfering from the own weight of future buildings, the normal test is the compression test.

Different types of test (tension test or lateral test) are sometimes executed in case of piles that reacts against an uplift pressure of structures below water table, or in case of isolated piles submitted to horizontal loads produced by wind or rotating machinery.

Usually preliminary load test are done up to failure, while maximum test load on working piles is twice their working load.

For applying such relevant compressive loads (from 1000 to 2000 kN or more), it is necessary to have an adequate counterweights against the test piles.

In general, two system of reaction can be used: kentledge system or reaction piles system.

Fig. 4 and Fig. 5 show respectively the two kinds of systems.

Since the kentledge system weight should be 20% greater than the applied load, for relevant test loads the required number of concrete blocks and the dimension of the kentledge itself becomes unsuitable.

In such case the only solution is to use a reaction frame that reacts against other bored piles or eventually vertical ground anchors.

Fig. 4 Pile loading test by kentledge (Ocean Place)

For the preparation of the test, the pile head has to be constructed with reinforced concrete up to the necessary elevation and capped appropriately to produce a bearing surface perpendicular to the axis of the pile.

The test load is applied by means of an hydraulic jack (or more) which reacts against the reaction system.

The pile head settlement can be measured by dial gauges or displacement transducers (LVDT). The reading of load is obtained by the pressure gauge on the hydraulic circuit of the jack.

Extensometers and strain gauges are the two typical sensors that can be installed at various levels along the pile length. They allow to determine the global behaviour of the pile discriminating the end point bearing capacity from the contribution of the friction of the various soil layers along the pile's shaft.

The extensometer provides the displacement (in mm) between the pile head and a fixed point of the pile's shaft. From its readings it is possible to define the elastic (or plastic) shortening of the pile.

The strain gauge provides directly the readings of "strain" at the location where it is installed.

All the information is extremely useful to define how the pile toe and the shaft behave during the loading process.

Two loading tests are here presented with regards to instrumented piles in HCMC. Fig. 6 shows the results of the loading test carried out for Park Hyatt Project.

Fig. 5 Pile loading test by reaction system (International Business Center)

The trial pile is 835mm diameter and 40.5m deep. Its toe is embedded 7 m in the hard clay (CH). Three extensometer rods have been installed at -6.0m; -20.0m and -39.7m. Three strain gauges have been fixed on four levels of the pile in order to measure the strains inside the pile.

The loading procedure presents three cycles, respectively at 100%, 200% of the working load (equal to 8000 kN) and up to failure. The load is applied using a kentledge of approx 10000 kN.

At each stage, the readings of load, time, settlement, extensometers, and strain gauges readings are recorded.

The graphs of Fig. 6 represent the diagrams of load vs. settlement, settlement and load vs. time, and load transfer along the pile depth for various load steps. Besides, a sketch of the recorded layers sequence is also shown with the location of the sensors inside the pile.

From those diagrams, the behaviour of the pile is well identified. At 3800 kN the pile has an elastic behavior with limited settlements and satisfactory rebound, also confirmed by the extensometers readings. Meanwhile at 7600 kN the settlement increases with a plastic trend. Its ultimate load can be estimated not to be greater than 7600 kN. In fact in the third cycle up to failure, the load of approx. 8500 kN was reached, but the settlement increased ceaselessly.

Tab. IIa - Load test results in Park Hyatt

Pile diameter		835 mm
Pile toe depth (from G.L. ±0.00)		40.5 m
1ST CYCLE	Unit	
Max load	kN	3800
Max. settlement	mm	2.87
Residual settlement	mm	0.35
2ND CYCLE		
Max. load	kN	7600
Max settlement	mm	15.79
Residual settlement	mm	8.99
Max Load at 3rd cycle	kN	8577

Tab. IIb - Load transfer at working stage

Depth	Unit	
m	kN	%
From 1.0 To 6.0	1033.6	27.2
From 6.0 To 20.0	250.8	6.6
From 20.0 To 33.7	570	15
From 33.7 To 39.7	1812.6	47.7
Pile base	133.0	3.5

Tab. IIa-IIb contain the summary results both as recorded settlements, and load transfer along the pile shaft.

Since the actual bearing capacity (skin friction) of the hard clay layer is well defined from the above results, the pile design has been verified by increasing of approx. 3.0m its embeddment.

As per Park Hyatt, the results of the loading test carried out in Ocean Place are summarized in Fig. 7, Tab.IIIa and IIIb.

The trial pile is 1200mm diameter and 61.5m deep, its toe based on the second silty sand layer (SM2). Three extensometer rods have been installed at -14.8m; -32.8m and -59.2m. Three strain gauges have been fixed on six levels of the pile in order to measure the strains inside the pile.

The loading procedure foresaw two cycles, at 100% and 200% of the working load (equal to 8000 kN). The load is applied by using a kentledge of approx. 20000 kN.

From the diagrams of Fig. 7, it is confirmed that at 8000 kN the pile has an elastic behaviour with limited settlements.

In the second cycle the applied load was 16800 kN, in order to take into account the undesired possible friction of the upper part of pile above cut-off level. During the 24 hours of maintained load the settlement was stable with a modest plastic trend.

Tab. IIIa - Load test results in Ocean Place

Pile diameter		1200 mm
Pile toe depth (from G.L. ±0.00)		61.5 m
1ST CYCLE	Unit	
Max load	kN	8000
Max. settlement	mm	6.12
Residual settlement	mm	1.06
2ND CYCLE		
Max. load	kN	16800
Max settlement	mm	24.65
Residual settlement	mm	10.54

Tab. IIIb - Load transfer at working stage

Depth	Unit	
m	kN	%
From 1.0 To 14.8	1033.0	19.18
From 14.8 To 26.8	523	6.65
From 26.8 To 37.8	2109.2	26.36
From 37.8 To 47.8	2805.9	35.07
From 47.8 To 61.5	643.68	8.05
Pile base	375.14	4.69

Fig. 6 Park Hyatt - Loading test results - 835 mm dia pile 40.5 m deep

Fig. 7 Ocean Place - Loading test results - 1200mm dia pile 61.5m deep

From such results, and on the basis of many other tests carried out on piles in HCMC's soil, some practical conclusions can be extracted.
For various pile diameters, the applicable working load range is shown in Tab. IV

Tab. IV - Nominated working load for piles of various diameter in HCMCity soil.

Pile diameter mm	Nominated Working Load (tonne)
600	110 ÷ 180
700	160 ÷ 250
800	230 ÷ 350
900	330 ÷ 400
1000	380 ÷ 500
1200	500 ÷ max 800 for piles 60m deep

For each case the pile's toe should be at least 2m embedded on the hard clay layer (CH) or in the deeper (SM2) dense silty sand layer.

The common criteria for acceptance of preliminary pile load test can be defined as:

- Maximum applied load at least 2.5 of working load
- Maximum settlement less than 10% of pile base diameter.

The common criteria for acceptance of working pile load test can be defined as:

- Maximum applied load 1.5 ÷ 2.0 times the working load
- Total maximum settlement ≤ 25 mm (depending also on the displacement allowed by the type of building)
- Maximum residual settlement ≤ 13 mm.
- Maximum settlement at working load ≤10mm.

The use of instrumentation inside the pile as extensometers and strain gauges is always recommended to properly define the contribution of the various soil layers.

2.2 Diaphragm wall

Deep excavation often requires lateral support to prevent excessive movement of the surrounding soil. The diaphragm wall technique is one of the method to create a earth retaining structure supporting the lateral pressure of both earth and water. The diaphragm wall construction sequence is shown in Fig. 8.

Fig. 8 Diaphragm wall - Construction sequence

Fig. 9 Diaphragm wall site (Saigon Tower)

The excavation tool is a rope suspended hydraulic grab mounted on crawler crane.

Prior to the commencement of the diaphragm wall, two concrete guide walls are constructed along the axis of the trench. They provide a permanent alignment to the grab during excavation.

The excavation, as for the bored piles, is performed under bentonite mud suspension that guarantees the stability of the trench.

Once the excavation has reached the desired depth, the panel is desanded until the prescribed sand content is obtained.

The steel cages are usually reinforced by diagonals and transversal stirrups providing an adequate rigidity to avoid deformations during lifting and lowering operations.

The diaphragm wall is concreted by the "Contractor" method, through a tremie pipe, equipped with a funnel on its upper end.

By this method, the concrete fills the trench from bottom upwards so that the top layer of concrete polluted by the mud can be removed when it reaches ground level.

The excavation sequence usually foresees the execution of alternate primary panels, followed by the excavation of the secondary panel in between.

The continuity of the concrete curtain is granted by the joints that are waterproof if the conctact concrete-concrete between two adjacent panels lengthenens the hydraulic path.

The design of a diaphragm wall structure is usually performed by means of computer programs since the solution of the equilibrium equations of a multi-tied structure is iterative.

In case of limited number of support, or for a cantiliver structure the limit equilibrium analysis is a typical approach that can be successfully applied.

The finite element method is now a normal design technique that better follows the excavation process, and can be fitted to different or anomalous loading condition where the limit equilibrium (Rankine or Coulomb) analysis are not suitable.

For limited excavation depth the diaphragm can be realized with no supports, and works as a cantiliver structure with adequate embeddment. Meanwhile for greater depths, or heavy loads applied beside, horizontal struts are needed on one ore more levels.

In such case the scope of the design is to optimize the levels of struts (or anchors) in order that the bending moment should be symmetrically disposed on earth and excavation side for all the excavation phases.

The calculation of the stress-strain relationship has to take into account the active and passive earth pressure, together with the water pressure that often has a relevant effect.

The problem of the presence of water, imply also that in case of dewatering the seepage forces should be considered since they are favourable on the earth side (alike increasing of soil weight), but are unfavourable on the passive side, because they provide an uplift force which unburden the soil (with also risks of piping).

For a better knowledge about the diaphragm wall technique the technical literature is reccomended.

3. UNDERPINNING

An existing structure may be underpinned, simply, by excavating and forming mass concrete foundations or by using any kind of soil improvement, piles, micropiles as deep foundation elements.

In the present paper the techniques of micropiling and grouting are shortly analyzed, then an application of both is described with reference to an underpinning application carried out in HCMCity for the Saigon Metropolitan Tower Project.

3.1. Micropiles

Bored piles with diameter not exceeding 250 mm are usually called micropiles.

The Ropress® micropile technology was studied, developed and utilized first in France and Italy. This type of pile is usually defined as "high capacity micropile". The main feature of a micropile is to be executable by small sized rigs through any type of soil, boulders and cemented layers included, and through masonry. Micropiles are hence suitable for underpinning existing structures, for providing deep foundations to new structures close to or inside existing ones, and where the site does not allow the operation of common-size piling rigs. They are also suitable for deep foundations in particular soil conditions, for instance where alternation of hard and weak layers precedes a deeper firm stratum.

The construction phases are illustrated in Fig. 10.

a) Drilling;
b) Installation of steel pipe;
c) Covering mortar injection;
d) Bulb injection in successive stages;
e) Completed Micropile.

Fig. 10 Micropiles - Construction sequence

Either rotary or rotary percussive drilling is carried out, with or without temporary casing, according to soil and environmental conditions, to design specifications and to general features of the site.

Once the drilling is complete, a steel pipe is installed into the borehole; having the double function of reinforcement and grouting pipe.

The first stage grouting is a simple sealing of the annular space between the pipe and the soil. Once this sleeve grout has set, pressure grouting is performed by installing a grout duct with a double packer along the grout pipe and forcing the grout through no-return valves ("manchettes") distributed along the grouted length at the design interval (usually 0.5÷1.0m).

Pressure grouting is repeated once or several times, each one after the setting of the previous stage and, at the end, the pipe is filled with grout. The grout is usually a water/cement mix with eventual addition of bentonite and/or plasticizer to prevent segregation. Grouting pressures in the final stage reach 1÷2 MPa, increasing with depth and soil resistance.

The bearing capacity of micropiles is relevant if compared to their diameter, as the disturbance of soil during drilling is comparatively smaller than for common bored piles. The pressure applied to the mortar (or mix) both increases the effective diameter of the pile and improves the soil/pipe bond strength. Allowable bearing capacities can range from 100 kN for a 85 mm root pile to 1000 kN for a 170 mm Ropress® micropile reinforced with a thick steel pipe.

Other types of micropiles are derived from the above basic type:

- A plain steel pipe can be used as reinforcement, the injection under pressure being performed through the casing as it is withdrawn.
- The Gewi pile, developed in Germany (Koreck, 1978) has a reinforcement provided by a high strength steel bar (a Dywidag bar). A multi-stage injection is performed through a plastic sleeved pipe parallel to the bar.
- In France, Ménard adopted an inflatable flexible cylinder to expand the mortar against the soil.

3.2. Grouting

Grouting is a procedure which can not be strictly standardized because of the variability of the soil nature. It consists of a sequence of different operations:

- drilling a hole of an appropriate diameter in the established pattern and depth,
- preparation, proportioning, weighting and mixing of the selected grout suspension,
- injecting the prepared suspension into the designed section of the borehole from which the voids are filled.

The construction phases are illustrated in Fig. 11.

Permeation grouting is feasible with a wide variety of mixtures ranging from particulate suspensions to colloidal and pure chemical solutions, but both technical and economical hazards increase with decreasing soil permeability.

In an actual permeation treatment the flow rate and pressure at each injection level must be selected and adjusted in order to prevent hydrofracturing, particularly when chemical mixes are used.

In terms of the permeability coefficient the normal permeation limits are of the order of 10^{-5} m/s for silicate-based mixture and 10^{-6} m/s for the most expensive resin-based grouts.

The sleeved pipe injection (tubes à manchettes) is a multiple phases process, which allows several successive injections in the same zone. This involves installing a sleeve pipe into a grout hole. This pipe is permanently sealed-in with a sleeve grout composed of a cement-bentonite-water mixture (usually C/W ratio by weight = 0.5).

The sleeve grout seals the borehole between the pipe and the soil to prevent the injection grout from channelling along the borehole. This means that, under pressure, the injection grout will break through in radial directions and penetrate into the soil. At fixed intervals, of the order of 0.5 m, small holes have been drilled into the pipe to act as outlets for the grout. The holes are tightly covered by rubber sleeves (manchettes) which open only when under pressure. The holes and sleeves work as one-way valves.

Once the sleeve grout has set, pressure grouting can be performed. In order to inject through a sleeve, a double packer fixed at the end of a smaller diameter injection pipe is inserted into the sleeve pipe and located close to the sleeve so to form a closed chamber with one-way valve outlets.

This method presents numerous advantages. Firstly, there is the possibility of repeating the injection several times, which permits to use, at a different time, different mixes with decreasing viscosity to ensure the penetration of the grout in the fine voids after the larger ones have been closed.

The more pervious soil layers may be firstly sealed regardless the order of injection level, which prevents loss of expensive low viscosity grouts.

Moreover, it is to be noted that the grouting operations are carried out completely independent from drilling.

3.3. Case history

For the construction of the Saigon Metropolitan Project (Nguyen Du - Dong Khoi Corner), a diaphragm wall structure has been foreseen to allow the excavation down to 9.35m from ground level, for the execution of two underground basements.

Being the diaphragm wall strictly close to the foundation of a three storyes school building, the underpinning of its shallow footings was required to guarantee its stability and to avoid any disturbance (settlement) on the structure of the building itself.

1. drilling and inserting of sleeved pipe for grouting
2. double packer set at the lowermost valve, forming of the sheath in the annular space between the pipe and the soil
3. double packer set at the lowermost valve for the up-stage grouting of soil and breaking of sheath
4. soil grouting through the lowermost valve
5. grouting through upper valves up to completion

Fig. 11 Grouting - Construction sequence

Fig. 12 Underpinning scheme (Saigon Metropolitan Tower)

Fig. 12 shows the layout of the foundation composed of three footings supporting three columns of the building. The same figure contains the vertical sections on the axis of the columns with the position of the expected diaphragm wall.

Since the diaphragm wall trench would have released the horizontal stresses of the soil just below the footings, the design solution was directed to consolidate this part of soil for the stability of the trench, then to transfer the vertical loads of the building to deeper levels near the bottom of the diaphragm wall. The load acting on the columns was estimated 700 kN for external ones and 1400 kN for the central one. The soil stratigraphy was characterized by the typical layers sequence as described in chapter 2.

With reference to Fig. 12, the execution of underpinning foresaw the following phases:

a) Soil treatment by grouting;
b) Micropiling;
c) Connection of micropiles, footings and columns with a r.c. beam.

No. 11 holes were drilled for each footing with different inclination (5° and 20°), inserting sleeved pipes (PVC pipes 1" inner dia with sleeves every 50cm).

The hole was filled from the bottom with cement grout without pressure (creation of an annular sheath around the tube).

The grouting was characterized by a cement mix in lower sleeves while on three upper sleeves a chemical mix was used in order to guarantee a higher permeation in the soil just below the footings where the risk of local failure was higher.

Once completed the grouting, no 20 holes (100mm dia) have been drilled laterally to each footings with inclination of 5° on vertical axis.

Plain steel pipes 63 mm dia, 6 mm thickness, 20m length were inserted into the holes. Such micropiles were not equipped with sleeves being their bearing capacity verified exclusively by the injection of cement grout from the bottom of the pipe.

In addition to those micropiles (5^o inclination), four additional micropiles (40^o inclination) were installed, acting as tension elements on the static force equilibrium scheme (see Fig. 12).

The upper ends of micropiles were roughened by welding short pieces of steel bar, in order to provide a good adherence with concrete of the zone embedded in the connecting beam.

For a proper connection of footings, columns and reaction elements, some dowel bars were fixed to the existing footings, by means of epoxy resin. Then the connecting beam was installed joining the three footings to guarantee the uniform distribution of all the forces (applied and reactions) along its stretch.

During all the above phases, the building movements were monitored by accurate level measurement on reference marks installed on three columns.

The critical phases were the drilling (release of soil stresses) and the pressure grouting (re-compression of soil with uplift risk) for which the surveyor reading was maintained full time.

The diaphragm wall was then executed with no relevant disturbance to the school building, recording a maximum movement of approx. 5 mm.

4. FUTURE DEVELOPMENTS

Since HCMCity is growing and renewing with an impressive rapidity, the future developments should meet the requirement of the city, supported by the knowledge and technology already experienced in the civil engineering of other countries.

In the present paper, some technologies, new for Vietnam engineering, have been already presented, as Bored piling, Diaphragm walling, Micropiling and grouting.

The introduction of new systems is advisable for better operate with the problems related to the geotechnical engineering. For instance the anchorage's technique and the jet grouting system are treated in the followings.

4.1. ANCHORAGES

In the recent past there has been a considerable upsurge in the use of ground anchors, and in many countries they have established a permanent place in civil engineering practice.

At present time, in Vietnam the anchorage technology has not been accustomed yet.

It is hoped that the current publication will prove useful in providing information on this subject.

Ground anchorages play an important and often decisive role for:

– the ever increasing necessity to build structures on geological unfavourable ground;
– the need to build deep foundations or create large underground spaces either by excavation or tunnelling in urban areas adjacent to existing buildings;
– the requirements to counteract uplift and horizontal forces during the construction of large structures;
– the convenience of supporting cuts and stabilizing slopes.

Ground anchor can be defined as high-grade steel tendon capable of transmitting an applied tensile load to a soil bearing stratum.

The "High Pressure Regroutable Anchor" (Rofix®) is made up of the following components (see Fig. 13):

a) fixed length (anchor bulb) secured deep into the ground;
b) sealing device (inflatable packer) which separates the grouted zone from the free zone of the anchor;
c) free length of the anchor;
d) anchor head (active anchorage).

Fig. 13 Layout of typical ground anchor

Drilling can be performed by rotary or rotary- percussive techniques, using water or air flush, bentonite or bentonite-cement circulation fluid. Diameters may range from 100 to 150 mm depending on the nature of the soil and the required loads.

The installation of the tendon into the hole is facilitated by its rigidity which also prevents undue bending. Once the borehole has been completely filled with cement grout, the fixed length is grouted under high pressure, through a double packer located into the sleeved pipe.

The composition of the grout injected in each sleeve depends on the nature of the ground surrounding the sleeve. Grouting can be repeated, if necessary in one or more successive phases until predetermined values of pressure or grout take are obtained. The maximum pressure is usually pre-established in order to avoid hydrofracturing the soil (refusal pressure).

Evidence of the successful formation of the anchorage is given when the pressure, required to force the grout into the ground, increases at each phase.

As the protection of the fixed length is a function of the service life of the anchor itself, the design solutions may range from double corrosion protection to simple grout cover.

Once the grout has hardened and the anchor plate has been placed, stressing is carried out either by applying a jack to each individual strand or by using a multistrand jack.

Rofix® anchor can be stressed to working loads ranging from 30 to 180 tons or more, and may be employed for anchoring retaining walls, concrete diaphragm walls, sheet piles, piers and shafts in unstable ground, slope stabilization and support of deep excavations.

Eventually instrumentation can be provided to monitor anchor behaviour with time.

Once the anchor has been installed and stressed, the construction is completed by filling the PVC lining pipe with setting fluids.

The special construction of Rofix® anchors ensures:

a) an efficient interlock between the tendon and the grout injected for the formation of the bulb;
b) the possibility of injecting pre-established grout volumes under pressure, in one or more phases;
c) separation between the free and fixed anchor lengths, which permits the injection of the fixed length to be isolated when high pressures are used;
d) multiple protection against tendon corrosion in the free length, fixed length and anchor head;
e) the possibility of checking possible stress variation which may take place with time and to re-stress the tendon to the designed load.

The technique of ground stabilization with reinforcing elements is based on balancing the forces and limiting the deformations by transferring the existing loads into more stable strata in the ground.

Before locking off the anchor at the required working load, anchor itself is tested to max. 1.5 times its load. Since every anchor is submitted to loading test, the verification of its correct execution is always guaranteed. For a better acknowledgment of the loading test procedures on anchors, the necessary information can be found in reference codes such as BS-8081 (1989).

4.1.1. COMPARISON BETWEEN BRACING AND ANCHORAGE SYSTEM

The walls retaining the sides of deep excavation are often supported during construction works by heavy steel temporary props. Nowadays the bracing system is the only method applied in HCMCity works.

Meanwhile in many countries, the use of ground anchors has overcome the bracings system since the former allows an easier way to perform the successive excavation.

For comparing the two methods, the case of a diaphragm wall 600m thickness on the HCMCity soil has been analyzed (see Fig. 14).

The executive phases can be summarized as follows:

0) Execution of Diaphragm wall (distributed load on earth side q=20 kPa);
1) Excavation down to the depth of -2.5 m on the excavation side;
2) Installation of prop or anchor at level -2.00m;
3) Excavation down to the depth of -5.0 m on the excavation side;
4) Installation of prop or anchor at level -4.50m;
5) Completion of excavation down to the required depth of -7.50 m for the execution of the concrete raft.

In the example the diagrams of the bending moment and horizontal displacement are presented for the phase no. 5.

The possibility to pre-stress the anchors to relevant amount of load (up to 60÷80% of its working load) provides an important reduction of the horizontal displacement of the diaphragm wall (almost 40% less of the case with bracing) and also a better balanced bending moment. Meanwhile the pre-loading of the bracing is generally limited by the large span usually present in large excavations, and to the risk of buckling for the compressed members.

Moreover the propping system is a passive method of reacting against the wall since it begins to operate only when the wall starts to displace.

Regarding the excavation method, the solution with anchors allows to excavate the whole area without any impediment for the trucks or the excavators paths. Therefore the installation of the basement can be performed with no interruption due to the kingposts (columns let to sustain the horizontal bracing frame) or to the props that are usually founded on the core of the basements itself.

Fig. 15 shows clearly how complex is the excavation sequence due to the presence of many layers of bracing.

On the contrary, Fig. 16 shows how much free space is provided by the anchorage system. For instance all the area can be excavated in a single step. A drilling rig can operate on this level for the installation of the first level of anchors that can be pre-stressed 3÷7 days after their execution.

Fig. 14 Comparison between bracing and anchorage system

Fig. 15 Picture of a braced diaphragm wall (International Business Center)

Afterward the excavation can safely proceed down to the successive level of anchors and so on until the bottom of the excavation is reached.

Regarding the problems related to the fact that the anchors are outside of the property for which the work is done, the anchors are usually provisional elements since the diaphragm wall is counteracted by the underground floors and do not need any more support. The anchor heads can be easily disconnected and its pre-stress released.

Therefore the adjacent property is not relevant affected by the presence of such tendons underground. For instance, if another building should be built on piles on the adjacent site, and the anchors' strands or the grouted bulbs are in the path of the new boring, they can be destroyed with quite limited difficulties.

Such procedure does not imply any damage for the diaphragm wall being the anchors already disconnected from their anchor heads.

4.2. JET GROUTING

Soil improvement by jet grouting is one of the methods applicable to solve problems connected to foundations, open cut excavations and tunnelling.

The general basis of jet grouting technique is a special high speed jet acting under a nozzle pressure up to 50 MPa or more.

The soil is fractured and simultaneously mixed in situ with a cement grout, or alternatively removed (to a certain extent depending on grain size and consistency) and replaced by grout jetting. Hence the treatment may imply either the use of single fluid (the grout) as fracturing medium and stabilising agent or the use of 2÷3 fluids (air + grout or air + water as fracturing media, and grout as stabilizing agent).

The sequence of operations related to the single fluid procedure (Rodinjet®1), consists of the main following phases:

- drilling down to the required depth by using a string of rods fitted at the bottom with a drilling andjetting tool (monitor);

Fig. 16 Picture of an anchored diaphragm wall (Latina- Italy)

- grout jetting through radial nozzles located along the monitor axis while revolving and drawing up the tool. In particular cases, the tool is only withdrawn (mono-directional jet grouting).

According to soil conditions, a casing may be used or, quite frequently, uncased boreholes are drilled with direct circulation of water or bentonite mud. The size and mechanical properties of treated soil columns depend on the combined effects of the type of soil and composition of the grout, grout discharge and pressure related to the number and size of nozzles, rotational speed and lifting rate of the monitor. The diameter of single columns (normally ranging between 0.5 and 1.0 m) may be increased to 1.5 m or more by alternative procedures:

- 2 fluids system, which involves air jetting through a coaxial nozzle placed around the grout nozzle (Rodinjet®2),
- 3 fluids system, which involves air-water jetting through coaxial nozzles placed just above the grout injection nozzles (Rodinjet®3).

4.3. BLANKET SYSTEM

One conceivable application of the above described techniques such as jet grouting and permeation grouting, is the execution of watertight blankets to seal the bottom of the underground openings.

For the excavation underwater table one of the main problem to face is the risk of piping, due to the upward hydrostatic pressure.

The possible solutions to by-pass such phenomena are as follows:

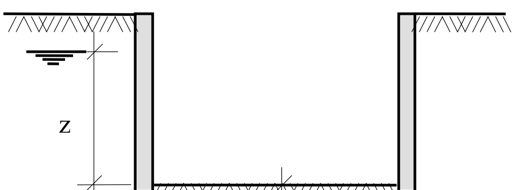
a) Keeping the hydraulic head at least 0.50m below the bottom of excavation by a non-stop dewatering system;
b) Deepening the diaphragm wall curtain down to the impervious layer so to close the basin to any external seepage;
c) Creating a blanket just at the base of the diaphragm wall whose depth can be limited behaving only as structural element and not as cut off wall of the seepage flow.

The blanket solution is generally adopted when the natural impervious layer is too deep to be reached, or the non-stop dewatering is not allowed for the risk of settlements on adjacent buildings.

Fig. 18 shows a typical blanket scheme together with the equilibrium equation that shall be verified with an adequate safety factor to avoid piping effects.

The blanket is usually performed by means of a drilling rig that, once reached the foreseen depth, can inject a cement mix at high pressure (blanket made of jet grouted columns), or a sleeved pipe can be inserted into the hole for a subsequent grouting process based on chemical or cement mix.

Fig. 17 Jet grouting - Construction sequence



$$(\gamma_S\, d + \gamma_B\, t) > \gamma_W\, (D+z)$$

γ_B

Fig. 18 Blanket scheme

Such construction method consents to carry out the excavation in dry condition since the dewatering is limited to pump out the inner water of the basin, with negligible water seepage from the external side of the diaphragm wall.

5. UNDERGROUND CAR PARKING

Nowadays, the problem of car traffic in HCMCity has not arisen yet, since fortunately, so far, the main transport means are motor bikes that can easily flow between any traffic jam.

Following the faster and faster economical development of all Vietnam, the expected growing number of cars in the cities should be controlled in order to avoid the experienced problems encountered in other towns (such as Bangkok, Jakarta and many European cities).

On the basis of such experiences, one possible reduction of the in-coming traffic problems is the construction of an adequate number of car parking.

Multi-story or underground parking can be two alternative solutions, and of course for every new building the number of car spaces should be at least equal to the number of rooms.

Therefore, the number of future car users should be considered and taken into account for a proper dimensioning of any urban project.

Two cases are here presented of underground parking as solution of growing car traffic:

a) Underground parking of Toschi Street in Parma - Italy;
b) Underground story for commercial and garage of Frederickstadt Passagen - Berlin - Germany

Parma Municipality, seen that the ratio car-inhabitants was approaching unity, foresaw the construction of an underground parking in a site located parallel to the Parma creek and close to existing buildings of its ancient center.

The work began in 1989 and finished in 1992.

The diaphragm wall curtain (600mm thick) allowed the excavation down to the design depth of -13.50m, by using two rows of anchors well evidenced in Fig. 19.

The geologic sequence was characterized by high water table (due to vicinity to the flow of Parma creek) and by a base layer of silty clay where the diaphragm wall could be embedded to prevent any piping effects due to the hydraulic head while excavating with a direct dewatering.

Four underground floors were foreseen, holding 7000 m^2 of parking area each.

Now, the total number of 973 parking spaces provided partly for residents, and partly for casual users, allows a reduction of the cars stationery, being such zone close to the down-town commercial center.

For Friedrichstadt Passagen, an important central street in East Berlin central area, three architectural outstanding buildings have been erected from 1992 to 1993.

For such gigantic project the construction pit, 15m depth, extends across three blocks and two underground railway tunnels border the site on two sides.

The foundation works spreading on an aerea of 21000m^2 should have been ready on 9 months.

Three underground floors were foreseen for parking and commercial sections.

The earth retaining structure was performed with a 600mm thickness plastic cut-off wall embedded down into the impermeable lignite layer at approx. 50m from ground level. The watertable in Berlin area is at 3.0 m from ground level (31m above sea level).

A sheet pile was inserted into the self-hardening mud with its toe 5m deeper than the final excavation level.

Wells have been used for lower down the water table inside the pit.

Two rows of inclined grouted ground anchors (30m length) were installed as support of the lateral earth and water pressure. In some zones, close to existing buildings, three levels of anchors were used.

The floor slab (2m thick) had to be constructed secure against uplift even without the weight of the building. For that reason 2000 vertical anchors were installed to tie down the slab against the upward hydrostatic water pressure.

Fig. 19 Underground parking of Toschi Street - Parma - Italy

Fig. 20 Underground story of Friedrickstadt Passagen - Berlin - Germany

Once the heavy reinforced concrete of the basement had been hardened enough, the dewatering was switched-off.

The building was then completed despite of its technical complexity and of the strict time requirements.

6. CONCLUSIONS

The present paper firstly has described the recently applied technology in foundation of high rise buildings in Vietnam. Bored piles and diaphragm wall have been shortly shown as efficient technical solutions for deep excavation works in urban areas.

Then two new techniques (Grouting and Micro-piling) have been depicted related to an application of underpinning of an existing building close to a foreseen deep excavation.

The future developments of a fast growing city like HCMCity has been focused with special regards to the introduction of advanced technologies. Anchors, grouting and jet grouting techniques are useful means of the geotechnical engineering, to overcome the most complex requirements of the new projects.

Alternative methods for the excavation under watertable are advisable by using grouting blanket instead of a continuous dewatering system, thus preventing its risks such as settlements or piping effects.

Underground car parking is then suggested as preventive solution to the expected future traffic development of a growing town like HCMCity.

It is hope of the authors that the above techniques could be applied (with the approval of authorities) in the future underground works in Vietnam.

ACKNOWLEDGMENTS

The authors wish to thank the following companies for the useful references of their sites:

PLAZA HOTEL & OFFICE TOWER
Owner: Plaza Hotel JV Co. Ltd.
Consultant: Sleadon Management Serv. Ltd
Architect: Meinhardt (Singapore) Pte.Ltd.
SAIGON TOWER
Main Contractor: Transfield Construction - Asia
OCEAN PLACE
Owner: Ocean Place JV Co. Ltd.
Consultant: RSP Architects Pte. Ltd
DIAMOND PLAZA
Main Contractor: Posec
SAIGON METROPOLITAN TOWER
Main Contractor: Vina Leighton
PARK HYATT HOTEL
Owner: Grand Imperial Saigon Hotel Ltd.
Consultant: Damansara Architect
Pakatan Cergas Sdn . Bhd.

The authors apologize for errors or missing acknowledgment.

LIST OF WORKS RODIO CARRIED OUT IN VIETNAM UP TO DATE

HCMCity

- Plaza Hotel & Office Tower - 800mm dia bored piles;
- Saigon Tower - 600mm thk diaphragm wall;
- Point Des Blagueres - Loading test on 800mm dia bored piles;
- Ocean Place - 800, 1000, 1200mm dia bored piles and 600-800mm thk diaphragm wall and barrettes;
- International Business Center (Diamond Plaza Hotel) - 1200mm dia bored piles and 800mm thk diaphragm wall;
- Saigon Metropolitan Tower - 600mm thk diaphragm wall;
- Park Hyatt Hotel - 750, 835mm dia bored piles and 600-800mm thk diaphragm wall and barrettes;
- Me Linh Point - 900mm, 1200mm dia bored piles and 800mm thk diaphragm wall and barrettes.

Hanoi

- Rose Garden - 1000mm dia bored piles and 600-800mm thk diaphragm wall and barrettes.

Others zones

- Cat Lai - Morning Star Cement handling Terminal - 1200mm dia bored piles;
- Hon Chong - Morning Star Cement Factory - Loading test on 1200mm dia bored piles.

REFERENCES

Bustamante M., Doix B,. (1985). "*Une méthode pour le calcul des tirants et des micropiueux injectés*", Lab. des Pont et Chauseé 140 Nov-Dec. - pp 75-92.

BS 8081, (1989). "*British Standard Code of practice for Ground anchorages*".

BS 8004, (1986). "*British Standard Code of practice for Foundations*".

De Paoli B., Stella C., Perelli Cippo A., Locatelli L. (1991). "*A monitoring system for the quality assessment of the jet grouting process through an energy approach".* Proc. 4th Int. Conf. on Piling and Deep Foundations. Stresa, Italy, April 7-12, pp.211-215.

ITA (International Tunnelling Association) Working Group no. 13, (1995). "*Underground Car Parks: international case Studies*", Tunnelling and Underground Space Technology, Vol.10, no.3, pp.321-342.

Koreck W., (1978). "*Small diameter bored injection piles*", Groung Engineering, May.

Manassero V., (1993). "*Different techniques for soil improvement and underpinning*", Novel Foundation Techniques Symposium of Cambridge Programme for Industry. Magdalene College, Cambridge, September 2-3.

Mascardi C., (1982), "*Design Criteria and Performance of Micropiles*", Symposium on Soil and Rock Improvement, Techniques Including Geotextiles, Reinforced Earth and Modern Piling Methods, Bangkok, December, Paper D-3.

Thorburn S., (1985), "*The philosophy of underpinning*", Underpinning, Surrey University press, pp. 1-39.

Tomlinson, M. J., (1987), "*Pile Design and Construction Practice* ", A Viewpoint Publication.

Tornaghi R., Bosco B., De Paoli B., (1988), "*Application of recently developed grouting procedures for tunnelling in the Milan urban area*", Fifth International Symposium Tunnelling 88, London, 18-21 April.

Various Authors, (1989). "*Anchors and injected piles*", 12th ICMSFE, Rio de Janeiro, pp. 1005-1052. Session 12. Vol. 2.

Various Authors, (1993). "*An underground parking garage in the historic center of Parma*", L'Industria Italiana del Cemento. N.683 - December - pp. 784-801.

Various Authors, (1975), "*Diaphragm walls & Anchorages*", Proceeding of Conference of Institution of Civil Engineers. London. September 18-20.

Warner J., (1982), "*Compaction grouting. The first thirty years*", Proceedings of the Conference on Grouting in Geotechnical Engineering, New Orleans, February 10-12.

Warner J., Schmidt N., Reed J., Shepardson D., Lamb R., Wong S., (1992), *"Recent advances in compacting grouting technology*", Proceedings of the conference on Grouting, Soil improvement and Geosynthetics, Vol. 1, New Orleans, February 25-28.

LIST OF FIGURES