Republic of Iraq
Ministry of Higher
Education and Scientific Research
University of Technology

Bored Pile

**Annual project Submitted to the Department of Building of and Construction Engineering of the University of Technology in Partial Fulfillment of Requirements for the Degree of B.Sc.
In Road Way and Bridge Engineering**

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الاهداء

الى من سهرت علي الليالي ....

وغمرتني بالحب والحنان .....

واشعرتني بالسعادة والامان ....

والدتي الحنونة

الى من ضحى بحياته كي نسعد .....

الى من بذل روحه لايصالي لهذا الطريق .....

الى النور الذي اضاء لي دربي .....

والدي الحبيب

الى اصدقاء عمري....

واحباء قلبي ....

اخوتي الاحباء

شكر وتقدير

بعد الحمد والشكر لله تعالى لا يسعني إلا إن أقدم امتناني وتقديري إلى رئاسة القسم وإلى الأساتذة الأفاضل وإلى الأستاذ

المشرف حسام حكمت الذي كان معي بجهوده المخلصة وتوجيهاته القيمة في الإشراف علي .

وأتقدم بالشكر إلى كل من أهلي وأساتذتي وزملائي وزميلاتي لدعمهم المستمر لي وإلى كل من كان له الفضل في تشجيعي وحثي على إكمال هذا العمل.

Contents :

Introduction

In our steady "Bored Piles" we focus on nine chapters as follow:

Chapter 1 " general specifications" in this chapter we explain that many requirements for bored piles such that construction, reinforcement, pile caps, concrete trim, problems of bored piles,…etc.

Chapter 2"Shaft Friction of Bored Piles in Hard Clay" in this chapter we explain that the Pile foundations are the part of a heavy structure used to carry and transfer its load to the bearing ground located at some depth below ground surface. Depending upon various factors like nature of substrata, depth of ground water table, depth of stronger stratum, type and quantum of load to be supported etc., piles are designed. Pile testing is considered a fundamental part of pile foundation design. It is one of the most effective means of dealing with uncertainties that inevitably arise during the design and construction of piles.

Chapter 3"experimental and numerical analyses of bored pile foundations tropical soil "in this chapter we explain that the numerical analysis was carried out with some of the piles, in order to simulate there in situ behavior with the existing (lab and field) data. Two types of piles were selected for this particular analysis, i.e., a mechanically bored and a "root" type cast-in-place pile. The numerical analysis was done with a semi analytical procedure. This software computes the settlement and the distribution of the normal force inside the pile and actual shear resistance at any depth in the pile's shaft

Chapter 4 "numerical analyses of load tests on bored piles"in this chapter we explain that theacceptance of numerical analyses in geotechnical problems is growing and finite element calculations are more and more used in the design of foundations in particular for the design of piled raft foundations. Nevertheless the bearing capacity of a single pile is most usually determined by pile load tests or by empirical methods. For this reason numerical analyses of a load test on a bored pile in stiff clay are shown in full detail and modeling aspects will be discussed. In the paper the influence of several variables on the load-settlement behavior of the pile are discussed, i.e. the mesh dependency as well as the effect of interface elements. Different constitutive models are used to simulate the mathematical numerical analyses are compared with the results of the pile load test.

Chapter 5 "concrete for wet processed bored piles" in this chapter we explain that thequality of piles depends on a good construction process including drilling, reinforcement installation and concrete pouring as well as on good quality of concrete. Quality of concrete has some influence on the workmanship with interrelated performances. For wet-process bored piles, concrete is cast under drilling slurry using termite pipes. Good quality concrete in bored piling sense means that the properties and characteristics of the concrete are suitable for the process of work and subsequently meet requirements of the finished product. Continuous concrete pouring which is mandatory in piling and it is sometime disrupted by blockage of segregated or prematurely set concrete mix in the termite pipe. Early setting of concrete after pouring in bored hole can also cause discontinuities in pile by accidental lifting of set concrete during extraction of the temporary casing. Dampness is sometimes found in top section of piles constructed in water-bearing permeable soil layer. The dampness was found to be caused by capillary action of ground water through interconnecting voids formed in the improperly mixed concrete.

Chapter 6 "particular specification for large diameter bored piles with bell-outs" in this chapter we explain that theLarge Diameter Bored Piles with Bell-outs are piles of a diameter exceeding 600 mm formed by boring, chiseling or grabbing with an enlarged base formed by under-reaming, plus filling with concrete. The bell-out at the pile base shall be formed within the bedrock with the use of a reverse circulation drill incorporating an under-reaming head.

Chapter 7 "particular specification for large diameter bored piles socketed into bedrocks" in this chapter we explain that theLarge Diameter Bored Piles Socketed into Bedrocks are those of a diameter exceeding 600 mm formed by boring, chiseling or grabbing, plus filling with concrete. The embedment depth into rocks shall be greater than 600 mm and formed by reverse circulation drill or other method approved by the Supervising Officer (SO), and design requirements for bored piles.

Chapter 8 "design and construction of bored pile foundation" in this chapter we explain that theBored piles are commonly used as foundation to support heavily loaded structures such as high-rise buildings and bridges in view of its

low noise, low vibration, and flexibility of sizes to suit different loading conditions and subsoil conditions. Such attributes are especially favored in urban areas where strict restrictions with regards to noise and vibration are imposed by relevant authorities which restricted the use of other conventional piling system, e.g. driven piles. This paper presents a summary of design methodologies commonly adopted for bored piles under axial compression together with a brief discussion on the construction aspects of bored pile and calculation for geotechnical capacity of bored piles.

Chapter 1

GENERAL SPAECIFICATIONS

Bored pile:

Bored piles are commonly as foundation to support heavily loaded Structures like high-rise buildings and bridges in view of its low noise, low vibration, and Flexibility of sizes to suite different loading conditions and subsoil conditions.

Construction Aspects of bored piles:

1. The size of the cutting tool should not be less than the diameter. Of the pile by more than 75mm. Use of drilling mud for stabilizing the sides of the borehole is also made. Permanent MS liner may be used on top 2-3m to prevent the bore collapse.

2. Reinforcement:
Minimum area of long reinforcement=.4% of the sectional area (calculated on the basis of outside area of the casing or the shaft).the minimum clear cover =40mm. The minimum clear distance between main reinforcement = 100mm. The minimum diameter of the links=6mm. Minimum clear distance between links = 150mm.

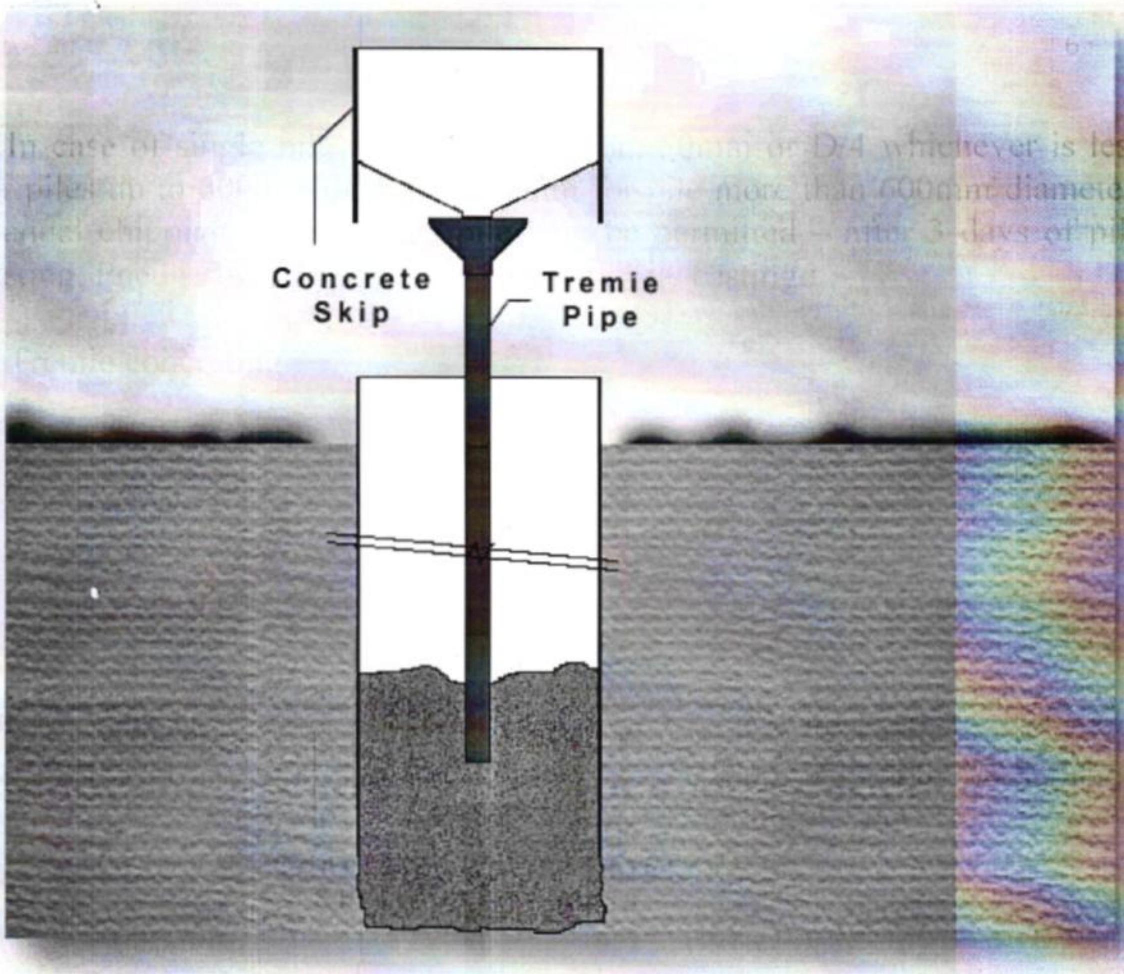
3. Pile cap:
The clear overhang of pile cap beyond the outermost pile in the group = 100mm to 150mm. The pile should project 50mm into the cap concrete. The minimum clear cover for the main rein in the cap 60mm. The cap is generally cast over a 75mm thick leveling course.

For piles of smaller diameter and depth up to 6m. Minimum quantity of cement 350 kg/m3. For piles of bigger diameter and depth more than 6m. Minimum quantity of cement 400 kg/m3 and grade M-20. 10% extra cement to be used for under water concreting. Slump of concrete 150 to 180 mm.

4. Alignment control:
For vertical piles a deviation of not more than 1.5%. For raker piles a deviation of not more than 4%. Pile should not deviate more than 75mm or D/4 whichever is less (for piles less than or equal to 600mm diameter). 75mm or D/10 whichever is more than 600mm diameter pile.

In case of single pile in column position, 50mm or D/4 whichever is less for piles up to 600mm diameter 100mm for pile more than 600mm diameter. Manual chipping of top of the pile may be permitted – after 3 days of pile casting. Pneumatic chipping after 7 days of pile casting.

5. Tremie concreting:



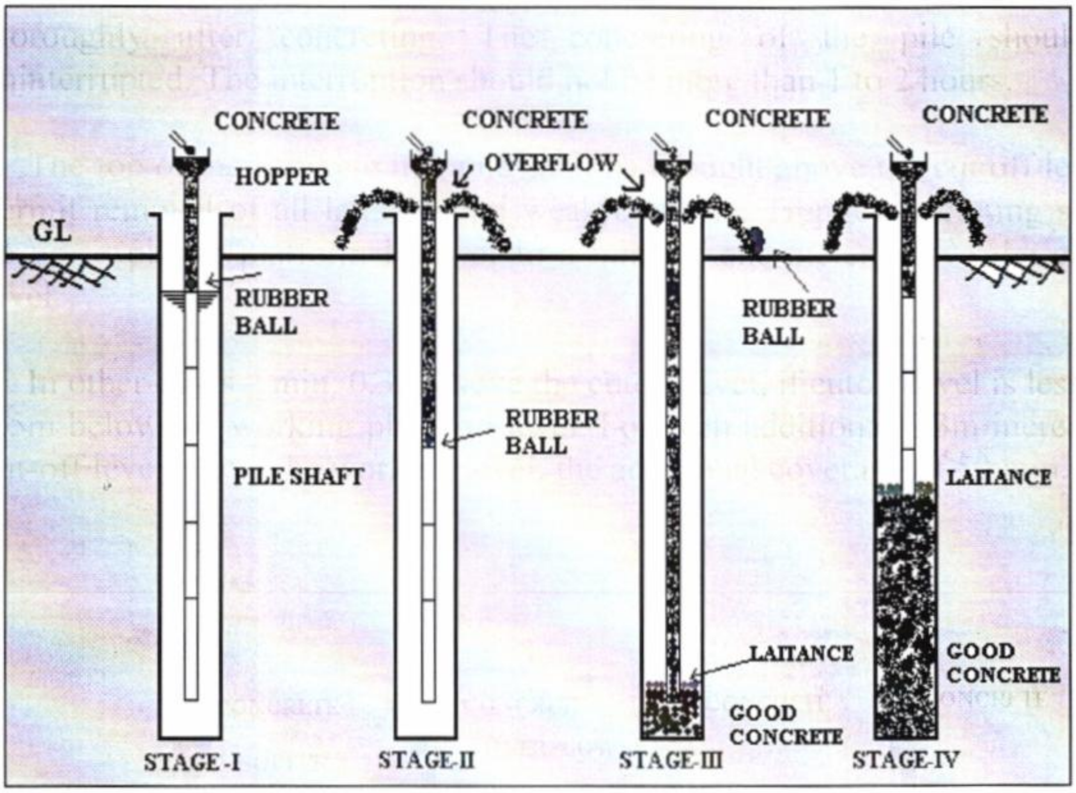
Concrete should be rich in cement (not less than 370 kg/m3). Slump = 150mm to 180mm. In under water – casting for full depth or 2m if non collapsible stratum. If under drilling mud – no casing is required except near the top.

No leakage at the joints. The tremie pipe should be of min. 200mm diameter. The first charge of the concrete should be placed with a sliding plug

pushed down the tube ahead of it. The plug should be taken out and not to be buried into concrete. The tremie pipe should always penetrate well into concrete with an adequate margin. All tremie pipes should be cleaned thoroughly after concreting. The concreting of the pile should be uninterrupted. The interruption should not be more than 1 to 2 hours.

The top of the concrete in a pile shall be brought above the cut off level to permit removal of all laitance and weak concrete. Tremie concreting should be up to pilling platform level or to a min. of one meter above the cut off level.

In other cases – min. 0.3m above the cutoff level, if cutoff level is less than 1.5m below the working platform level. For each additional 0.3m increase in cut-off level below the working level, the additional coverage of 50 mm.



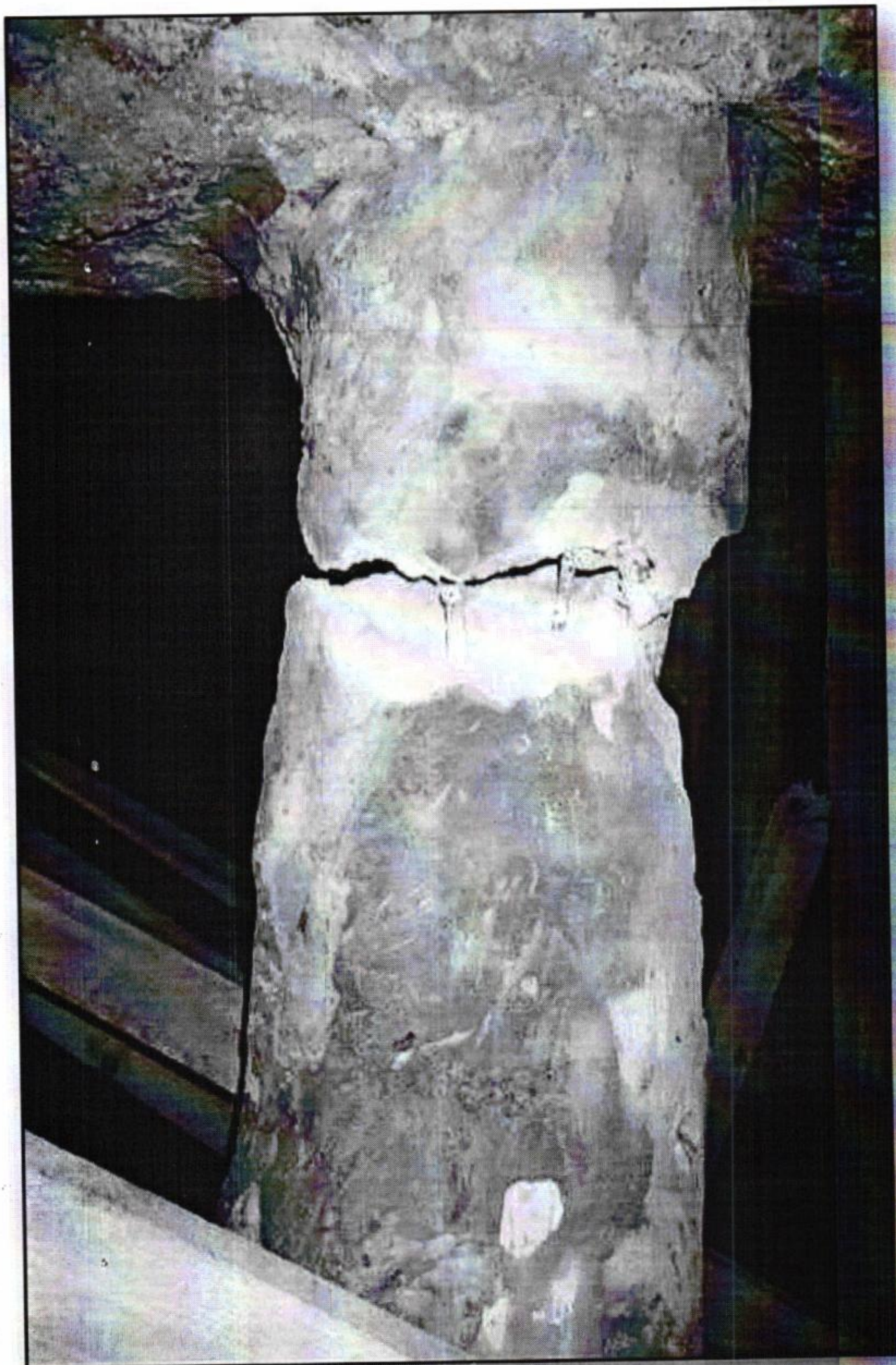
Tremie concreting

6. Rock socketing : for the end pile
Sound, relatively homogenous rock including granite and gneiss -1 to 2d.Moderately weathered closely formed including schist & slate -2 to 3d.For soft rock –3 to 4d

7. Problem in bored pile construction:
*Over break
*Base of borehole
*Extracting temporary casing
*Soft ground
*Drilling mud

Necking in pile

Necking in pile



Necking in pile

Bored piles:

Bored pile are cast in place cylindrical pile excavated either by use of rotary equipment operated augers, buckets, under static drilling fluid or large drill bit (for hard rock) with reverse circulation, with chisel grab and casing oscillator for bouldery ground, with large diameter DTH hammers and compressed air (drilled piles), among others.

Most common large diameter bored piles, are installed through an overburden of cohesive or cohesion less soil strata, with or without water tale, down to firmer ground, to achieve the design bearing capacity by skin friction, base bearing or both, to serve as foundation piles for residential, commercial, institutional building, industrial complexes or infrastructures.

Bored piles installed in common soil with the presence of water table, generally require the use of a short temporary steel casing and a drilling fluid as static suspension to provide support to the surrounding soil while excavating the pile and until complete backfill of the pile excavation with concrete, in order to prevent cave-in of the excavation and destabilizing the surrounding soil formation.
The preparation and handling by most effective drilling fluid, Betonies Mud, is a sophisticated technology by itself and requires a complete set up of dedicated equipment and (basic) field laboratory.
The most common diameters of bored piles range from 0.6 meter to 2.0 m meters, likewise length can range from few meters to sixty or more meters, depending upon design loads and soil parameters. Bored piles can be heavily reinforced if required by design, rebar cages usually are prefabricated in segments with length and weight depending upon available commercial lengths of rebar's and available lifting equipment. Splicing of rebar cages can be done by lap splice, welded lap splice or mechanical threaded couplers. Casting is done by pouring concrete with the design strength and slump as required, through watertight segmental Tremie Pipes, starting from the pile bottom and letting the tremie pipe bottom end remain at least 3 meters submerged in concrete until the completion of pouring, to guarantee the pile continuity and the final good quality of the concrete cast.
Drilling fluids, if needed, can be water, a suspension of betonies (betonies mud), a suspension of polymers, depending upon soil type, soil conditions, presence and elevation of water table, chemical properties of water table (Ph, Salinity).

To allow many uses, are provided with collars for easy handling by vibro hammers and diameter slightly larger than bored piles' nominal diameter, to

allow easy passage of drilling tools. Permanent casings, if needed, are sacrificial casings and as such the wall thickness is as small as allowed by the need to drive the casing through the ground.
Bored piles are commonly employed for bridge foundations, on land and water, because the versatility of bored piles design and execution allows the construction of practically any needed diameter, including the very large diameters, and the pile reinforcement can be provided as heavy as needed by seismic design and the codes (it is not uncommon to have double wall rebar cages in order to accommodate all the needed bars, however care shall be taken to leave sufficient space between bars for concrete to low through).

Bored piles are also used to form retaining walls, as contiguous pile wall or secant pile wall or aligned pile wall, with or without post tensioned soil anchors as tie back.
Bored piles testing is usually done in two (2) stages, first stage testing to verify design assumptions and achievable design load, is done before starting the execution of the working piles: test piles are installed in the proposed construction area, as per design, and tested, first for integrity and continuity by P.I.T. (Pile Integrity Test) then for load bearing capacity , either by Static Load Test or by Dynamic Load Test PDA, (Pile Dynamic Analysis). At times the Designer might require a Pullout test and a lateral load test.
Once the design pile capacity has been confirmed, Bored Piles construction for the working piles starts and quality control is then done on representative piles. Quality Control consists of testing the material used for the bored piles, i.e. reinforcing bars and concrete, then testing of the piles at random with PIT and PDA, and predetermined piles with static pile load test. Predetermined piles can also be tested using the cross-hole_ultrasonic test, by inserting instruments through vertical pipes installed within the reinforcing steel cage all throughout the bored pile length.

Sometime referred to as drilled piers, bored piles are cast-in-place piles ranging from 600mm to 6000mm in diameter with depth that can reach down to 100 meters. Bored piles are installed by first removing the soil by a drilling process and then constructing the pile by placing concrete in the hole. The simplest form of construction consists of drilling an unlined or unprotected hole and filling it with concrete. Complications that may arise such as difficult ground conditions and the presence of ground water have led to the development of special drilling technologies. The choice of the correct drilling technology must be done in a way as to minimize disturbance of the surrounding soil. For cohesion less soils (sands, gravels, silts), whether under the water table or not, the pile borehole must be supported using steel casing or stabilizing mud's such as betonies suspension.

Bored piles founded in rock provide an effective means of minimizing Foundation settlements and a small number of high capacity bored Piles can often provide significant savings in pile cap costs over other, lower capacity, pile types.Being non-displacement type piles, bored piles can be installed with little or no vibration, and with much lower noise levels than driven piles.

Soil and or rock is removed using purpose designed drill tools including soil and rock augers, drilling buckets, core barrels, and down hole hammer drills. Drilling to depths of up to 60 meters and to diameters from 300mm to in excess of 2 meters is possible in soil and rock.
Various methods of support for the sides of bored piles during Construction is available. These can be selected to suit the type of Formation being drilled, the ground water regime encountered, and site environmental constraints.

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BORED PILES, can't Shaft support methods available include:

* Vibrated temporary casing.
* Drilled temporary casing.
* Oscillated temporary casing.
* Permanent liners.
* Drilling under water.
* Drilling under betonies.
* Drilling under polymer fluids.

The load capacity of bored piles is a function of the geotechnical Capacity of the pile, the installation technique chosen, and the Structural capacity of the pile shaft. The capacity of piles socketed into good quality rock is often limited by settlement considerations.

Bored piles are also particularly suited to providing resistance to high lateral loads such as those induced by wind loading and earthquake loading.
In these circumstances the larger diameters available, combined with heavy steel reinforcement cages if required, provide the required structural strength
In very hard rock, bored piles can be constructed using down the hole hammer drills or roller cutter core barrels. These techniques allow the drilling of hard

and abrasive rocks, which would otherwise be very difficult to penetrate economically with conventional drilling equipment.

In some circumstances, belling techniques may prove economical to take advantage of high end-bearing resistance. Typical bell diameters of up to 2 times the shaft diameter can be constructed by Mechanical means.

The actual load capacity of bored piles can be verified by static Geotechnical calculations, by logging of shafts during drilling, by static or dynamic load testing, or by Statnamic load testing which provides an economical means of testing to high load levels.

Advantages:

- Length can be varied
- Removed soil can be compared with design data
- Penetration tests can be carried out in boreholes
- Very large bases can be formed in favorable ground
- Drilling tools can break up boulders and other obstructions
- Pile is designed to working stresses
- Very long lengths possible
- Little noise and vibration during construction
- No ground heave

Disadvantages:

- Piles liable to squeezing and necking in soft soils
- Special techniques required for concreting in water bearing ground
- Concrete cannot be inspected after installation
- Enlarged bases cannot be formed in collapseable soil
- Cannot be easily extended above ground
- Boring may cause instability and settlement of adjacent structures

Rotary Boring:

Rotary bored piles can be constructed either as continuous flight auger (CFA) or using a short auger and Kelly bar.

The CFA technique uses the selferecting hydraulic units and is one of the quietest forms of piling and virtually vibration free. The system obviates the need for any form of casing, using the auger itself to maintain the integrity of the bore during drilling, and the concrete during auger extraction. After concreting, the reinforcing cage is inserted into the fluid concrete – cages up

to 12 meters length are common. Rotary augered piles are constructed with crane mounted or hydraulic units using a short auger and Kelly bar. Where ground conditions dictate, a length of temporary casing can be installed to support non-cohesive strata and prevent ingress of ground water.

A reinforcement cage is placed within the pile prior to concreting and any temporary casing is removed as the final operation in construction.

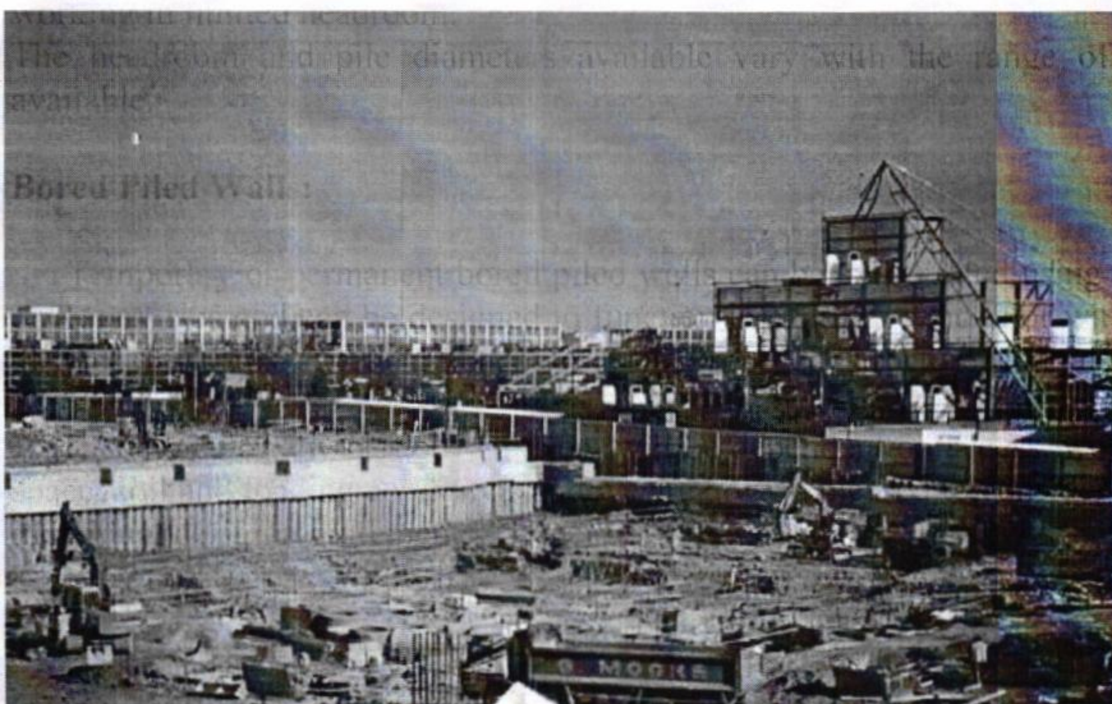
Where design conditions require, a heavy reinforcing cage can be installed to withstand horizontal loads or long reinforcing cages to withstand tension loads. Pile diameters of 300mm, 400mm, 500mm and 600mm are available with depths of up to 30 meters for CFA and 40 meters for Rotary augered piles in favorable ground conditions.

Where access and working space are restricted, purpose built smaller rigs is used to construct Small Diameter bored piles. Piles can be installed working in limited headroom.
The headroom and pile diameters available vary with the range of rigs available.

Bored Piled Walls:

Temporary or permanent bored piled walls can be constructed using CFA or Rotary rigs and can be designed to function in cantilever mode or with the support of props. Vertical loads from theProposed structure can also be accommodated.

The choice of any particular system will be dependent on access, working space, soil and loading conditions and we are always pleased to consider your individual design requirements.

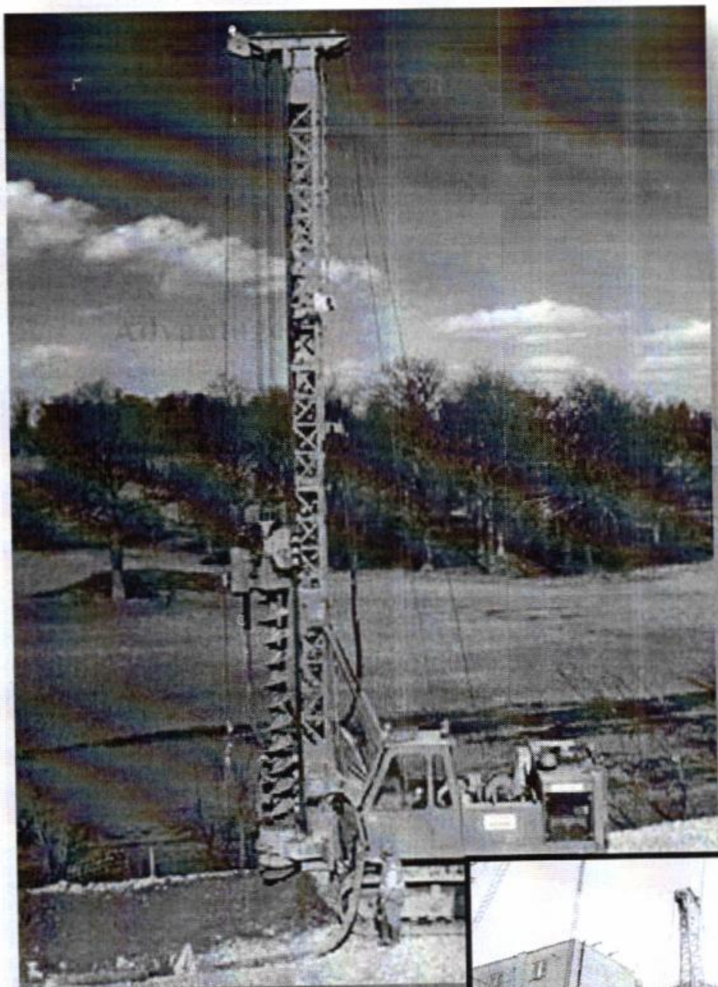


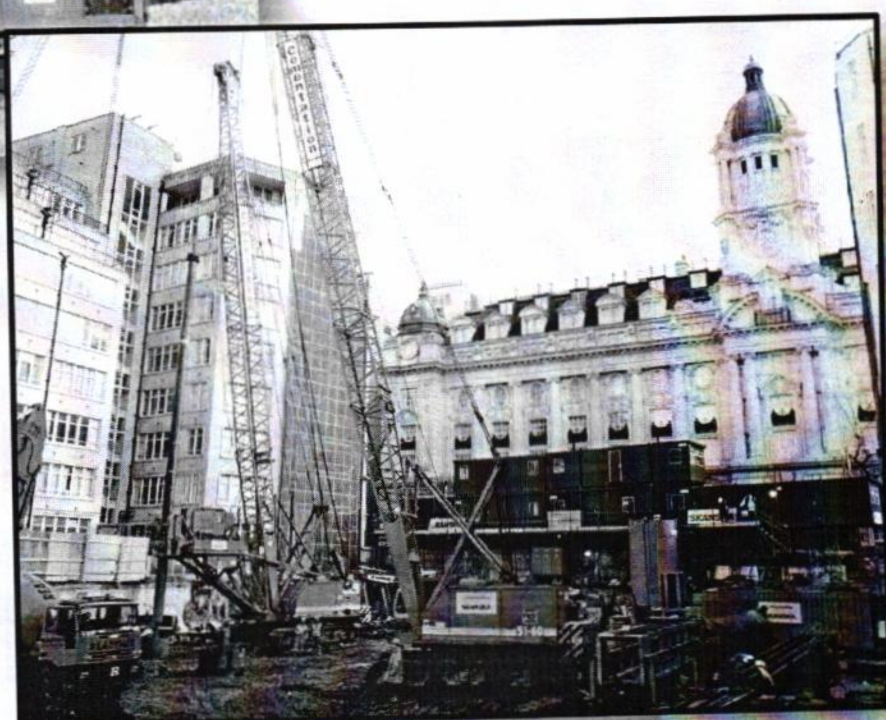
Application:

Suited to most types of soft and/or water bearing soils.Temporary casings can be used on rotary bored piles through Non-cohesive strata.
Diameters generally within the range of 300mm to 600mm with depths up to 30 meters for CFA piles and 40 meters for Rotary augured piles.

Advantages:

- CFA is vibration less and quietest known form of piling
- Vibration less and quietest known form of piling
- Smaller rigs can operate in confined spaces with limited access
- Small diameter piles can be installed near to existing services
- Generally reduced noise levels





Large diameter bored pile:

All of the rigs are fitted with a rotary boring unit, which operates a Kelly bar. The boring tools are fitted to the Kelly bar. This equipment has been used to construct large diameter piles in a wide range of Strata, including very soft silty clays, noncohesive soils and weak rocks. The rigs can impart a vertical load on to the Kelly bar to improve production in difficult strata. In addition to installing vertical piles, rigs can install raking piles, up to 1:4 rakes, without Modificatio

Depths:

Kelly bars can be of single or telescopic construction. Standard triple telescopic Kelly bars allow for depths of up to 55m.
Exceptionally, when required, extended triple telescopic Kelly bars can reach to 70m below ground.

Boring Tools:

Boring tools are available to cope with different strata. The range includes general purpose augers, rock augers, boring bucket and Coring barrels amongst others.

Standard sizes of boring tools range from 600mm to 1800mm in increments of 150mm, and thereafter in increments of 300mm up to 3000mm. In suitable strata it is possible to construct a dry bore; in water-bearing strata it is often necessary to progress the bore under flooded conditions, i.e. water, betonies or polymer.

Concrete:

Concrete grades up to 40N/mm2 are common, and higher grades can be used where necessary.

Where the pile bore is dry, a hopper with a short tube is used to direct the concrete down the center of the reinforcement. Under flooded conditions a full-length tremmie pipe is used.

In both situations it is often practical to terminate the concrete at a low level. This is typically used for top-down construction of deep basements. In this situation it is possible for a steel column to be plunged into the head of the concrete.

Pile Load Bearing Capacity:

Depending on ground conditions pile loads of 20,000kN or greater can be safely carried on straight-shafted piles.

Special tools are available to form enlarged bases or "under-reamed" piles in suitable strata, typically stiff clays. Under-ream diameters are usually specified in increments of 150mm, and diameters of 5400mm or greater can be provided. Piles of 30,000kN

Application:

Can be used in a wide range of strata.

Where non-cohesive or water bearing soils exist temporary casings and/or betonies or polymer suspension can be used to progress the bores.

Diameters range from 600mm to 3000mm with pile depths up to 70 meters and maximum working loads of 20,000kN for straight-shafted piles or 30,000kN for under-reamed piles.

Advantages:

• High load capacity
• In stiff clays under-reamed bases can be formed to increase pile capacity
• Depending on ground conditions, pile concrete can be terminated below piling platform level.

Large diameter bored pile and large hole drilling:

In-situ concrete piles with diameters of 0.3 m up to 3.0 m are referred to as large-diameter bored piles. For their construction hollow spaces are made in the soil by means of drilling equipment.
Depending on soil conditions the excavation is carried out under the protection of a casing or without casing. Subsequently the drilling holes are filled with concrete; according to static requirements a rebar cage is placed before concreting.

Large-diameter bored piles:

Have a broad scope of application: They are used as

- Foundation elements for carrying vertical building loads .
- Foundation elements for retaining walls .
- Temporary building pit walls.
- Components of the final structure.

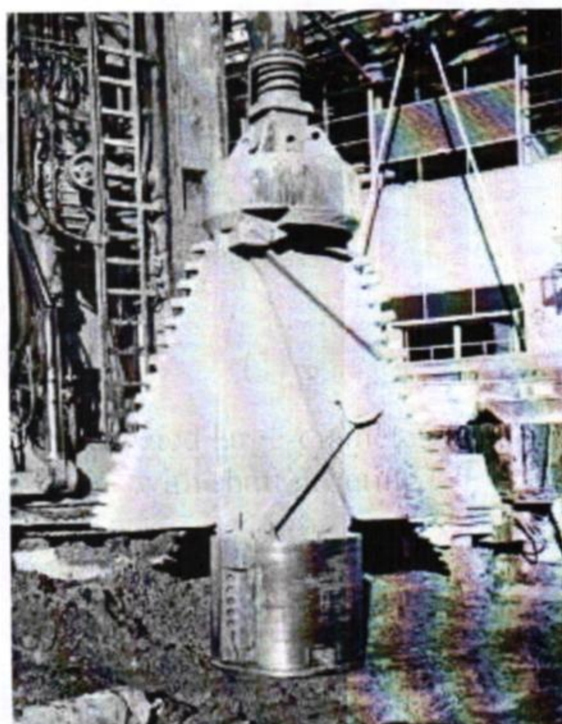
- Protection against uplift and for taking up tension loads.
- Slope security and Energy piles.

Large-hole drillings are applied for:

- The construction of large-diameter bored piles
- The construction of a soldier wall with lagging
- The construction of wells
- Removing obstacles in planned retaining wall or diaphragm wall lines
- Underground demolition of structures under groundwater
- Soil replacement in order to improve the soil.
- Soil replacement in order to remediate contamination.

(1)

Excavated large-diameter piles with shaft grouting



(2)
Pile base under reaming device
„System Hart fuss"1050 - 2500mm

Large-diameter bored piles as foundation elements for Carrying vertical building loads:

If the building ground for a spread foundation is not stable enough large diameter bored piles, which can transfer heavy building loads into deeper and more sustainable soil layers, are placed as foundation elements. The transfer of the loads into the soil takes place via end bearing under the pile bottom and via skin friction along the shaft area. The centering and transfer of the building loads into the piles usually take place via pile caps, pile cap beams, pile cap grids and via a continuous concrete slab.

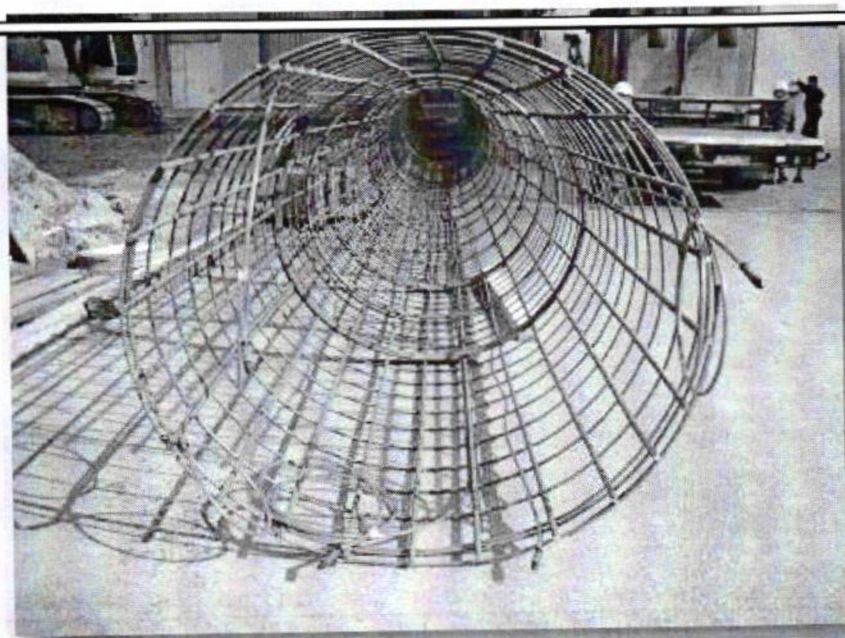
Measures to reduce settlements resp. to increase the pile load bearing capacity. With pile foundations next to built-up areas sensitive to settlement the adjacent settlements must be minimized by the deformation of the piles. The same applies if the settlement difference of adjacent foundations is restricted.

On the other hand the measures to increase the load bearing capacity can reduce the pile diameter and / or the pile length.

a) Increasing the pile diameter and / or increasing the pile length:

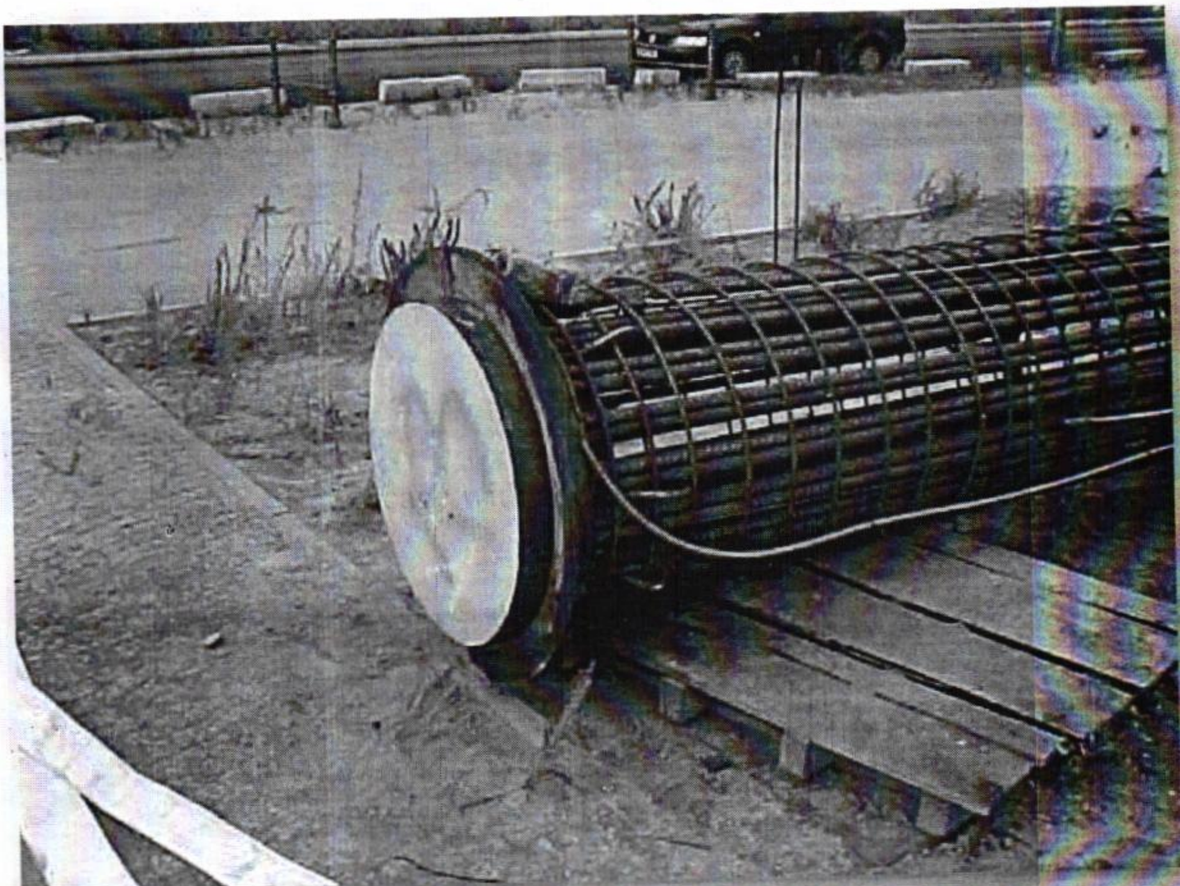
With the increase of the pile diameter the point pressure is reduced quadratically and the shaft friction rises linear to the increase. The Settlements can be reduced.

(1)
New silo 3, piles d = 180 cm



(2)

Rebar cage with additional injection tubes



(3)
Pressure bag for base grouting

(4)
Rebar cage with shaft grouting tubes

b) Shaft grouting:
With shaft grouting, the shaft friction is increased by additional Selective injections into the pile shaft area. For this purpose a thin plastic tube with valve is attached to the rebar cage for each injection point. When the pile concrete starts hardening the concrete cover of the valves is blasted with water high pressure and subsequently injected with cement slurry. On blasting the concrete cover of the rebar cage is partially shifted against the soil. The gap is „cured“by cement slurry, so that the concrete steel is secured against corrosion.

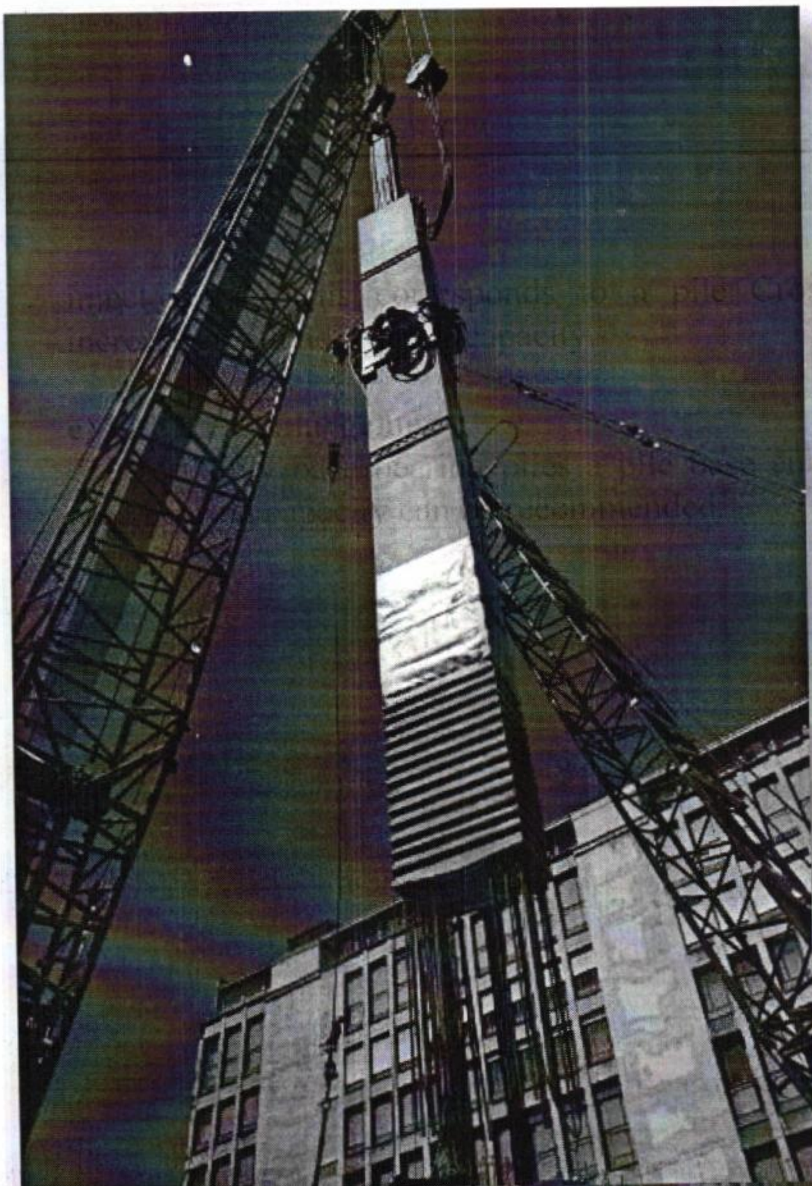
c) Pile base grouting:
During pile base grouting a pressure bag, a pressure bubble or a pressure box at the bottom of the rebar cage is injected with cement slurry by an injection tube after construction of the piles.
This measure extends the pile base injection facility and therefore a cause a preload of the soil under the pile base (anticipation of the settlement) before the building load is placed.

d) Additional shaft grouting:
For the rehabilitation of bored piles with a non-sufficient load bearing capacity it is possible to carry out injection drillings around the piles and to

inject grout. This corresponds to a pile Cross-section enlargement and increases the load bearing capacity.

e) Pile base enlargement:

For mainly base bearing piles a pile base enlargement for increasing the load bearing capacity can be recommended.



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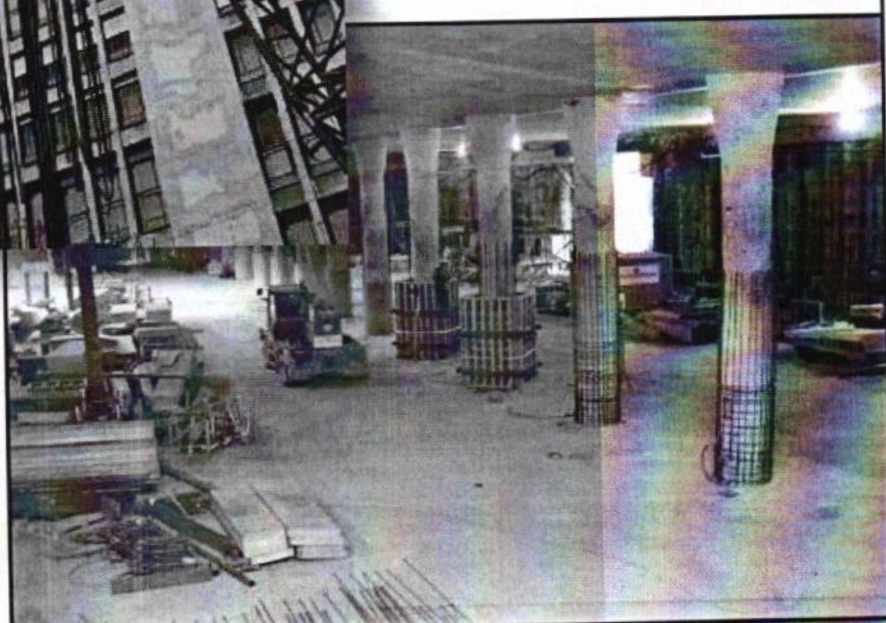
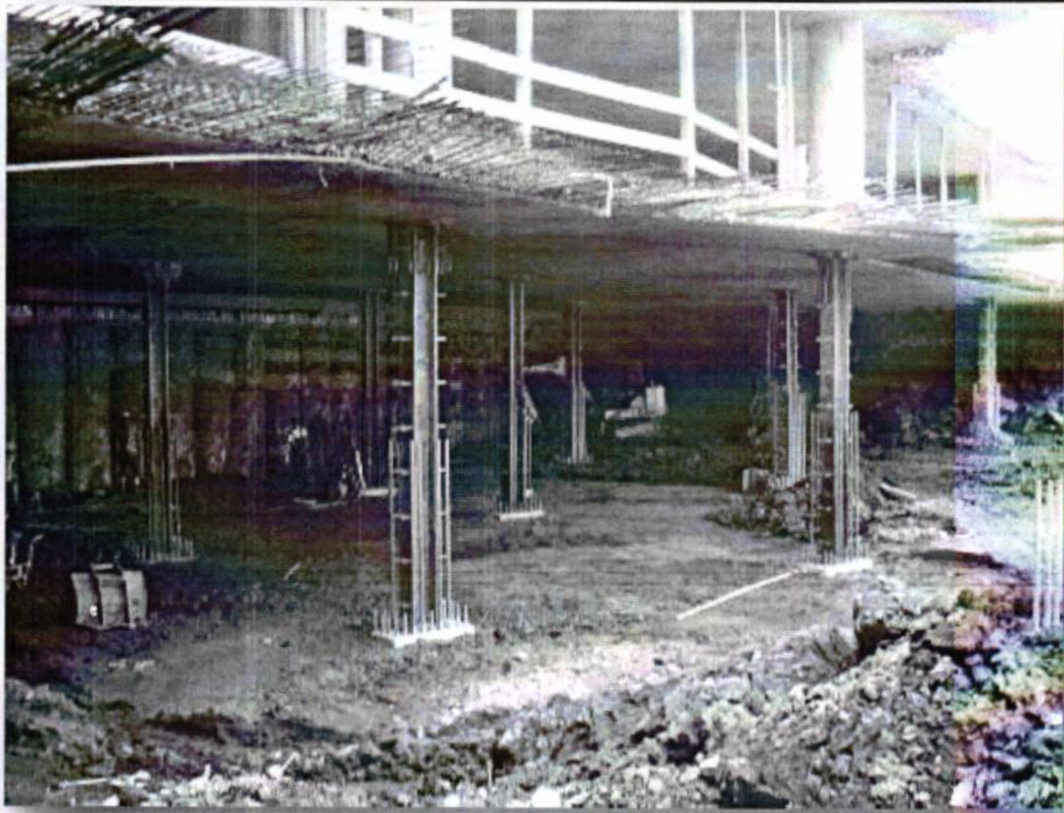


(2)
Primary columns
Under ground parking

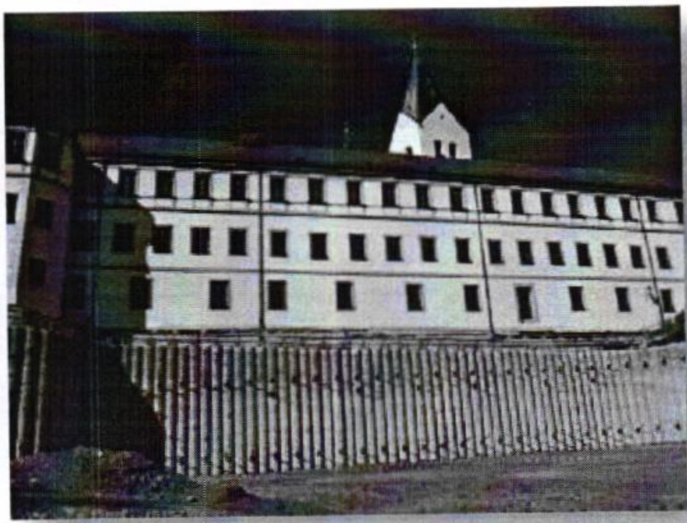


(3)
Primary columns

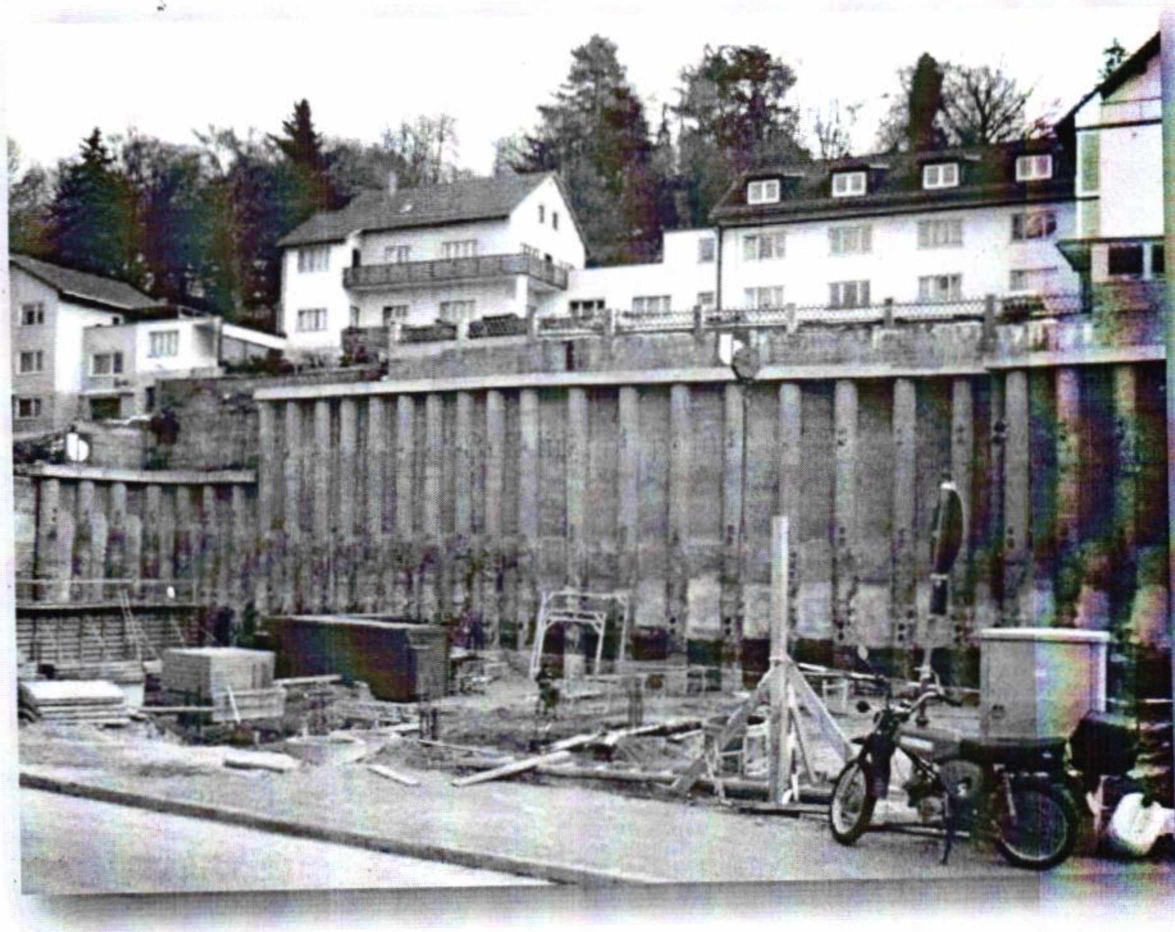
Primary columns for top-down construction:

The top-down construction method was first applied for underground railway construction. In these constructions the covers (Concrete slabs) were used to maintain the traffic on the building site and to ensure a continuous working flow, as excavations and construction, under the cover. Moreover the covers were used for the bracing of building pits. In recent years the top-down construction method has been applied more often for bracing large building pits. The construction period for building construction sites can be reduced by this technology as after completion of the top or cover slab (ground level slab) simultaneous upwards and downwards construction is possible.

Before construction of the top slab the bearings or supports must be provided. Usually the slabs rest at the building pit edge on the building pit boundary wall and on primary columns within the building pit. For the construction of the primary columns drillings are carried out up to the required maximum depth and concreted up to the planned final building pit bottom slab. Elements of steel profiles or precast concrete elements are integrated in the borehole with the exact required position and level. The remaining ring space within the drilling is filled with appropriate material.



(1)
Building pit piling



(2)
Building pit with contiguous pile wall

Large-diameter bored piles as foundation elements for retaining walls:

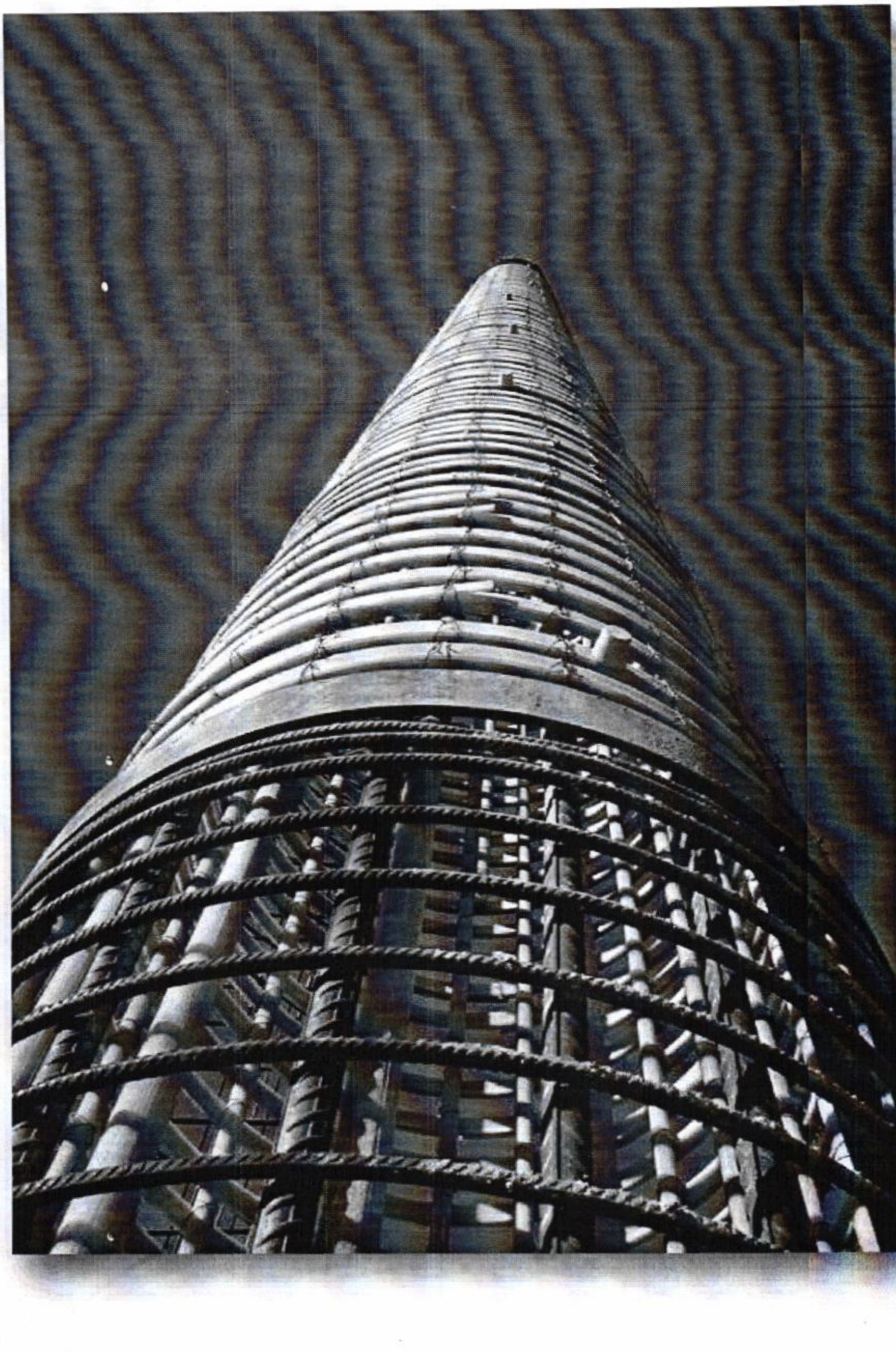
Building pits for the construction of foundations for spread founded retaining walls are sometimes very elaborate. Frequently due to adjacent building structures there is not sufficient space available for the construction of the spread foundation elements.
To solve this problem laterally soil supported large-diameter bored piles for the foundation of retaining walls are recommended.

Bored pile walls:

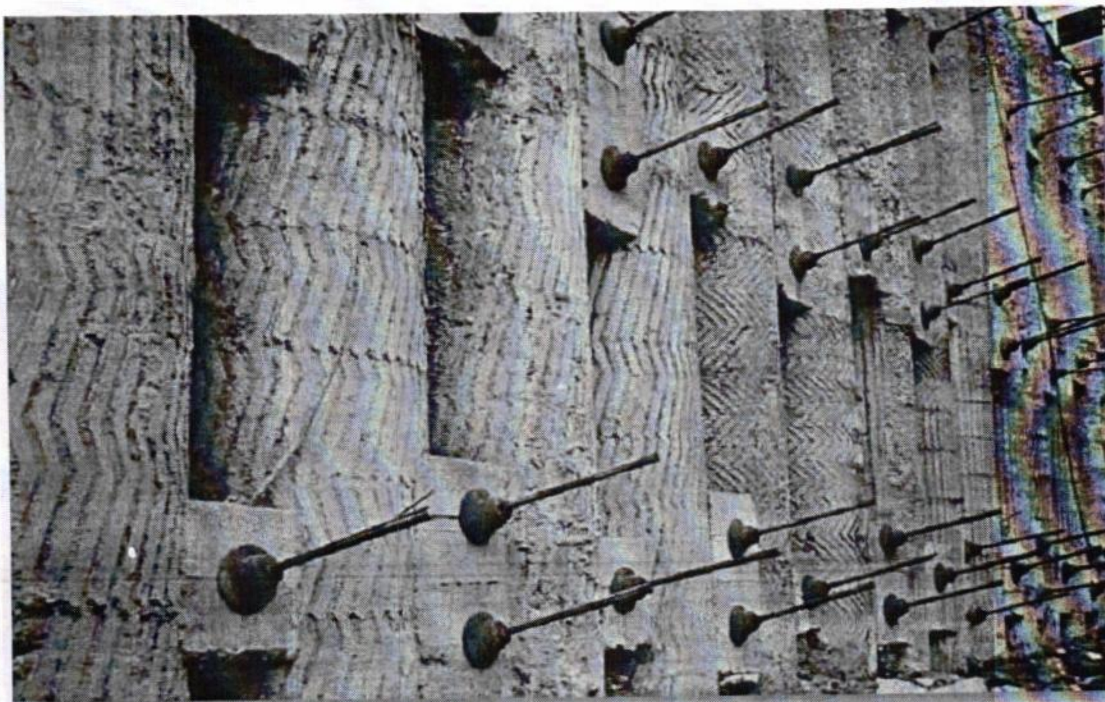
Bored pile walls are executed as single, contiguous or secant pile walls.
In the case of the secant-bored pile wall every second, third or fourth pile is reinforced as supporting secondary pile, the primary piles in between are without reinforcement.
In the case of the single and contiguous pile wall all piles are supporting and thus reinforced. The interspaces of the single bored pile wall are generally closed by shotcrete laggings, which can be drained.

Bored pile walls as temporary building pit walls:

Bored pile walls are used as temporary building pit walls for the construction of building pits. Because of the high stiffness, compared to soldier walls with lagging or sheet pile walls, it is called a building pit wall with low deformation. Bored pile walls are chosen when in the area of the building pit construction buildings or other structures are to be protected against settlements.

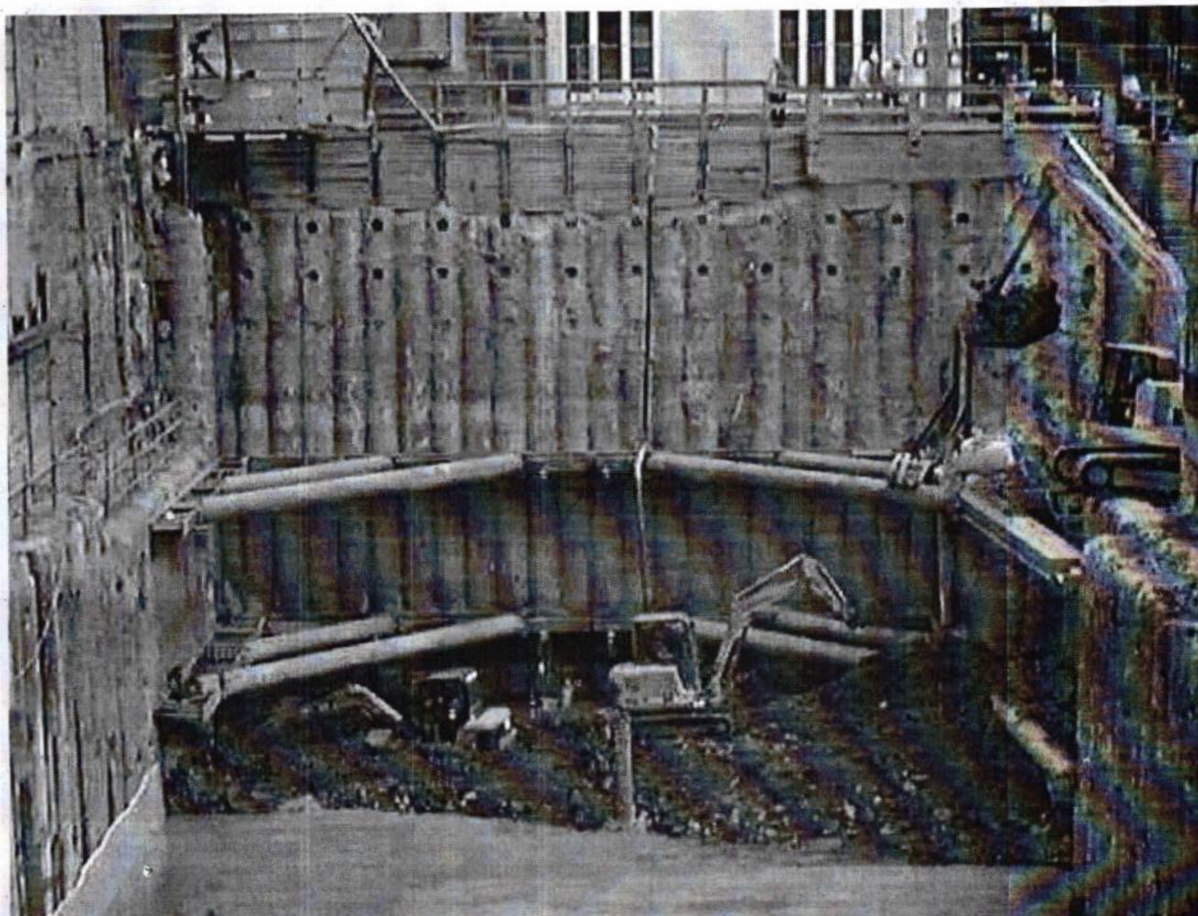


Rebar cage with glass fibre reinforcement for „spectacle"wall



(2)

Building pit with contiguous bored pile wal



(3)

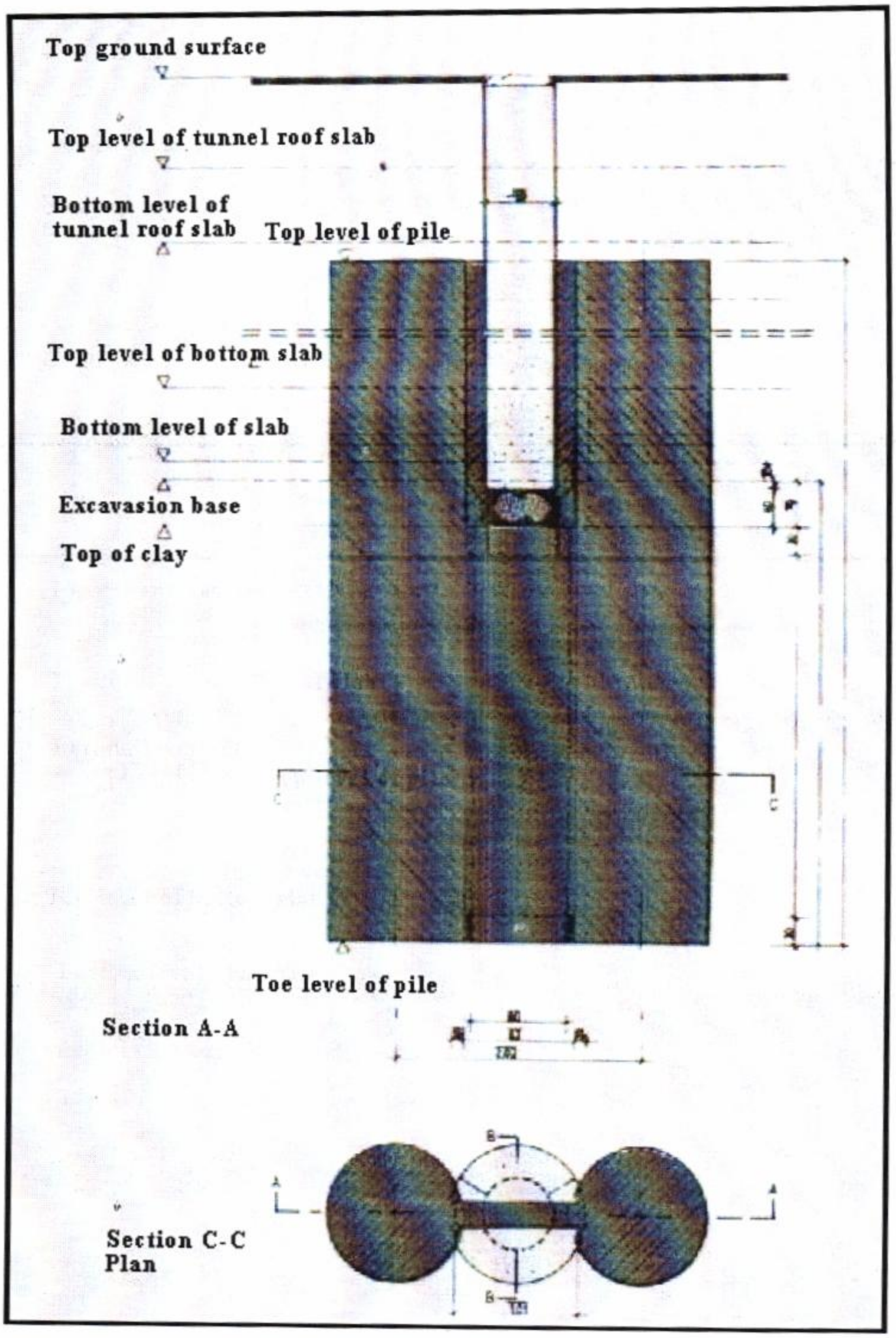
Building pit with secant bored pile wall with inserted beams

Secant bored pile walls are used for "watertight" building pits and apart from their function as building pit walls also used as impermeable walls. Therefore they must reach up to an artificial impermeable bottom slab or they should be socketed in a natural impermeable layer.

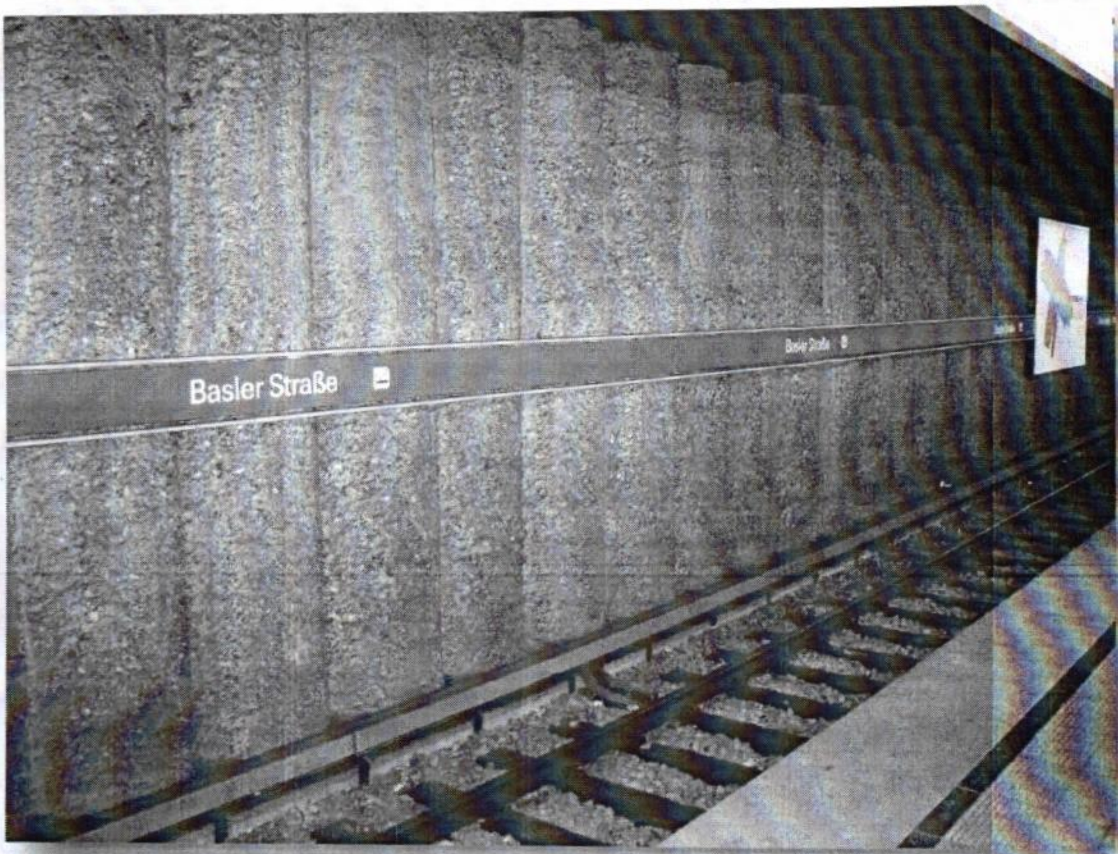
Impermeable walls of secant large-diameter piles have been executed by Bilfinger Berger as seepage barriers against seepage water contaminated with heavy metals and micro-elements. As sealing material a cement-free, very solid sealing compound was used, fully developed by Bilfinger Berger.

The upper 1 to 2 m of bored pile walls are often designed as walls with inserted beam support. In the case of pile construction this depth is executed as empty drilling and in the reinforced piles in the pile cap area steal beams are built in, which end at the ground surface. During excavation between the vertical beams a horizontal lagging of wood or shotcrete is carried out. This sheeting can be dismounted more easily for later activities like underground facilities.

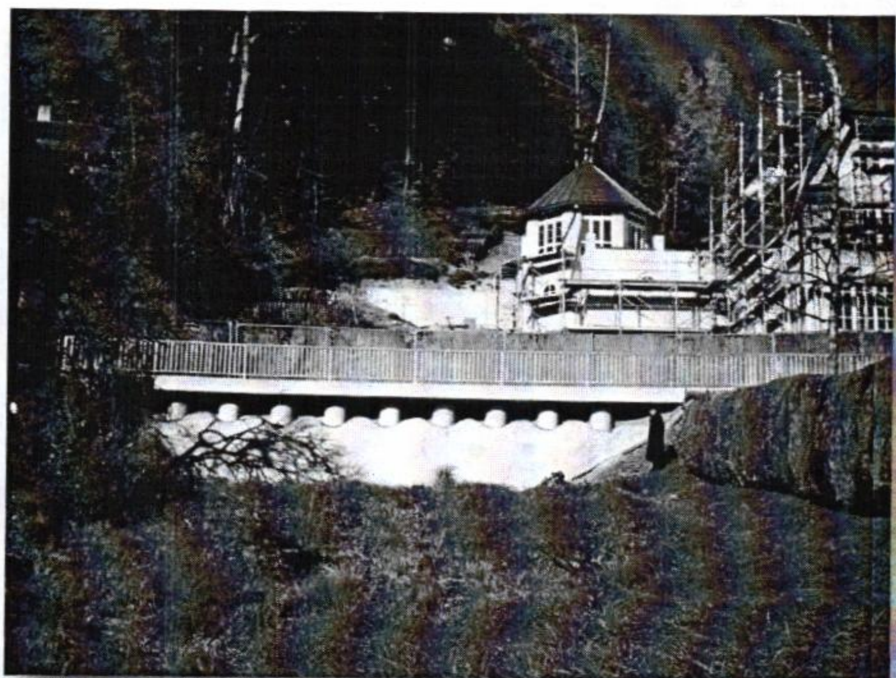
Bored pile walls, which are to be passed through by a tunnel boring machine (TBM), so called "spectacle walls" are reinforced with glass fibre reinforcement in the passage area (so called soft-eye construction). This reinforcement material is so brittle and easy to cut through that no damage to the TBM is to be expected.


Top ground surface
Top level of tunnel roof slab
Bottom level of
tunnel roof slab
Top level of pile
Top level of bottom slab
Bottom level of slab
Excavasion base
Top of clay
Toe level of pile
Section A-A
Section C-C
Plan

(1)



(2)
Secant bored pile wall as
component of the final construction



(3)
Contiguous pile wall as component of the final structure

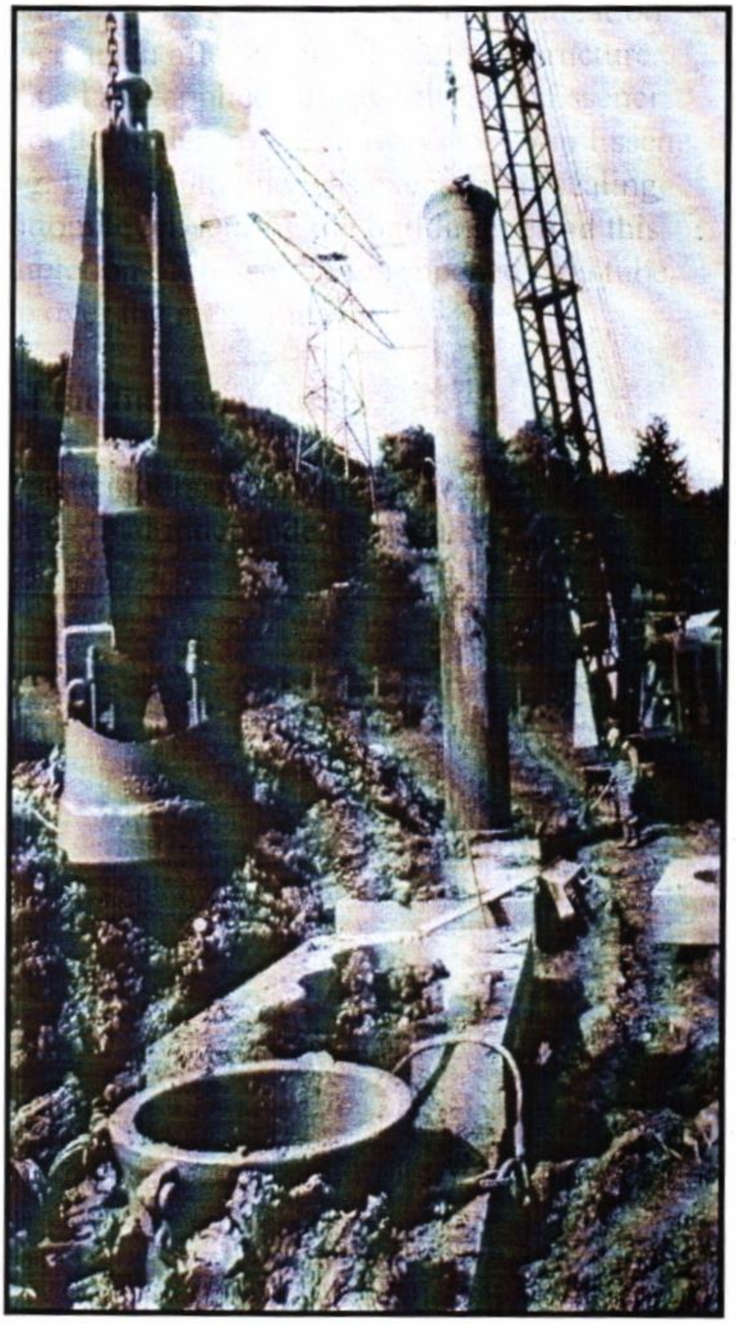
In many cases for "watertight"building pits a groundwater communication under the bottom slab must take place again after completion of the structure. To this end Bilfinger Berger developed and applied successfully the "Essener Dichtlamelle" (sealing membrane) for the underground railway, lot 25 in Essen as a replacement for partial freezing. Every 10th pile was executed as sealing membrane under the excavation bottom level. During the building period this membrane was sealed and after completion of the structure reopened by a tube set in concrete over the membrane.

Bored pile walls as components of the final structure:

Bored pile walls are not only used as temporary building pit walls but also as components of the final structure or as final independent structures.

In underground railway or underground parking construction a watertight concrete cover wall is generally carried out on the inside of the bored pile wall. During the construction period the pile wall has to transfer the earth and water pressure loads, but after completion only earth load - the water load is carried by the Concrete wall.

In Munich at several underground stations the secant bored pile wall is a component of the final structure as a visual supporting element.



(1)

Securing of slopes against slides by deep drain trenches

(2)
Construction tension pile

(3)
Large-diameter piles as foundation element for retaining walls as slope securing retaining wall

Large diameter piles against uplift and for transferring tension loads:

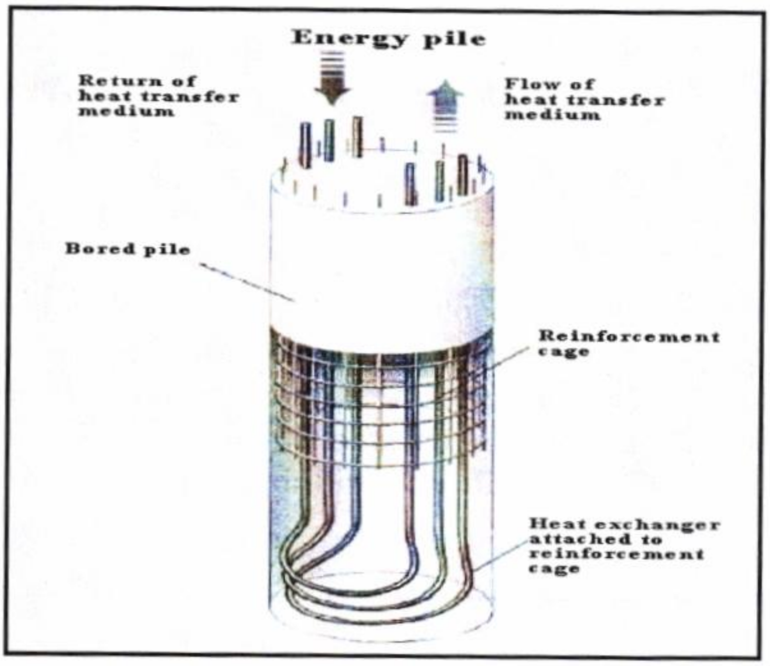
Large-diameter piles are often used as foundation elements for alternating loads (pressure and tension). These alternating loads occur due to the system on cantilevering hall or stadium roofs. In the case of pile foundation of high-rise buildings tension loads from uplift can appear during the construction period, if the dewatering for the building pit is turned off, the building pit is flooded and the weight of the building is not yet sufficient due to the degree of completion. In the final state these piles support like pressure piles.

Large-diameter piles as slope securing:

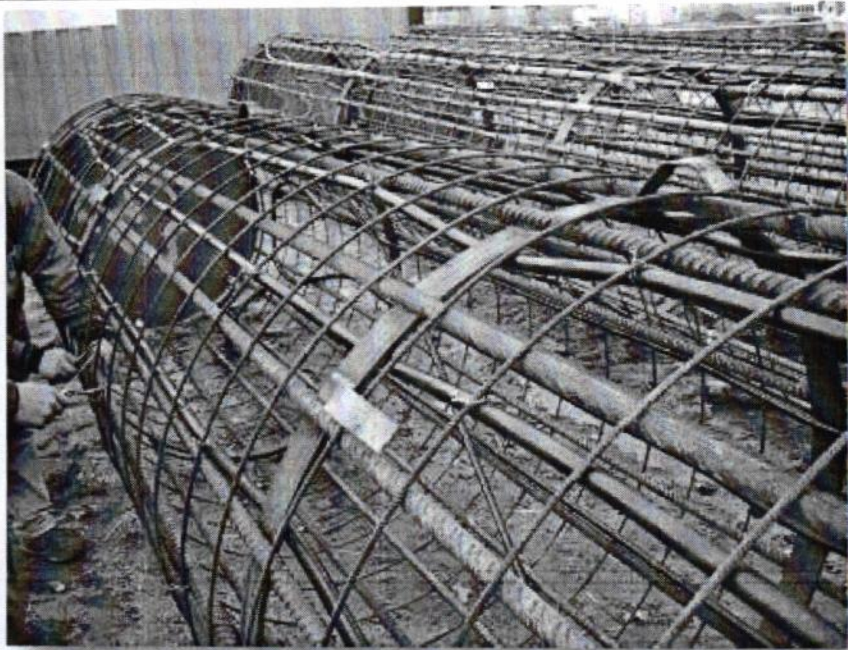
Large-diameter piles are applied for slope securing in different versions.
The stability is increased by active elements like:

- Bored pile walls as retaining walls.
- Bored pile walls as supporting shear walls.
- Bored piles for dowelling.

A further possibility for securing slopes against sliding exists in the construction of deep drain trenches. To this end large-diameter secant, tangential or contiguous drillings are carried out vertically to the main gradient of the slope, on the bottom slab, a bottom drain tube is installed for the effluent overflow and the drillings are filled with filter material.



(1)
Principle of an energy pile



(2)
View in a rebar cage for an energy pile

(3)
View in a rebar cage for an energy pile

Large-diameter piles as energy piles:

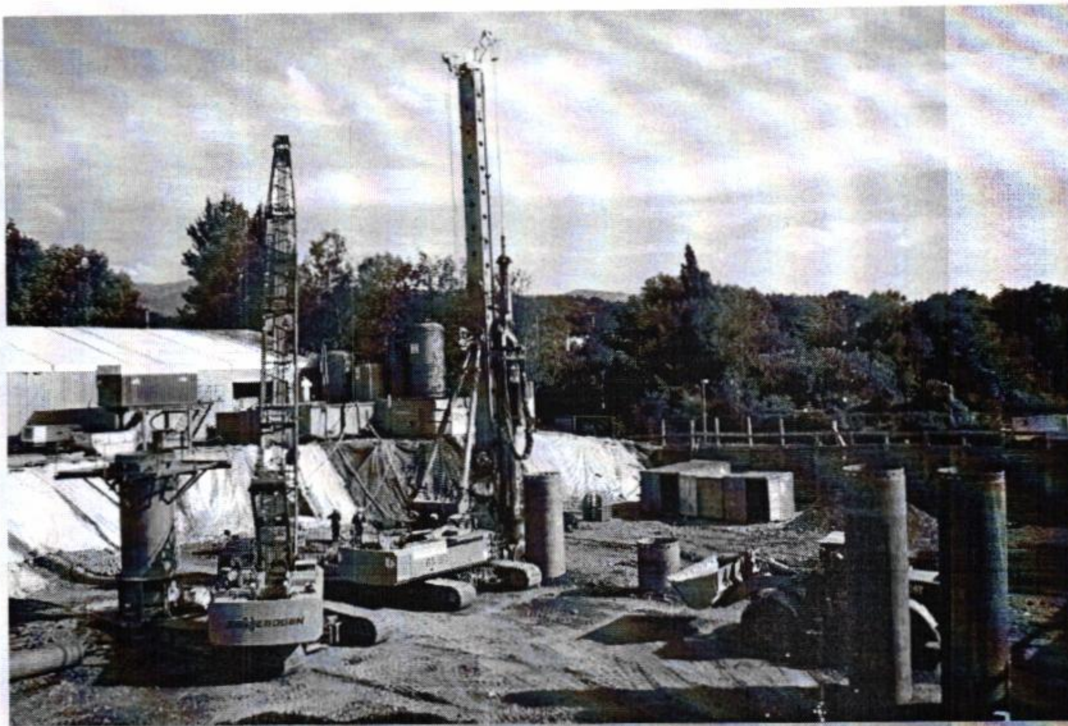
For new structures the regenerative usage of energy is growing incrementally. The heat of the earth close to the surface as part of the geothermal energy is used for the climate control of buildings.

The substratum is suited as a store for heat and cold, the required quantities depend on the time of day and year.

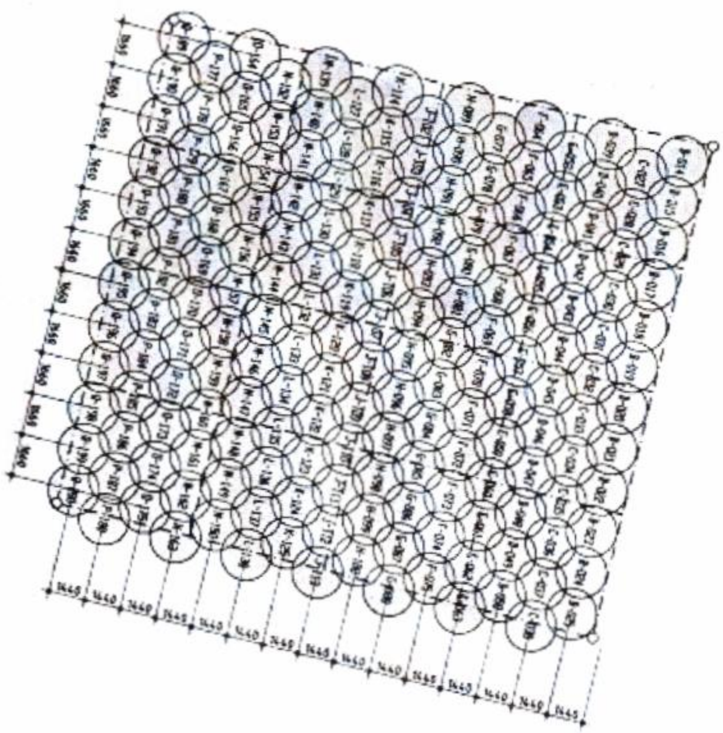
By using necessary static building parts (no additional parts required) such as in-situ piles of the foundation or building pit piles as energy piles the investment costs for the usage of energy can be kept low.

Heat exchanging pipes fixed to the rebar cage are installed in the in-situ piles. In these pipes a heat transfer medium flows for the temperature balance between the earth store and the interior rooms.

(1)
Site clearing traffic area in the Central area



(2)
Reconstruction



(3)
Site plan secant drillings for soil replacement

1. Introduction :

Pile foundations are the part of a heavy structure used to carry and transfer its load to the bearing ground located at some depth below ground surface. Depending upon various factors like nature of substrata, depth of ground water table, depth of stronger stratum, type and quantum of load to be supported etc., piles are designed. Pile testing is considered a fundamental part of pile foundation design. It is one of the most effective means of dealing with uncertainties that inevitably arise during the design and construction of piles.

In some places improvement in foundation practice has led to an increased reliance on bored cast-in-situ RC piles for supporting tall and heavily loaded structures and cross drainage structures. New cement plant at Kohat also comprised some heavily loaded structures and needed to be supported on piles. Before the construction of bored cast-in-situ working piles, four test piles were subjected to loading tests. This paper presents an analysis of pile load tests data collected from these four pile load tests on cast-in-situ bored piles.

Results of these pile load tests have been compared with the load carrying capacity of the pile computed by empirical relations proposed by different researchers. In addition seven different methods to interpret ultimate load from load/settlement relationship have been used with the purpose to select the method most suitable for local conditions. Similarly tip bearing and shaft resistance have been interpreted from load/settlement relationship. The percentage of load taken by piles along with slip needed to develop full mobilization of shaft friction, in local conditions, have been computed.

From the experience of load testing of four piles in hard clays in Pakistan, the authors have attempted to establish the pile design parameters for local conditions from back calculations of pile load tests results. This study aims to provide guidelines regarding the pile design parameters most appropriate for local conditions and the best procedure for estimating ultimate capacity from pile load tests in hard clays.

2. Pile Design Parameters :

Various field and laboratory tests have been carried out during the geotechnical investigations for the evaluation of subsurface conditions and the pile design parameters (un-drained cohesion Cu and adhesion factor) at the project site.

Figure 1 shows the variation of un-drained cohesion (Cu) at various depths at the project site. The un-drained Cohesion was determined through unconfined compression tests on undisturbed samples collected from the boreholes. There is a large scatter of the un-drained cohesion values at different depths at the project site; however, average values based on boreholes near the test pile have been taken.

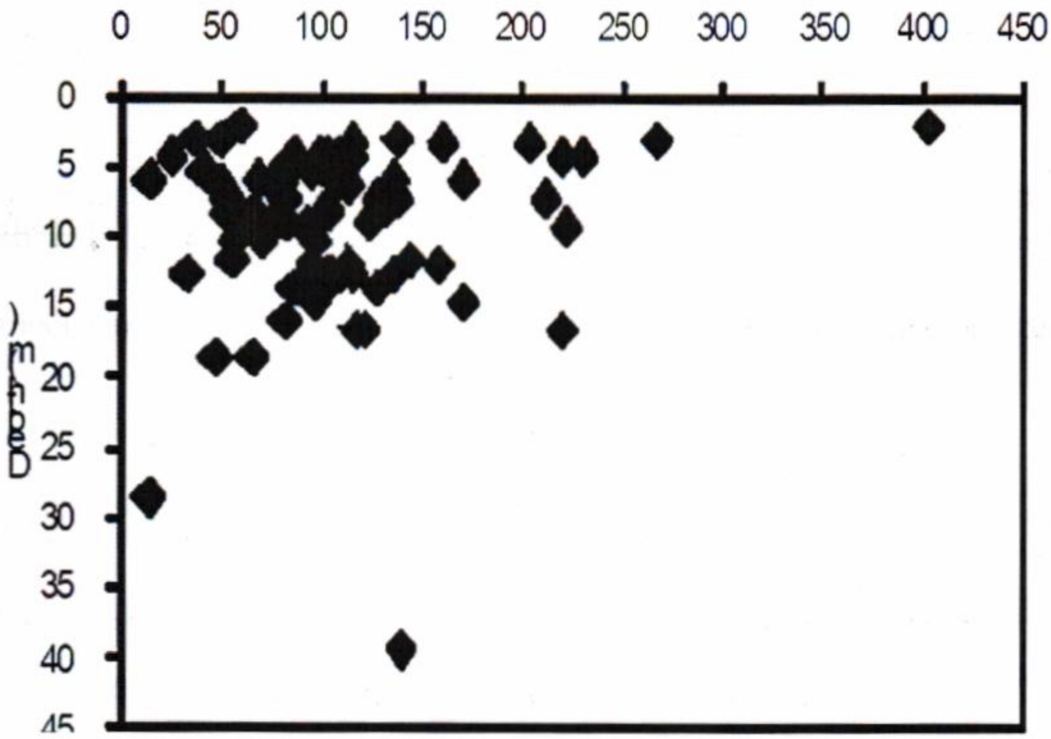


Figure 1: Variation of un-drained cohesion (Cu) with depth at the project site.

3. Pile Load Tests :

Four pile load tests were performed on piles of 660 mm and 760 mm diameters and of lengths varying from 20m to 47.5 m.

The reaction load was arranged through a system of jacking bearing against the dead load resting on a platform. The dead weight was supplied by piling soil filled plastic bags on the platform as shown through Plate 1. The platform was supported

on three wide flanged girder beams (reaction beams) placed side by side (and bolted together) over the jack. A hydraulic jack system comprising a 550 tons jack (Plate 2), pressure gauge, oil reservoir, pump (Plate 3) and piping was used in this test.

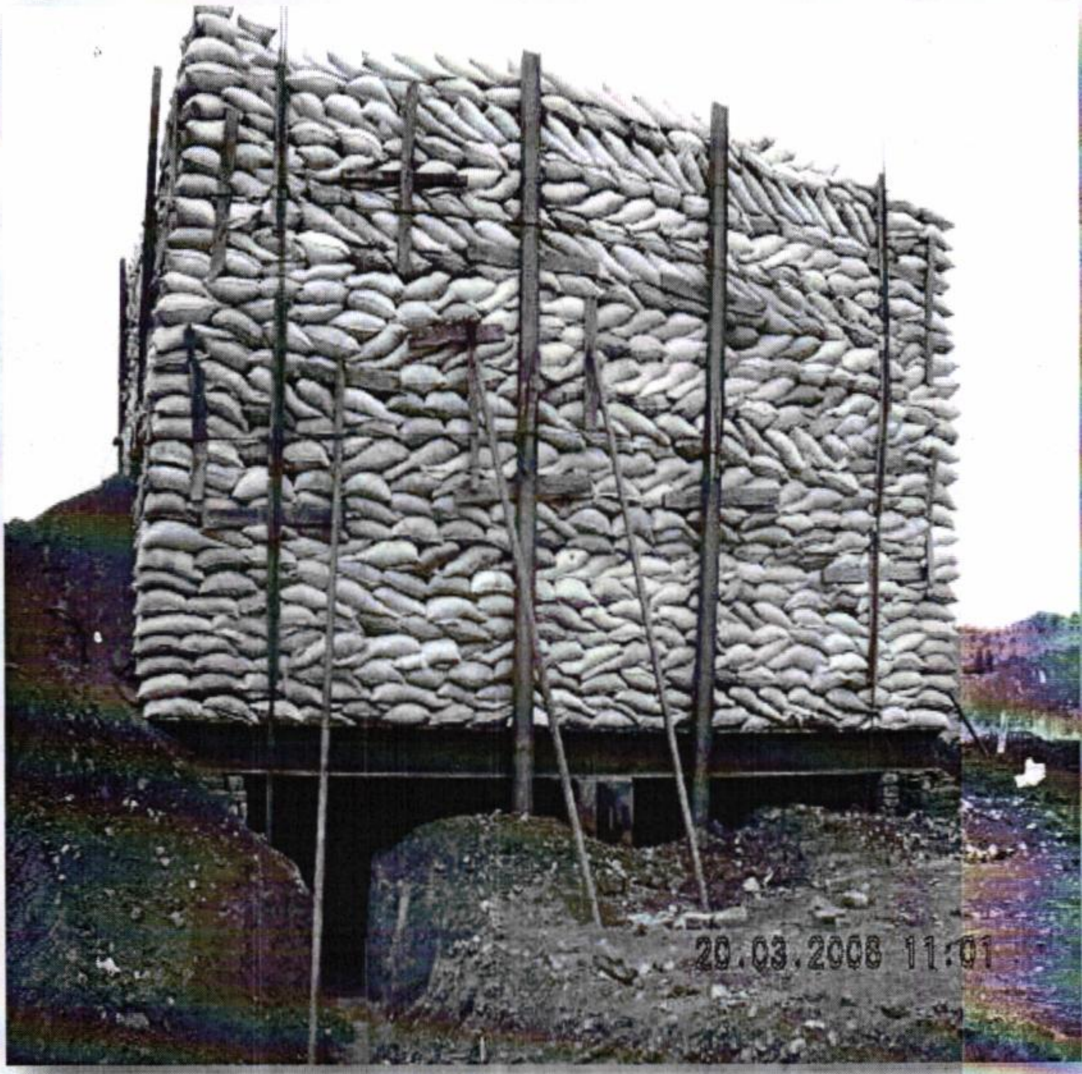


Plate 1: A view of soil filled bags on loading platform to obtain dead load reaction

Settlement of the piles was recorded by means of three settlement gauges capable of reading to 0.01 mm precision. These gauges were capable to record a total settlement of about 30 mm. The gauges were mounted on two reference I-beams. The stand for the gauges was bearing on the steel plate placed on the pile head (Plate 2). The reference beam supports were at a clear distance > 2.5 m from the test pile.

All piles except Test Pile No. 2 were loaded in one cycle. Each increment (about 25% of the design load) was maintained for a maximum period of two hours or when settlement rate was observed to be less than 0.25 mm per hour. The average of the three gauges gives settlement after each interval.

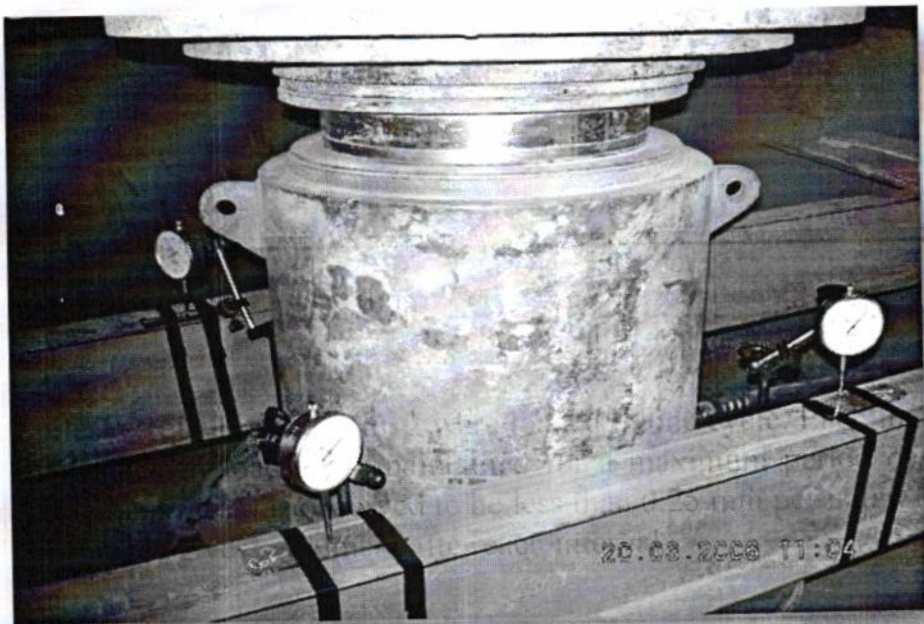


Plate 2: A view of the jack, settlement dial gauges, and reference beams used under the loading plat-form.

Plate 3: Oil reservoir pump and pressure gaug

Test results reveal that piles of 660 mm diameter settled more than 12 mm while settlement of the 760 mm diameter piles was observed less than 12 mm.

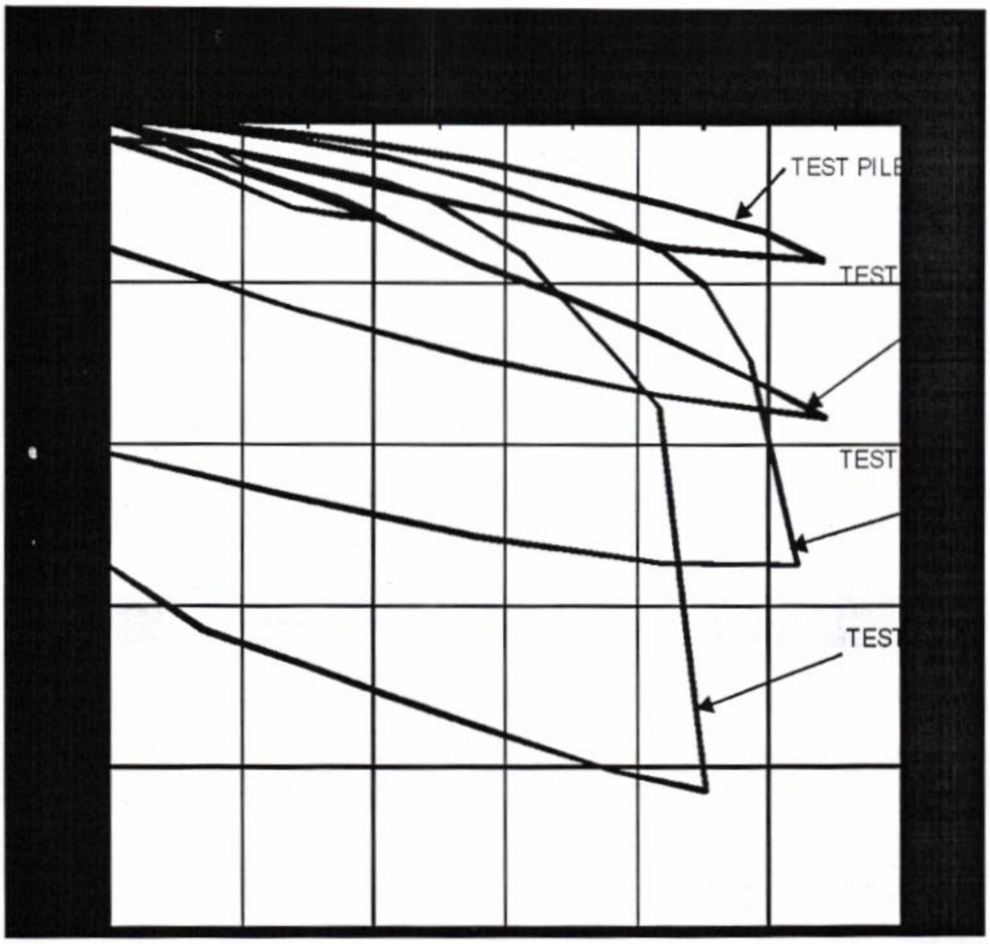


Figure 2: Load-settlement graphs for four pile load tests in clay.

Using 90% Hansen, 1963 method the extension of curve through a trend line of 5th polynomial, does not provide results for test pile no.3. 280% Hansen, 1963 methods does not provide results for settlement < 12mm 3Load Settlement data obtained from Test pile no. 3 does not provides results for Chin method

- All the methods provide ultimate load more than the theoretical pile capacity determined using pile design parameters, except for pile No. 4 using Chin method.

- 80% Hansen [7] method cannot predict ultimate load for pile settlement <12mm (Test pile Nos. 3 and 4)

- 90% Hansen [7] method does not predict ultimate load for test pile No. 3, however, for test pile No. 4, the ultimate load has been predicted by extending the load settlement curve by 5th polynomial that resembles typical load settlement curve.
- Limit value [2] method has been used to predict ultimate load for test pile

Nos. 3 and 4 (pile settlement < 12mm) by extending the load settlement curve by 5th polynomial that bear a resemblance to the typical load settlement curve.

▪ Chin method [3] provides ultimate load higher than theoretical pile capacity for pile settlement > 12 mm but approximately equal to theoretical pile capacity for test pile No. 4, having settlement < 12 mm. On the other hand, load settlement data from test pile No. 3 are insufficient to predict ultimate load.

Based on the above discussion and findings, it is recommended to estimate the ultimate load in hard clays by taking the average of loads obtained from intercept of two tangents, point of change of slope and 6 mm net settlement methods.

Interpreting tip bearing (Qb) and shaft resistance (Qs) components by Van Wheele (1957) method indicates that at failure about 7 to 20 % of load was taken by the piles at the base, and that up to 80 to 93 % of load was resisted along the shaft.

Based on the load test results, study of the relevant literature and different techniques to find ultimate load from pile load test results.

There is a great variation of pile design parameters for the piles loaded to settlement > 12mm and the piles loaded to settlement < 12mm. Figure 4 presents the variation of obtained from back calculation of pile load tests by different theoretical methods.

For test piles settling >12 mm, the value of from back calculations is 16 %, 44%, 33% and 40 % higher than those recommended by [4, 5, 6, 7] respectively. Therefore, the value of should be increased to the above percentage for theoretical estimation of ultimate pile capacity.

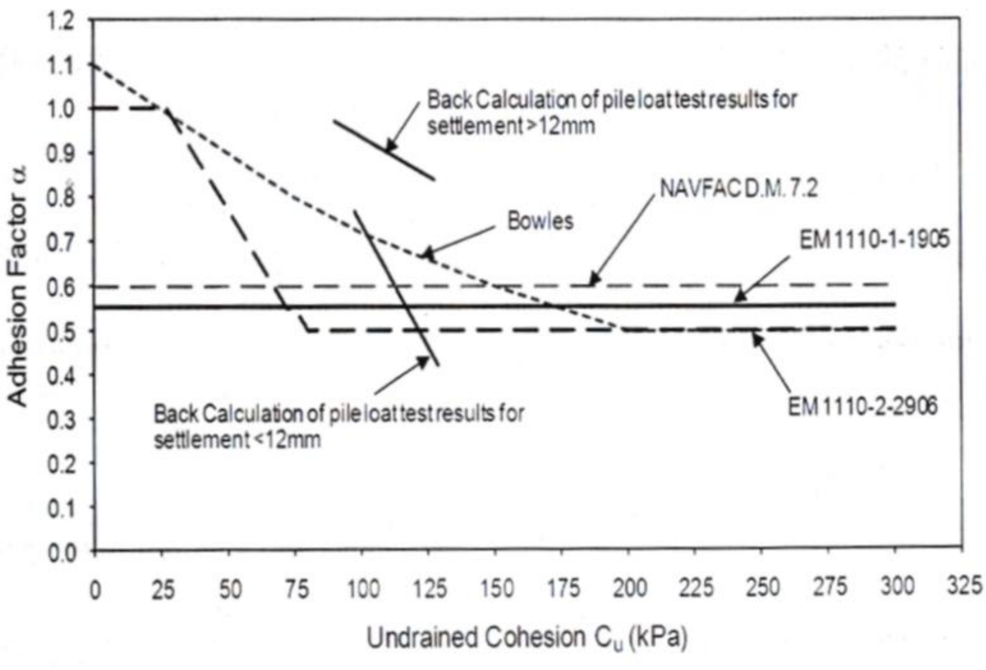


Figure 4: Relationship between adhesion factor and un-drained cohesion Cu. (From sources noted)

For test pile No. 3 settling < 12 mm, the value of a 6 %, 34%, 21% and 28 % higher than that recommended by the above four sources respectively. On the other hand, for test pile No. 4 for which net settlement is nearly equal to zero, the value of is 54 %, 19%, 43% and 31 % lower than that recommended by the above four sources respectively.

Figure 4 also shows that the gradient of line for pile settlement > 12 mm is mild as compare to gradient of line for test pi settlement < 12 mm which is steep.

4. Conclusions :

The ultimate loads determined from load/settlement curves of load tests by methods are higher than theoretically computed capacities for all the piles except the pile for which net settlement is nearly equal to zero. The best method of estimating ultimate load from pile load test results is to use the average value of ultimate loads obtained from three methods (i.e. two tangents method, load corresponding to 6 mm net settlement and load at point of change of slope of load settlement curve).

For bored pile at failure (for pile settlement > 12 mm) about 8 % of load is resisted by the pile at the base, and that up to 92 % of load is resisted along the shaft. The pile design parameters obtained from back calculations of pile load test data for piles loaded to settlement > 12 mm are entirely different from piles loaded to settlement < 12mm.

For our local conditions, the value of in current practice is on conservative side, and should be increased to the recommended trend as shown in Figure 4 for theoretical estimation of ultimate pile capacity. The above conclusions are based on load test data on four piles only. There is a need to include several more pile load tests data in order to validate the above conclusions and express them in more general terms.

Chapter 3

EXPERIMENTAL AND NUMERICAL ANALYSES OF BORED PILE FOUNDATIONS IN A TROPICAL SOIL

1. Introduction :

A numerical analysis was carried out with some of the piles, in order to simulate there in situ behavior with the existing (lab and field) data. Two types of piles were selected for this particular analysis, i.e., a mechanically bored and a "root" type cast-in-place pile. The numerical analysis was done with a semi analytical procedure. This software computes the settlement and the distribution of the normal force inside the pile and actual shear resistance at any depth in the pile's shaft. The deformation variant of the solution was selected. The pile-soil interface function is modeled using nonlinear soil springs. Nevertheless, the shear forces are limited to the skin friction of the pile/soil interface. The value of the normal stress in this same interface is a function of the geostatic earth pressure and it can be user modified. Besides, the shear forces strongly depend on the friction behavior of the soil/pile interface, which on the other hand is affected by field construction (displacement, non-displacement, etc.) techniques used with the distinct piles. In this particular case, the known analytical solutions of layered sub soils for the shear response of the soil were adopted herein. These solutions are related to the Young modulus and Poisson's ratio of the soil, and the depth of the influence zone around the pile. This zone varies from one to two and half diameters around the pile, being variable during the analysis (it increases with the increase of load on top of the pile). Thus, the in situ experimental data of the field loading tests were used, together with the lab results from triaxial K0 tests on this same soil type, to numerically assess the behavior of two of the tested piles.

2. Method of analysis :

The solution was developed for a layered sub-soil. The pile is discretized into a finite number of cylindrical bar elements, and the soil-pile interface is concentrated in the nodal points. The ultimate shear force in the node is obtained from the formula given below:

$$Tk\,,\lim\, rk = 2\pi\,\tau\, lk\; k\,,\lim$$

where rk is the radius of the pile in the k node,
lk is the length of the shear influence around the k node,
and $\tau\, k$, lim is the ultimate shear strength (skin friction), being proportional to the lateral earth pressure in the soil-pile interface.

3. Experimental Site and Field Testing :

The research site is located in the city of Brasília, the Brazilian capital, which is encompassed by the Federal District of Brazil. Within this district, in the center of this country, it is common the occurrence of extensive areas (more than 80 % of the total area) covered by a weathered laterite of the tertiary-quaternary age. This "latosol" has been extensively subjected to a leaching process and it presents a variable thickness throughout the District, varying from few centimeters to around 40 meters. It is basically a red residual soil developed in humid, tropical and subtropical regions of good drainage. It is leached of silica and contains concentrations particularly of iron oxides and hydroxides and aluminum hydroxides. It also has a predominance of the clay mineral caulinite and, in localized points of the Federal District, it overlays a saprolitic/residual soil with a strong anisotropic mechanical behavior and high standard penetration resistance, which is originated from a weathered slate, a typical parent rock of the region.

The superficial latosol has a dark reddish coloration, and displays a much lower resistance and a much higher permeability than the bottom saprolitic/residual soil. The studied latosol constitutes into "collapsible" sandy clay with traces of silt, with a high void ratio and coefficient of collapse. Its coefficient of permeability is also high for typical clay, being close to those found for fine to silt sands. This soil is the so-called "porous" clay of Brasília, and is the predominant soil of the experimental research site of the University of Brasília.

In this site several deep foundations were constructed under differing construction techniques, all of them "typical" in the city of Brasília. Herein, solely the mechanically bored and the "root" cast-in-place piles were analyzed. For further information on this particular site and the details of the experimental field loading tests in these and other piles .

In order to obtain the experimental data it was required the establishment of vertical loading tests on well-instrumented piles founded in this site. This was accomplished with the cooperation of local contractors and the aid of the University of Brasília post-graduate students and staff. All the tests were done in accordance to the recommendations put forward by the Brazilian, and in the majority of the experiments they consisted of slow maintained field loading tests. Besides, the tests were performed in loading intervals of 20 % of the working load (which had an average estimated value of 180 (kN) for all piles), up to failure. The piles were subsequently unloaded in approximate 4 intervals. These load tests adopted a reaction frame and "reaction" piles 4 m apart. Both the top foundation block and the reaction frame were monitored for tilting and vertical displacements, by using 0.01 (mm) precision dial gauges. A 1000 (kN) hydraulic jack was used in conjunction with a 100 (N)

precision load cell and the tests were carried out with the soil in its natural moisture content (in the range of 30's %).

The following piles were tested and numerically analyzed:

• One root type (injected, cast-in-place) pile: Also known a"micropile", it was constructed by adopting an injection pressure of 200 kPa during the formation of the mortar shaft, and it is defined herein as R2. This pile was constructed with a specially devised drill rig, which operated hydraulically. The soil was excavated by a continuous and static introduction of a rotating casing with pressurized water. The water "washed out" the generated mud in front of this casing, opening a small annular gap between the casing and the excavated hole. Once drilling was finished, the interior of the casing was cleaned up and the rebar's were introduced. Mortar was then poured inside the casing until it was filled. The top 5 of the casing was then connected to an air pressurizing system, and air pressure was applied to the inner fluid mortar. By simultaneously applying air pressure and lifting up the casing, it was possible to form the corrugated pile's shaft (for the piles with injection pressure). This operation was done in sequence, continuously filling up the remaining casing with fluid mortar, thus leading at the end to a pile with an approximate length of 8 m and final average dia. of ≈ 25 cm.

• One mechanically screwed (or bored cast-in-place) pile: Defined as MSP0, it was constructed with concrete being poured just after the soil excavation ("0" means just after excavation). Using a continuous hollow flight auger, which was introduced into the soil by rotation, excavated this pile. The hydraulic mechanical auger was assembled in the back part of a truck specially devised for this type of work. No soil was removed during auger introduction, and, after the final depth was reached, the auger was withdrawn leaving a freshly excavated hole. The designed rebar were then introduced and using the transportable service of a local concrete company promptly poured the concrete. This pile had a length of ≈ 8 m and diameter of ≈ 30 cm.

4. Numerical Analysis and Comparisons :

In order to pursue the numerical analyses with the GEO 4 software product, some material characteristics of the (unique) soil layer were adopted herein. However, it shall be pointed out that the "true" soil mechanical and deformation characteristics are variable, since it is tropical clay subjected to leaching, linearization and weathering process, hence, with variable values along the site.

Therefore, in order to employ the numerical Pile GEO4 procedure some average values had to be chosen for the soil characteristics. These values are given below:

- drained friction angle 30 degrees.
- Cohesion 14 (kPa);
- Specific weight 18 (kNm-3);
- Young's modulus 8 (MPa);
- Coefficient of earth pressure in rest 0.49.

The comparison of results between field and numerical analyses, in terms of the load-settlement curve of the pile MSP0, is given in Figure 3. Figure 4 presents this same comparison, but with the use of a Young modulus of the soil of 16 (Mpa). This latter value is more close to the value proposed by [5] during their backanalyses with this same pile (but by using another software program & theoretical approach).

In general terms, it is observed that the numerical procedure (& theoretical model) proposed and adopted herein was able to capture reasonably well the field behavior of the tested piles. This procedure has, therefore, a very high potential for a practical use in the foundation area, in special in the design process of distinct foundation types. The simplicity of the input parameters, and the high accuracy (in geotechnical terms) of the obtained results, corroborates such affirmation.

Conclusions :

This work presented a simple exercise in which a numerical prediction of the field behavior of bored, cast-in-place, piles founded in tropical collapsible Brazilian clay was carried out. It has briefly presented the general lines of the numerical tool adopted, and it has discussed the results, which were presented in terms of load deflection curves. The few presented results already demonstrate that the procedure advocated herein has a high potential for usage in practical engineering applications, in special to those related to the design of deep foundations in conventional or "non -conventional" (tropical) soils. Nevertheless, more research is still necessary (and is under way) in this area.

Chapter 4

Numerical Analyses of Load Tests on Bored Piles

Eurref is approximately 3 times higher than Eoedref. So κ* must be 3 times smaller than λ*.

All FE calculations were performed with an axisymmetric mesh. The radius of the domain is 9 m and the height is 15 m. 6-noded triangular elements with a second order interpolation for displacements and three Gauss points for the numerical integration were used. Analyses with interface elements as well as analyses without interface elements were carried out. Each calculation is divided into 9 phases. In the first phase the initial soil stresses are generated. Then the pile is wished in place by changing the parameter sets in the elements that represent the pile. The pile weight is larger than the soil weight so that the state of stress is slightly changed. This simple approach is often used to model the installation process of a bored pile. Katzenbach & Arslan (1995) showed that the change of the in-situ stress state next to the pile shaft is only marginal while installing a bored pile with a casing. In phase 3 the displacement were set to zero and the loading begins. The loading is simulated by a prescribed displacement of 2, 4, 8, 16, 30, 70 and 130 mm at the pile head. When using interface elements, the strength of these elements is not reduced (Rinter = 1). Moormann (2002) performed experiments with a similar soil and rough concrete surfaces typical for bored piles. For these configurations he found that there is no considerable reduce for both the friction and the cohesion.

3.1 Mesh dependency :

To study the mesh dependency 6 analyses were performed, i.e. three different calculations with interface elements and three without interface elements – one with a very fine, one with a medium course and one with a very coarse mesh. The very fine and the very coarse mesh are shown in Figure 3. The results are illustrated for the MC model in Figure 4 and in Figure 5 for the HS model. In the figures the load settlement curves for the base resistance Rb, for the shaft resistance Rs and for the to resistance R are given. The curves for the calculations with interface elements are mapped in the left graph and the curves for the calculations without interface elements are mapped in the right graph. For clarity only the results for the MC and the HS model for the very fine (vf) and for the very coarse (vc) mesh are shown. Similar results were found with the SS model. These results are not shown here due to the lack of space.

One can easily see the advantage of interface elements. The difference between calculations with a very fine and a very coarse mesh especially for the shaft resistance negligible. Only for the base resistance one can observe a small difference, because for the coarse mesh there is only one element

underneath the pile base. For calculations without interface elements this difference is much bigger, but still not as big as for the shaft resistance. For a pile head settlement of 130 mm (0.1 D) Rs is for the very coarse mesh four times higher than for the very fine mesh.

3.2 Results of the different models for the load test :

The results for the MC, the SS and the HS model are now compared. Interface elements were used in all analyses. Figure 6 shows the load settlement curves for the base resistance Rb and the total resistance R in the left chart and for the shaft resistance Rs in the right chart for the three models. Comparing the curves of the base resistance it is obvious that all curves have the same shape. But the MC model behaves stiffer than the HS model and the SS model behaves even stiffer than the MC model. This is due to the different formulation of the oedometer stiffness in the different models. In the MC model a constant stiffness is used, while in the SS and the HS model a stress dependent stiffness is included. This stress dependency is linear for the SS model ($m = 1$) and parabolic for the HS model ($m = 0.5$). As a consequence the stiffness of the SS and the HS model is the same for a vertical stress of 100 kPa. For higher stresses Eoed is higher in the SS model than in the HS model. As a consequence one gets a higher base resistance for the same settlement with the SS model, as seen in Figure 6. Considering the significant different models it would seem that for the base resistance the choice of a constitutive model is not as important as the right choice of the stiffness.

Comparing the shaft resistance one observes also differences in the results of the three models. The shaft resistance for the MC and the SS model is increasing more or less linear up to a peak. Whereas the peak value of Rs is higher for the SS model. In the HS model there is no linear increase. The shape of the curve is hyperbolic and the peak value is even higher than for the SS model. After the peak the shaft resistance is decreasing a bit for all models. Above all the difference in the peak value of the shaft resistance and the hyperbolic shape for the HS model is surprising. In all calculations interface elements were used. The behavior of interfaces is described with an elastic-plastic model in the FE-code used in the calculations. To distinguish between elastic and plastic behavior the Coulomb criterion.

Where ø is the friction angle and ci the cohesion of the interface element calculated from the strength of the soil using Equation 6. For the MC as well as the SS and the HS model the strength is the same, so also øi and ci are the same. Thus only the normal stress σn at the pile shaft could result in different peak values of the shaft resistance. One could explain this, when comparing

the horizontal stresses in two points next to the pile shaft. In Figure 7 the development of the horizontal stresses at a depth of 2.5 and 7.5 m over the pile head displacement is shown. It is obvious that σxx for the HS model is increasing during the load test up to a settlement of approximately 16 mm much more than for the SS model, while σxx for the MC model is nearly constant. This increase is due to the deviator and volumetric hardening in the HS model. Both the SS and the HS model account for over consolidation. In these analyses OCR is equal to 1, because of missing data on soil testing. In the literature only an increased value for K0NC was found and also used in the calculations. Therefore one gets directly plastic strains when the loading of the pile begins. For very small settlements (1 and 2 mm) there are for both models only stress points on the cap. With increasing settlements the deviatory hardening in the HS model starts which is not included in the SS model. So one gets in the HS model deviator and volumetric hardening next to the pile shaft, whereas in the SS model one gets only the volumetric hardening and in the MC model there is no hardening at all.

The difference in the shape of the load settlement curve for the shaft resistance is also referred to the shear hardening in the HS model. That there is pure shear at the pile shaft, like in a direct shear apparatus, one can see in the rotation of the principal stresses and in the vertical and horizontal stresses at the shaft. After a few millimeters of settlement the vertical stress near the pile shaft is exactly as big as the horizontal stress. Thus it is important to model the plastic shear strains before failure in a correct way, which is only possible with the HS model. Therefore the MC and the SS model behave stiffer in the beginning of the load test than the HS model (peak of Rs at a settlement of 4 to 8 mm for MC and SS and at a settlement of 16 mm for HS).

This is also visible when looking on the settlements of the soil next to the pile shaft shows the settlements in the soil at a depth of 0.5 m and 5 m for a pile head displacement of 70 mm. It is obvious that the soil in the calculation with the MC and the SS model behaves much stiffer than with the HS model. At a depth of 0.5 m the pile settles nearly 70 mm while the soil next to the pile settles only 7 mm for the MC model, 18 mm for the SS model and 61 mm for the HS model. Thus in the interface there is a relative settlement of 63 mm for MC, 52 mm for SS and only 9 mm for HS. The difference between MC and SS can be explained by the higher shear modulus of the interface element. This shear modulus in the HS model is also lower than in the SS model, but because of the plastic strains due to deviatory hardening the soil next to the pile behaves much softer. The decrease of the shaft resistance after the peak is well known out of tests and is also visible in Figure 6. This can be explained with arching (Franke 1976 and Touma & Reese 1974). The arching causes a decrease in stresses in the soil next to the shaft above the tip. This is shown in

the right diagram in Figure 7 for all models. The decrease in the horizontal stress at a depth of 7.5 m starts at a pile settlement of 8 mm for MC and SS and at 16 mm for HS. As an example the contribution of the skin friction along the shaft and the principle stresses for the SS model are mapped at a pile head settlement of 70 mm.

3.3 Comparison of test data and FE calculation :

The results of the FE calculations are now compared with the results of the pile load test. In Figure 10 the load settlement curve of the base resistance, the total resistance and the shaft resistance for the MC and the HS model as well as the results of the pile load test (PLT) are shown. The results of the SS model are not mapped because of the clarity and because the conclusions, which can be drawn out of the comparison, for the MC model count also for the SS model. It is obvious that the results of the HS model match the pile load test quite well. Only the inclination of the curve for the base resistance and therefore also for the total resistance at bigger settlements is not that good. The results of the MC and the SS model are not as good as of the HS model, when comparing them with the pile load test. The MC and the SS model underestimate the shaft resistance and overestimate the base resistance. The problem with the inclination of the base resistance is also visible.

Comparing the settlements of the soil next to the pile it is observable that the MC and the SS model result in the same settlements whereas the HS model causes bigger settlements which are closer to the measured values.

4. CONCLUSION :

A back analysis of a pile load test in stiff clay was presented using three different models to describe the soil behavior. Results were presented for the base, the shaft and the total resistance.

First of all the importance of interface elements was shown. Especially for the shaft resistance the results of a calculation without interface elements were heavily mesh dependent. When using interface elements the mesh dependency is negligible. For the base resistance, one needs at least two or three elements at the pile tip to get rid of the mesh dependency.

In the second part of this paper three different constitutive models were used. For the base resistance, the differences between computational results appeared to be remarkably small. For all three models, the shape of computed load-displacement curves is more or less the same. The choice of the constitutive model is thus not that important for the base resistance. The most

significant thing for the modeling of the base resistance is the right choice of the soil stiffness. Parameter selection is especially important as one has to consider also possible disturbances of the soil at the pile tip due to the installation process of the pile. It would seem that this effect is especially important for pile tips in sand. As yet we examined one such a pile numerically (Wehnert & Vermeer 2004) to find considerable base disturbance.

Results for the shaft resistance appear to depend significantly on the choice of the constitutive model. For small settlements the MC and the SS model lead to the same curve and behave too stiff. The peak value for the SS model is a bit bigger than for MC and a bit smaller than for HS. Another thing to consider for the shaft resistance is the modeling of the dilatant behavior of soil in sand and gravel. In this example the dilatancy angle is zero and therefore this problem is not visible. But for problems with a dilatancy angle unequal to zero a dilatancy cut-off is essential. Brinkgreve (1994) showed this for tension tests on piles.

Comparing the results of the three models with the results of the pile load test, the HS model would seem to be the best. But it is necessary to perform more back analyses of pile load tests to give general recommendations.

Chapter 5

Concrete for wet processed bored piles

INTRODUCTION :

The quality of piles depends on a good construction process including drilling, reinforcement installation and concrete pouring as well as on good quality of concrete. Quality of concrete has some influence on the workmanship with interrelated performances. For wet-process bored piles, concrete is cast under drilling slurry using tremie pipes. Good quality concrete in bored piling sense means that the properties and characteristics of the concrete are suitable for the process of work and subsequently meet requirements of the finished product. Continuous concrete pouring which is mandatory in piling and it is sometime disrupted by blockage of segregated or prematurely set concrete mix in the tremie pipe. Early setting of concrete after pouring in bored hole can also cause discontinuities in pile by accidental lifting of set concrete during extraction of the temporary casing. Dampness is sometimes found in top section of piles constructed in water-bearing permeable soil layer. The dampness was found to be caused by capillary action of ground water through interconnecting voids formed in the improperly mixed concrete.

Ready mixed concrete for bored piles usually specified as self-compacting concrete and its "compacting factor" is very important for achieving required strength, especially at the top section of pile which usually carries the maximum portion of transferred load. However, the compacting factor is seldom mentioned in piling practice. Besides, no vibration is allowed for tremie concrete while structural concrete is compacted by vibrator during casting. The concrete strength test is usually carried out on the compacted test samples and thus the actual strength of pile can be different from sample strength according to the compacting factor.

WET PROCESS BORED PILING METHOD :

In wet process bored piling, betonies or other type of suitable drilling slurry is used as drilling fluid to support the borehole during construction. A steel temporary casing is usually 348 used to case the top weak soil subjected to heavy construction loads. Drilling and reinforcement cage installation and concrete placing are successively executed under drilling slurry. The concrete is poured with tremie pipes, displacing the slurry well above the cutoff level. The temporary casing is then extracted immediately after concreting.

CHARACTERISTICS OF CONCRETE AND CONCRETE MIX :

Concrete mix for bored piles is designed according to concrete pouring process and mechanical properties required. Concrete for wet process piles needs to be specially mixed having cohesiveness with high workability (high slump/excellent fluidity) which is not prone to segregation and retain it workability as far as possible throughout the tremie placing operation for the complete pour. Addition to those characteristics, compaction under self-weight, resistance to harsh environment, resistance to leaching, and appropriate strength are essential.

Workability :

Excellent fluidity is essential that the concrete has the ability to flow readily through the tremie pipe, to flow laterally through a reinforcement cage and a high lateral stress against the sides of borehole. High workability is best achieved with rounded natural aggregates and natural sand in the mix.

Self Compaction :

Compaction under self-weight is essential as vibration of concrete is impractical, except near the surface. The degree of compaction achieved is determined by the density ratio (the ratio of density actually achieved to the density of the same concrete fully compacted). The recommended compacting factor for the required workability of tremie concrete is 0.95 to 0.96 .Fresh concrete is usually placed through tremie pipes and displaces the slurry by gravity action only. In some cases, lack of self-compaction in the concrete will lead to defects, such as reversed "hanging up", and "whirls" in the completed pile. If the initial shear of the concrete is very high, the flow is likely to restrain, resulting in bentonite trapped in areas not reached by the concrete.

Resistance to Segregation :

The concrete mix should be cohesive and resistant to segregation, as improperly designed mixes will segregate during placement, resulting in inferior concrete containing honeycombs and high permeable zones within the pile shaft. Concrete that bleeds or disintegrates under the pressures of its own weight can also block the tremie pipe or accept bentonite.

Controlled Setting :

The concrete must retain it fluidity thorough the depth of borehole during complete placement of the concrete in the borehole and attain an appropriate strength within a reasonable time after placement. Retarders are used to prevent premature stiffening of some cement or to delay stiffening under difficult placing conditions. The setting time must be checked against the time necessary to complete the placement. The retarders should be used under competent technical advice and after adequate testing.

Resistance to Aggressive conditions :

The concrete should have high density and low permeability to resist the possible (chemical and physical) attack of an aggressive sub-subsurface condition. In some instances there is an underground flow of water that can cause a weakening of the concrete after it is placed, and a properly designed mix should be resistant to such flow. However, if the rate of ground water flow is substantial, a permanent casing will be necessary.

Good Mechanical Performance :

The mechanical properties of hardened concrete can be satisfied in most instances. However, appropriate tensile strength for the concrete without reinforcement in piles and high level of bending and axial stress must be considered in some cases.

Reese and O'Neil (1988) emphasis that the design of the concrete mix must be given appropriate attention and the design of the mix is dependent strongly on the particular job, and the cement will be selected to be consistent with the design requirement. They observed that bleeding is not a problem for concrete mixes that are properly designed. The trial mix method is usually used in the laboratory. It is necessary to follow-up to see that the materials and proportions used by the batch plant are those of that are recommended. Inspection at the batch plant should include checking the nature, quantity and temperature of the components of the mix, the aggregates, cement, water, admixtures and of the completed mix for conformance with the specifications. For testing at the job site, the organization of the job must be such that time required to perform tests at the job site is kept to a minimum. Excessive job-site testing can lead to harmful effects. No delay in pouring should occur due to field test.

Adding of water to the concrete with very low slump on site to increase the workability can have detrimental effect of reducing the strength, compactability and impermeability of the concrete. The results of adding

water could be a significant change in the characteristics of mix and the possibility of segregation as the pour is made. Segregation of concrete during pouring can also lead to increase in permeability of concrete, especially at the top section of piles due to upward migration of water in the concrete mix. Adding of water to the concrete on site must not be allowed unless specified.

Suggested concrete mix by Fleming & Sliwinski (1977) for bored piles cast under Bentonite was shown in Table 1 while range of cement content and water-cement ratio in general use for concrete mixes by Bartholomew (1979).

Concrete for bored piles compared to that for pre-cast drive piles is the least dense concrete due to pouring and casting process. Bored pile concrete was cast in the aggressive subsurface conditions such as, high salinity, ground water fluctuation or in the vicinity of sea and river. In such environment, the concrete can easily be leached out by ground water. In this case cement content and water cement ratio needs to be reviewed considering the site conditions.

Fleming et al (1977) pointed out that the high cement content favored for in-situ pile construction enable the necessary workability mixes to be used with adequate margins of safety against the inevitable variations in strength and workability. It also compensates for some reduction in strength, which may occur on interfaces during displacement.

Concrete is permeable to water to the extent that it has interconnecting void spaces through which water can move. Calcium hydroxide liberated by hydrating cement is water-soluble and may leach out of harden concrete, leaving voids for the ingress of water. Permeability of concrete is governed by amount of cementations material, water content, aggregate grading, consolidation and curing efficiency.

METHOD OF CONCRETE POURING :

The quality of piles depends on a good pouring procedure as well as on good quality of concrete. In wet process, the concrete is usually placed by a steel tremie pipe of 20-25cm in diameter (minimum 6 times of coarse aggregate size).

Prior to charging the tremie pipe with concrete, the bottom of tremie needs to be sealed by a plug of some descriptions may be inserted at the top of tremie before or after the tremie is placed in thebored hole as appropriate. There are two potential problems that are associate with the initial charging of tremie with concrete; the concrete can segregate during placement and the air in tremie will prevent the complete filling of the tremie. These problems can be avoided if the tremie is filled slowly. Faulty initial charging of tremie during concreting can cause entrapment of mud within the concrete.

Excessive initial lifting of tremie can result in possible distribution of

leached concrete caused by concrete falling through the slurry. The bottom of tremie must stay well below the top of the column of fresh concrete all the time. Moreover, the tremie must not be lifted and lowered rapidly to avoid the cause of contamination of concrete with slurry. It is suggested that the tremie pipe must not be lift and lowered rapidly to start or restart the flow of the concrete.

In the area of high ground water level, the concrete must be deposited above the external water table before the casing is withdrawn. The hydrostatic pressure in the concrete column should be greater at all time than the pressure in any column of fluid outside the casing.

COMMON DEFECTS :

The common defects of piles are cold joints, zone of segregated or contaminated concrete, trapping of bentonite mud and cavities. The first two types of defect result from interruptions in the concrete placement or premature extracting of the tremie pipe either partially or completely above the concrete-slurry interface. Mud trappings are caused by concrete of low workability and impediment to the flow of concrete due to closely spaced bars. Discontinuities or partial separation in the piles at the bottom edge of temporary casing can be caused by accidental lifting of low workability concrete or concrete without controlled setting during casing extraction. If the concrete in the casing is too stiff and has considerable frictional resistance against the casing, a column of concrete can be pulled up with the casing.

Permeability of concrete depends on the capillary porosity, water-cement ratio and degree of hydration. High permeability of concrete can be contributed by presence of capillary pores that are interconnecting voids in the concrete. The interconnecting voids are caused by bleeding of concrete due to excessive water used in the mix. Bleeding raises the water and air bubbles to top surface of fresh concrete and raises the water-cement ratio of concrete upper part of the forms, thereby reducing the strength and increasing porosity of concrete. Concrete mixed with a water-cement ratio higher than 0.6 can be more permeable. Function of concrete self-compaction can also reduce the permeability and increase the density. In piles, usually the fresh concrete compacts under its own weight, resulting in an increasing density with depth. Besides, concrete in a great depth of pile is generally cured in stable temperature and moisture.

Usually one type of concrete mix is used for piling in a particular project in most cases. However, the project in the vicinity of the river, some bored piles are to be installed closer to the river than the remaining piles and thus the concrete mixes need to be designed accordingly. For the piles installed in the deposition side of the river where sand deposits occur, these piles are usually

subjected to be effected by the ground water flow. If the cement content is not high enough in the concrete mix and bleeding or segregation occurs, dampness or wet patches caused by capillary suction to ground water through the previous tremie location and vertical steels can be found on the pile head. Figure 3 is a photo of core sample obtained from the pile that exhibits the dampness on top, showing segregated concrete caused by bleeding.

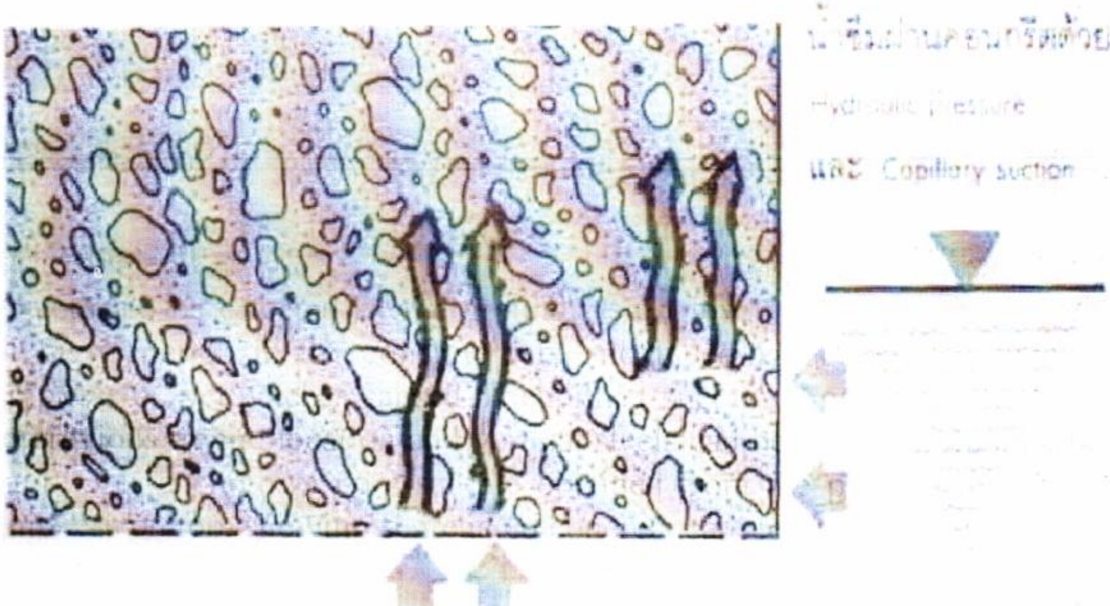


Figure 1. Capillary suction and hydraulic pressure in concrete

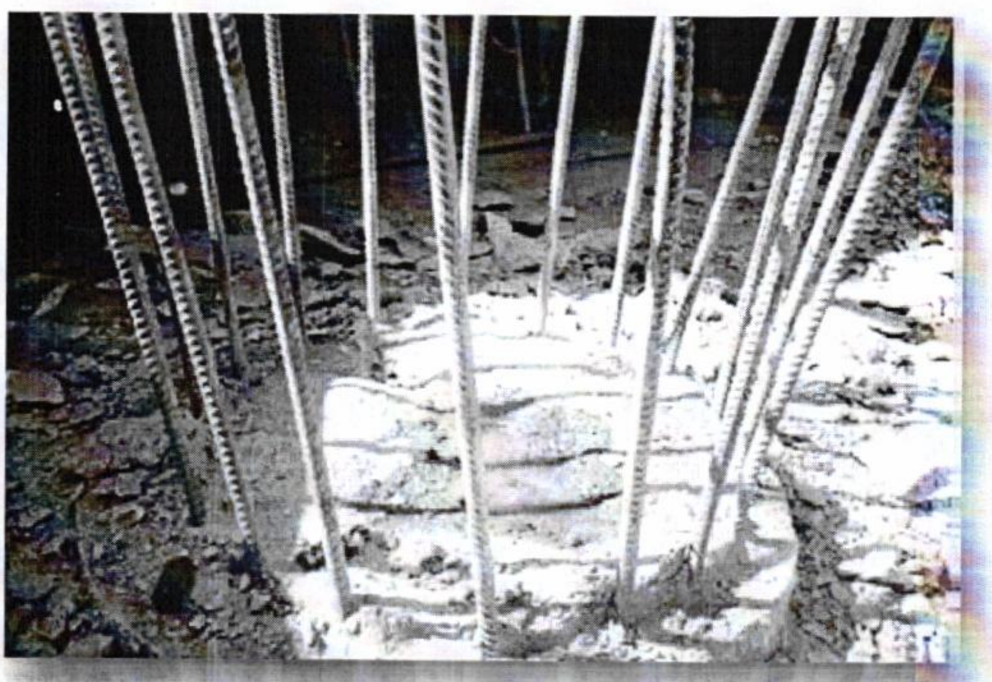


Figure 2. Wet pile head caused by capillary flow of ground water through segregated segregated concrete in the pile

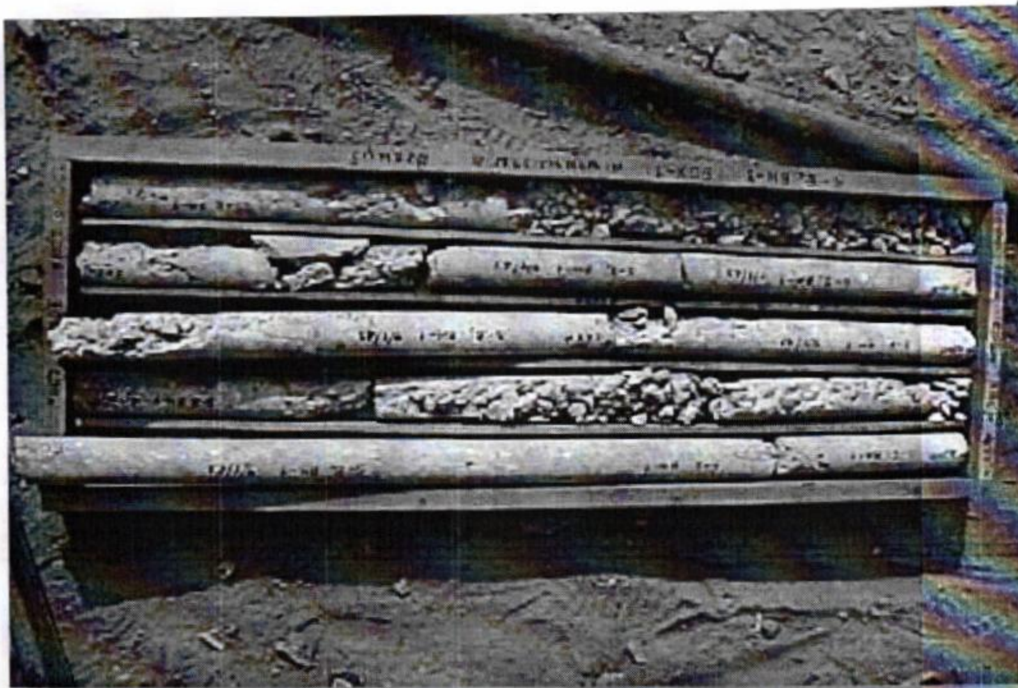


Figure 3. Core samples showing concrete in the pile

Figure (4) show the some segregated concrete extracted from the tremie pipe, which was blocked with such concrete. Settlement of solid particles or aggregates in the concrete mix cause bleeding by migration of the water to the top surface of fresh concrete, reducing water-cement ratio of the lower part of the mix. As a result, lower part of the concrete mix stiffens rapidly with aggregates. In such case stiffening concrete can also block the tremie pipe during concrete pouring (Fig. 5) causing disruption in casting process and thus effects the quality of pile. Equipment for concrete pouring must also be adequate and reliable to avoid any interruption due to a breakdown in continuous tremie concrete pouring operation.

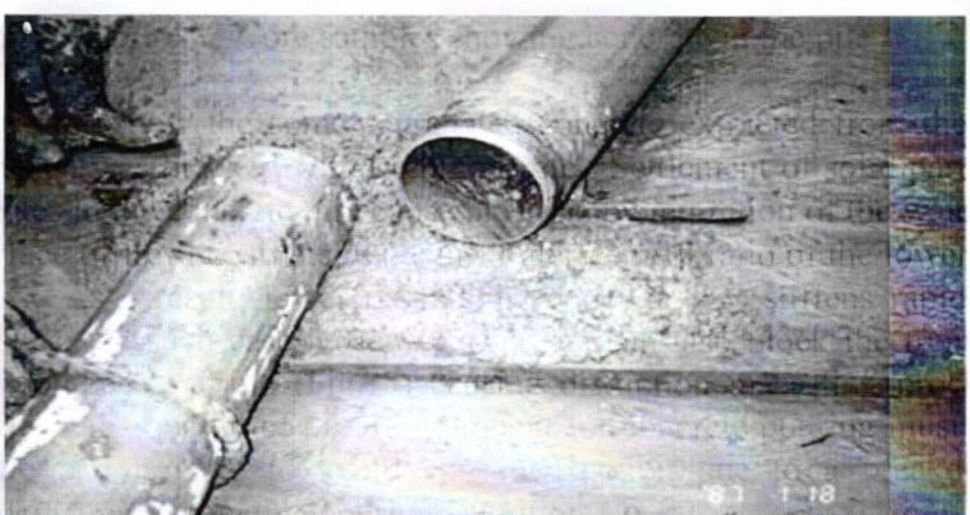


Figure 4. Tremie blocked by segregated concrete.

Figure 5. Tremie blocked by stiffening concrete

Due to a congested bar arrangement, concrete cannot flow through the bars and as a result concrete cover can be lost. Good arrangement of reinforcing bars is thus necessary. The horizontal spacing of main bars should be at least 10cm and 15cm (minimum 4 times of the maximum size of aggregate) for small and large diameter bars respectively.

RECOMMENDATIONS AND CONCLUSIONS :

Foundation designers and concrete suppliers should pay more attention to workability and many other important factors required for cast in-situ tremie concrete than strength alone. It is essential to design the better quality mix than the concrete for other structural works in some aspects, considering the process of work.

For the projects, such as elevated highways, viaducts and bridges, variable soil and ground water conditions can be encountered along the project area. In such cases, concrete mixes need to be designed to suit these conditions and used accordingly in bored pile construction.

An adequate cement content and water cement ratio is necessary to have good impermeability of concrete which is one of factors influencing the durability of concrete, especially for bored piles in water bearing subsoil.

It is concluded that appropriate concrete mix and casting practice is essential in bored piling work to achieve good quality piles.

Chapter 6

Particular Specification for Large Diameter Bored Piles with Bell-outs

1. Definition :

Large Diameter Bored Piles with Bell-outs are piles of a diameter exceeding 600 mm formed by boring, chiseling or grabbing with an enlarged base formed by under-reaming, plus filling with concrete. The bell-out at the pile base shall be formed within the bedrock with the use of a reverse circulation drill incorporating an under-reaming head.

2. Design Requirements :

2.1 Design Assumptions :

The theoretical safe loading capacity of large diameter bored piles with bell-out shall be the allowable bearing pressure on bedrock times the pile base area. Combining the end-bearing capacity and rock socket side resistance to increase the load-carrying capacity shall not be allowed.

The presumed allowable maximum bearing pressure of piles on bedrock shall be in accordance with Clause 5.19 (iv) of the General Specification for Building. Large Diameter Bored Piles with Bell-outs shall not be founded on rocks inferior to Grade III.

The use of presumptive values in accordance with Clause 5.19 (iv) of the General Specification for Building does not preclude the requirement for consideration of settlement of the structure.

The gradient of bell-out shall not exceed 30 degrees from vertical, and the diameter of pile at bell-out shall not exceed 1.5 times the diameter of pile shaft. The bell-out shall start at 300 mm below the bedrock level (see Fig. 1). In order to achieve good quality concrete, the diameter of piles at bell-out shall not exceed 3.75 m.

The Contractor shall satisfy himself that the above method of calculating the theoretical safe loading capacity provides sufficient factor of safety in his design. Should he consider that this method does not provide an adequate factor of safety in his design, he shall submit an alternative method of calculations for the approval of the Supervising Officer (SO).

2.2 Reinforcement Detail :

Reinforcement should be provided as shown in Fig. 1. The depth 'H' in Fig. 1 is bell-out tool dependent. It shall be verified on site and agreed with the SO before commencement of pile installation. However, the effect of reinforcement shall not be included in calculating the bearing capacity of pile.

2.3 Drilling before Construction:

Site borings to pre-determine the piles founding levels shall be carried out by an independent Ground Investigation Contractor from Group I and Group II of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works – Ground Investigation Field Work Category. Drill hole(s) (one for pile diameter less than or equal to 2.5 m and two for pile diameter greater than 2.5 m) shall be sunk at each bored pile position to determine the pile founding level. For this purpose, continuous rock core samples of N size with length not less than 5 m plus the socket length (as defined in Fig. 1) shall be taken below the bedrock level for inspection. However, this predicted founding level cannot be considered as final and the actual rock quality at base of pile should be inspected and verified during pile excavation before approval.

2.4 Other Requirements :

The requirements related to Large Diameter Bored Piles as stipulated in the General Specification for Building apply equally to Large Diameter Bored Piles with Bell-out.

3. Testing :

3.1 Ultrasonic Echo Sounding Test :

Ultrasonic Echo Sounding Test shall be carried out by an approved independent testing laboratory employed by the Contractor to check the shaft and bell-out profile, dimensions and verticality of each bored pile prior to installation of reinforcement cage and concreting.

3.2 Coring Test for the Pile Under-ream :

Carry out coring tests outside the pile shaft and within the enlarged base areas in accordance with Clause 5.29 of the General Specification for Building to investigate the integrity of the pile under-ream as directed by the SO.

4. Controlling and Monitoring the Verticality and Alignment of Bored Piles :

The Contractor shall submit a detailed method statement and procedures for controlling and monitoring the verticality and alignment of

piles for the SO's approval before commencement of pile installation.

During the pile installation, tolerance for verticality of each pile shall be frequently checked as directed by the SO. In case the deviation exceeding the allowable tolerance, the Contractor shall propose method of rectification for the SO's approval prior to further pile installation.

5.0 Design Submissions :

In addition to those stated in Clause 5.02 of the General Specification for Building and 4.0 above, submit 2 copies of each of the following with the design submissions:-

a. Details of boring method and equipment.
b. Details of concrete mix.
c. Sequence and details of concreting operation.

No piling works shall commence on site unless the design submissions are approved by the SO.

Chapter 7

Particular Specification for Large Diameter Bored Piles Socketed into Bedrocks

1. Definition :

Large Diameter Bored Piles Socketed into Bedrocks are those of a diameter exceeding 600 mm formed by boring, chiselling or grabbing, plus filling with concrete. The embedment depth into rocks shall be greater than 600 mm and formed by reverse circulation drill or other method approved by the Supervising Officer (SO).

2. Design Requirements :

2.1 Design Assumptions :

The theoretical safe loading capacity of large diameter bored piles socketed into granitic and volcanic bedrocks may be taken as the sum of the frictional resistance of the rock socket and the end bearing resistance of the piles provided that the socket length used in the calculation of frictional resistance does not exceed 2 pile diameters or 6 m, whichever is the shorter. However, the contribution from the minimum socket depth of 600mm stipulated in Clause 5.19 (iii) of the General Specification for Building shall be ignored in the calculation of frictional resistance (i.e. the top level of effective rock socket shall be 600 mm below the bedrock level).

2.2 Bell-out at Pile Base :

In the design of pile capacity, combining the rock socket side resistance and bell-out at the pile base to increase the load-carrying capacity shall not be allowed. For bell-out piles, the side resistance of rock socket shall be ignored.

2.3 Reinforcement Details :

Reinforcement should be provided.

2.4 Drilling before Construction :

Site borings to pre-determine the piles founding levels shall be carried out by an independent Ground Investigation Contractor from Group I and Group II of the List of Approved Suppliers of Materials and Specialist Contractors for Public Works – Ground Investigation Field Work Category.

Drill hole(s) (one for pile diameter less than or equal to 2.5 m and two for pile diameter greater than 2.5 m) shall be sunk at each bored pile position to determine the pile founding level and rock properties. For this purpose, continuous rock core samples of N size with length not less than 1.5 pile diameters plus the rock socket length plus 600 mm shall be taken below the bedrock level for inspection. However, this predicted founding level cannot be considered as final and the actual rock quality at base of pile should be inspected and verified during pile excavation before approval.

2.5 Other Requirements :

The requirements related to Large Diameter Bored Piles as stipulated in the General Specification for Building apply equally to Large Diameter Bored Piles Socketed into Bedrocks.

3. Controlling and Monitoring the Verticality and Alignment of Bored Piles :

The Contractor shall submit a detailed method statement and procedures for controlling and monitoring the verticality and alignment of piles for the SO's approval before commencement of pile installation.

During the pile installation, tolerance for verticality of each pile shall be frequently checked as directed by the SO. In case the deviation exceeding the allowable tolerance, the Contractor shall propose method of rectification for the SO's approval prior to further pile installation.

4. Design Submissions :

In addition to those stated in Clause 5.02 of the General Specification for Building and 3.0 above, submit 2 copies of each of the following with the design submissions:-

a. Details of boring method and equipment.

b. Details of concrete mix.

c. Sequence and details of concreting operation.

No piling works shall commence on site unless the design submissions are approved by the SO.

Chapter 8

DESIGN AND CONSTRUCTION OF BORED PILE FOUNDATION

1. Introduction :

Bored piles are commonly used as foundation to support heavily loaded structures such as high-rise buildings and bridges in view of its low noise, low vibration, and flexibility of sizes to suit different loading conditions and subsoil conditions. Such attributes are especially favoured in urban areas where strict restrictions with regards to noise and vibration are imposed by relevant authorities which restricted the use of other conventional piling system, e.g. driven piles. This paper presents a summary of design methodologies commonly adopted for bored piles under axial compression together with a brief discussion on the construction aspects of bored piles.

2. Geotechnical Capacity of Bored Piles :

2.1 Factor of Safety :

The Factors of Safety (FOS) normally used in static evaluation of bored pile geotechnical capacity are partial FOS on shaft (*Fs*) and base (*Fb*) respectively; and global FOS (*Fg*) on total capacity. The lower geotechnical capacity obtained from both methods is adopted as allowable geotechnical capacity

Note: Use the lower of Qag obtained from eq. 1 and eq. 2 above.

Where:
Qag = Allowable geotechnical capacity (have not included down-drag force, if any)
Qsu = Ultimate shaft capacity = $\sum i$(fsu x AS)
i = Number of soil layers
Qbu = Ultimate base capacity = fbu Ab
fs = Unit shaft resistance for each layer of embedded soil
fb = Unit base resistance for the bearing layer of soil
As = Pile shaft area
Ab = Pile base area
Fs = Partial Factor of Safety for Shaft Resistance = 1.5
Fb = Partial Factor of Safety for Base Resistance > 3.0
Fg = Global Factor of Safety for Total Resistance (Base + Shaft) = 2.0

In general, the contribution of base resistance in bored piles shall be ignored due to difficulty of proper base cleaning especially in wet hole (with drilling fluid). The contribution of base resistance can only be used if it is constructed in dry hole, proper inspection of the base can be carried out or base grouting is implemented.

2.2 Design of Geotechnical Capacity in Soil :

The design of bored pile geotechnical capacity commonly used can be divided into two major categories namely:

a) Semi-empirical Method

b) Simplified Soil Mechanics Method

2.2.1 Semi-empirical Method :

Bored piles are constructed in tropical residual soils that generally have complex soil characteristics. The complexity of these founding medium with significant changes in ground properties over short distance and friable nature of the materials make undisturbed sampling and laboratory strength and stiffness testing of the material difficult. Furthermore current theoretically based formulae also do not consider the effects of soil disturbance, stress relief and partial reestablishment of ground stresses that occur during the construction of bored piles; therefore, the sophistication involved in using such formulae may not be necessary.

Semi-empirical correlations have been extensively developed relating both shaft resistance and base resistance of bored piles to N-values from Standard Penetration Tests (SPT'N' values). In the correlations established, the SPT'N' values generally refer to uncorrected values before pile installation.

The commonly used correlations for bored piles are as follows:

$$fsu = Ksu \times SPT'N' \quad \text{(in kPa)}$$

$$fbu = Kbu \times SPT'N' \quad \text{(in kPa)}$$

Where:

Ksu = Ultimate shaft resistance factor

Kbu = Ultimate base resistance factor

SPT'N' = Standard Penetration Tests blow counts (blows/300mm)

difficulty in obtaining proper and consistent base cleaning during construction of bored piles. It is very dangerous if the base resistance is relied upon when the proper cleaning of the base cannot be assured. From back-analyses of test piles, Chang & Broms (1991) shows that Kbu equals to 30 to 45 and Toh *et al*. (1989) reports that Kbu falls between 27 and 60 as obtained from the two piles that were tested to failure.

Lower values of Kbu between 7 and 10 were reported by Tan *et al*. (1998). The relatively low Kbu values are most probably due to soft toe effect which is very much dependent on the workmanship and pile geometry.

This is even more pronouncing in long pile. Furthermore, a relatively larger base movement is required to mobilise the maximum base resistance as compared to the displacement needed to fully mobilise shaft resistance. The base displacement of approximately 5% to 10% of the pile diameter is generally required to mobilise the ultimate base resistance provided that the base is properly cleaned and checked.

2.2.2 Simplified Soil Mechanics Methods :

Generally the simplified soil mechanics methods for bored pile design can be classified into fine grained soils (e.g. clays, silts) and coarse grained soils (e.g. sands and gravels).

Fine Grained Soils :

The ultimate shaft resistance (fsu) of bored piles in fine grained soils can be estimated based on the semi-empirical undrained method as follows:

$$fsu = \alpha \text{ x } su$$

Where :
α = adhesion factor
su = undrained shear strength (kPa)

Whitaker & Cooke (1966) reports that the α value lies in the range of 0.3 to 0.6 for stiff over consolidated clays, while Tomlinson (1994) and Reese & O'Neill (1988) report α values in the range of 0.4 to 0.9. a preliminary α value of 0.8 to 1.0 is usually adopted together with the corrected undrained shear strength from the vane shear test. This method is useful if the bored piles are to be constructed on soft clay near river or at coastal area. The value of α to be used shall be verified by preliminary pile load test.

In the case where bored piles are subjected to significant variations in stress levels after installation (e.g. excavation for basement, rise in groundwater table) the use of the effective stress method is more representative as compared to undrained method. This is because the effective stress can take account of the effects of effective stress change on the Kse values to be used. The value of ultimate shaft resistance may be estimated from the following expression:

$$fsu = Kse \text{ x } \sigma v' \text{ x } \tan \varphi'$$

Where :
Kse = Effective Stress Shaft Resistance Factor = [can be assumed as Ko]
$\sigma v'$ = Vertical Effective Stress (kPa)
φ' = Effective Angle of Friction (degree) of fined grained soils.

Although the theoretical ultimate base resistance for bored pile in fine

grained soil can be related to undrained shear strength as follows;

$$fbu = Nc \times su$$

Where:

Nc = bearing capacity factor

it is not recommended to include base resistance in the calculation of the bored pile geotechnical capacity due to difficulty and uncertainty in base cleaning.

Coarse Grained Soils :

The ultimate shaft resistance (fsu) of bored piles in coarse grained soils can be expressed in terms of effective stresses as follows:

$$fsu = \beta \times \sigma v'$$

Where:

β = shaft resistance factor for coarse grained soils.

The β values can be obtained from back-analyses of pile load tests. The typical β values of bored piles in loose sand and dense sand are 0.15 to 0.3 and 0.25 to 0.6 respectively based on Davies & Chan (1981).

Although the theoretical ultimate base resistance for bored pile in coarse grained soil can be related to plasticity theories, it is not recommended to be included in the calculation of the bored pile geotechnical capacity due to difficulty and uncertainty in base cleaning.

2.3 Design of Geotechnical Capacity in Rock :

The are three major rock formations, namely sedimentary, igneous and metamorphic rocks. When designing structures over these formations using bored pile, the design approaches could vary significantly depending on the formations and the local experience established on a particular formation.

Bored pile design in rocks is heavily based on semi-empirical method. Generally, the design rock socket friction is the function of surface roughness of rock socket, unconfined compressive strength of intact rock, confining stiffness around the socket in relation to fractures of rock mass and socket diameter, and the geometry ratio of socket length-to-diameter. Roughness is important factor in rock socket pile design as it has significant effect on the normal contact stress at the socket interface during shearing. The normal contact stress increases due to dilation resulting increase of socket friction. The level 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 these aspects in the rock socket pile design. Therefore, based on the conservative approach and local experience, some semi-empirical methods have evolved to facilitate the quick socket design with considerations to all these aspects. In most cases, roughness of socket is qualitatively considered as a result of lacking of systematic assessing method. Whereas the other three factors can be quantified through strength tests on the rock cores and point load tests on the recovered fragments, the RQD values of the core samples and some analytical method on assessing the socket friction distribution. It is also customary to perform working load test to verify the rock socket design using such semi-empirical method. Safety factor of two is the common requirements for rock socket pile design. Table 1 summarises the typical design socket friction values for various rock formations .

* Note: Lower range to Grade III and higher range for Grade II or better

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

$$fs = \alpha \times \beta \times quc'$$

Where:

quc is the unconfined compressive strength of intact rock

α is the reduction factor with respect to quc (Figure 1)

β is the reduction factor with respect to the rock mass effect (Figure 2)

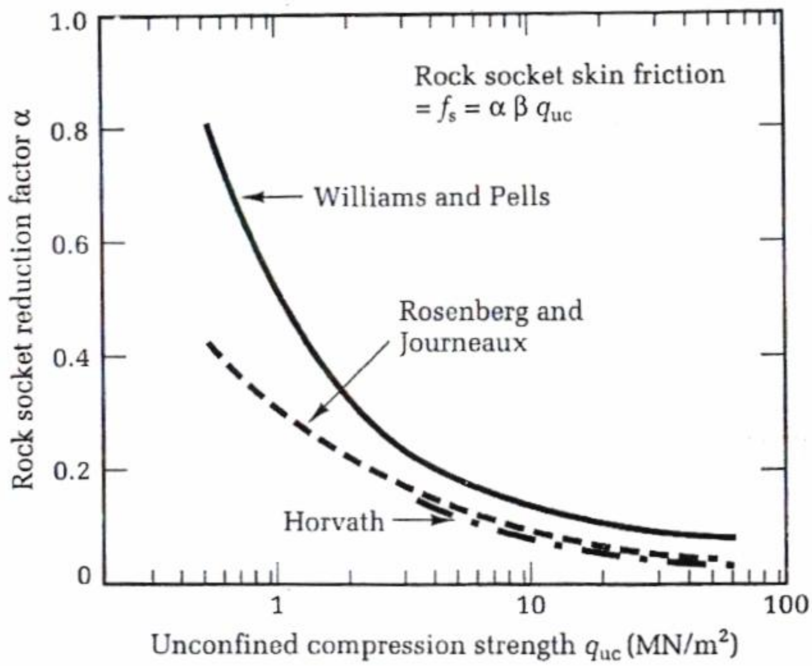


Figure 1 Rock Socket Reduction Factor, α, w.r.t. Unconfined Compressive Strength (after Tomlinson, 1995)

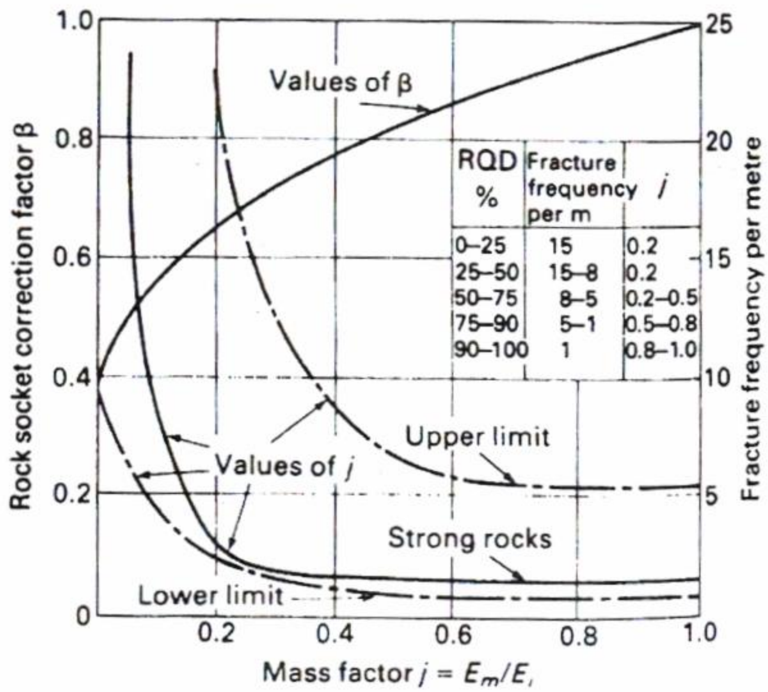


Figure 2 Rock Socket Reduction Factor, β, w.r.t. Rock Mass Discontinuity (after Tomlinson, 1995)

During borehole exploration, statistics of quc can be established for different weathering grade of bedrock and the rock fracture can be assessed

through the Rock Quality Designation 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 mass factor j, which is the ratio of elastic modulus of rock mass to that of intact rock, as in Figure 2. Alternatively, Figure 3 can provide some indications of the modulus ratio of the rock mass. In the some cases, at very small cost, point load test equipment is used to assess and verify the rock strength on the recovered rock fragment during bored pile drilling after proper calibration with borehole results.

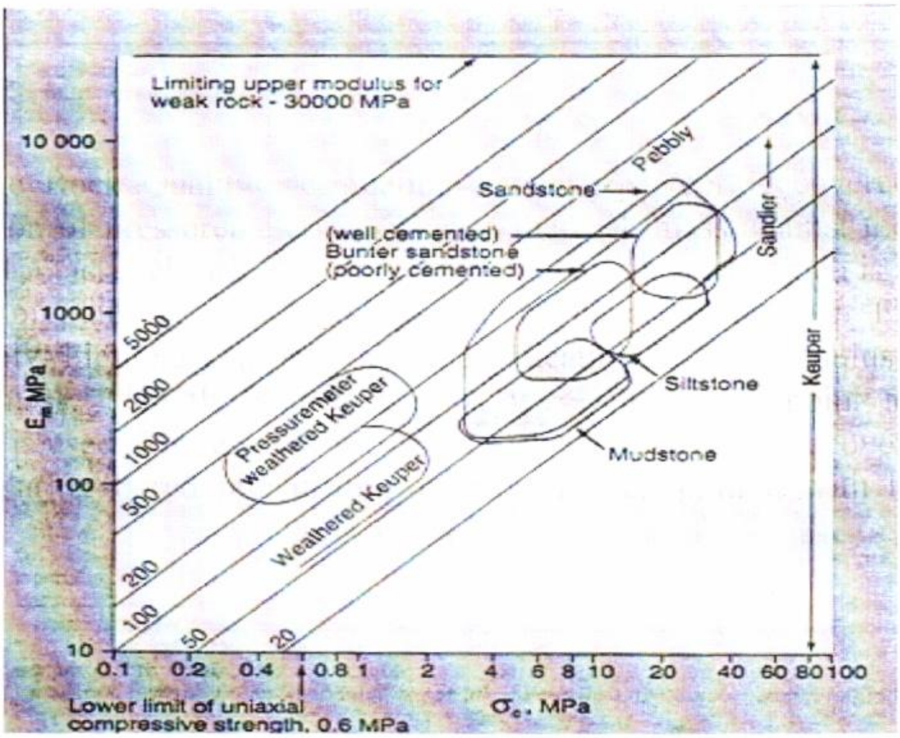


Figure 3 Modulus Ratio Ranges (after Hobbs,1974)

Due to difficulties on quantification of socket roughness, the effect of roughness has not been explicitly addressed in the above approach, but rather implicitly included in the α factor with certain socket construction method. Based on the works by Kulhawy & Phoon (1993), in which is an extension of the above mentioned model by modifying the friction reduction factor with respect to different socket roughness as shown in the following expression and Figure 4, Seidel & Haberfield (1995) have further developed the theoretical methodology and a computer program, "Rocket" for rock socket design. However, it has not gained wide acceptance in Malaysia as a result of requiring special measuring equipment for the socket roughness for the input of the said computer program. Nevertheless, Figure 4 does provide useful reference on limestone, sandstone, shale, mudstone and clay to account for the socket roughness. The parameter, ψ, is used to represent the socket roughness.

$$\alpha = \psi \times (quc/2pa)-1/2$$

Where:
ψ : Indicator of socket roughness
pa : Atmospheric pressure for normalisation

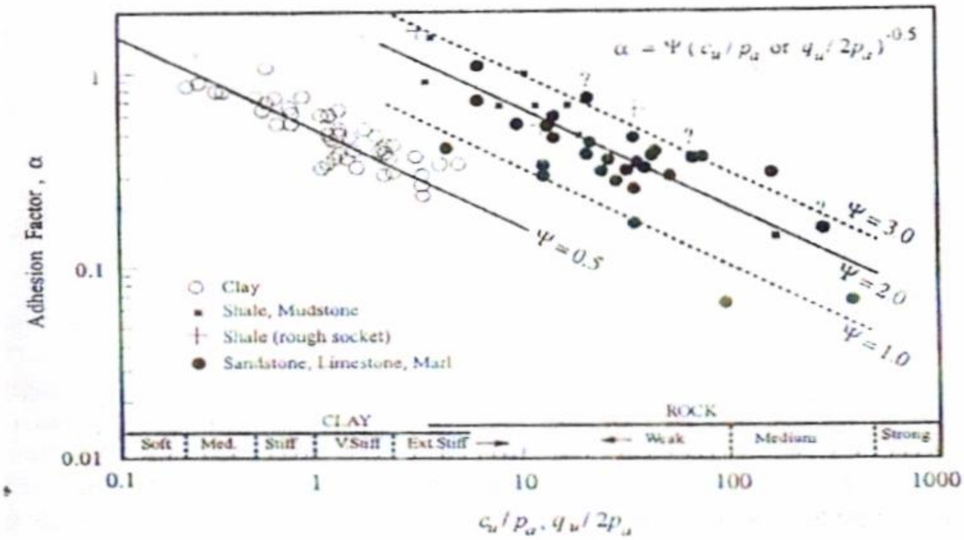


Figure 4 Relation between Socket Roughness, Socket/ Reduction Factor and Normalised Rock Strength (after Kulhawy & Phoon,1993)

It is also important to optimise rock socket design with due consideration of the load transfer behaviour of the socket. Figure 5 shows the analytical results of the socket load transfer behaviour for modulus ratio, Ep/Er ranging from 0.25 to 1000. As shown in the figure, it is obvious that there is really no reason to extend the socket beyond 5 times the pile diameter for Ep/Er =0.25 (very competent intact rock) as no load will be transferred below this socket length.

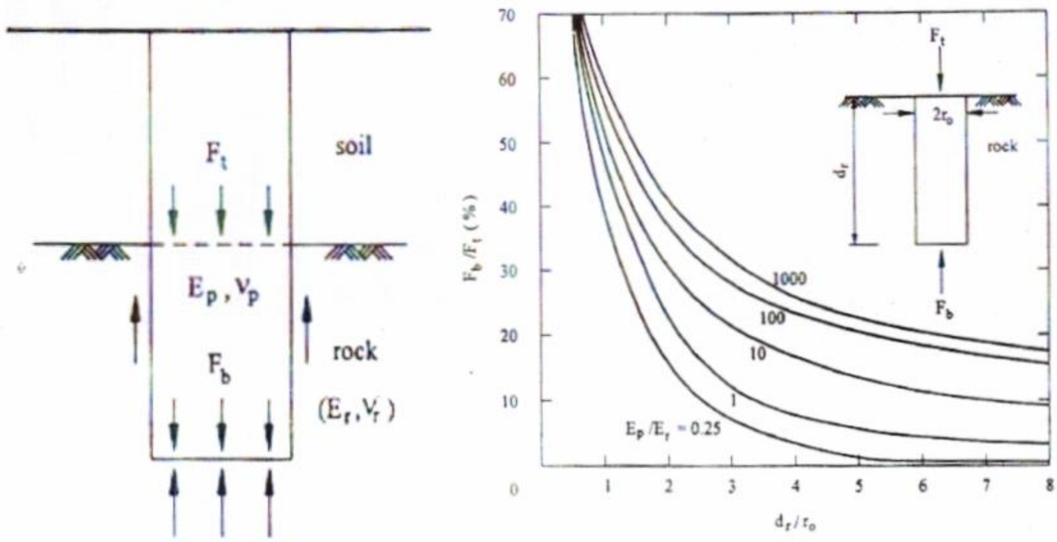


Figure 5 Distribution of Socket Resistance w.r.t. Socket Length and Modulus Ratio (after Pells & Tuner, 1979)

Sometimes, the borehole is a dry hole and at shallow depth, then base resistance will be considered if the base cleaning and inspection of the base condition can be carried out.

Very often, the movement to mobilise the base resistance is few folds higher than that to mobilise the socket friction despite the ultimate base resistance could be very high. As such, with consideration of compatibility of the pile movement in mobilising both the socket and base, appropriate mobilising factors to both the socket and base shall be applied to the foundation design after verification from the fully instrumented pile load test. Such mobilising factor shall be at least 3, but finally subjected to verification by instrumented load test prior to production of working piles if there is large number of piles for value engineering. The assessment of ultimate end bearing capacity of bored pile in rock can be carried using the following expression.

$$Qub = cNc + \gamma BN\gamma/2 + \gamma DNq$$

Where:

c : Cohesion

B : Pile diameter

D : Depth of pile base below rock surface

γ : Effective density of rock mass

Nc, $N\gamma$ & Nq : Bearing capacity factors related to friction angle, φ

Nc : $2N\varphi 1/2(N\varphi+1)$

$N\gamma$: $N\varphi 1/2(N\varphi 2-1)$

Nq : $N\varphi 2$

$N\varphi$: $Tan2(45° + \varphi/2)$

If the pile length is significant, the contribution of the shaft resistance in the soil embedment above the rock socket shall also be considered in the overall pile resistance assessment. In most cases for rock socket pile, the settlement performance is usually governed by the elastic shortening of the pile shaft. The socket displacement is usually insignificant. However, load transfer analyses would provide the over all settlement performance. Construction method is another important aspect to be considered in the bored pile design on rock.There are two most common methods in forming the rock socket, namely rock coring with rock cutting bits and chiselling by mechanical impact. Both methods have their own merits and need skilful operator to form a proper rock socket. In general, rock coring method will form a smoother, but intact, socket surface. Whereas chiselling method will form relatively rougher socket, but could be more fracture due to disturbance to the inherent discontinuities in bedrock. Chiselling is usually used as a supplementary technique in drilling through hard rock.

There are also other inherent problems associated with some of the aforementioned rock formations such as:

a. Limestone: Existence of erratic karst features will need further consideration in the foundation pile design. Downgrading of pile capacity for piles founded on these karst features or install the pile at deeper depth to penetrate these features or treatment to strengthen them can be considered depending on the cost-benefit analyses of the viable options. Another problem in limestone formation is the existence of slime made of very loose sand or soft silty clay immediately above the bedrock, which can cause frequent cave-in and pose difficulties in cleaning up the rock socket.
b. Degradable sedimentary formations: These formations easily subject to rapid degradation in terms of strength and stiffness as a result of stress relief and ingression of drilling fluid. Slow progress in drilling operation due to inefficient coring method or inter-layered hard and soft rocks and delay in concreting the piles are the usual causes of such softening. The solutions to these problems are to use powerful drilling equipments and avoid delay in concreting.
c. Granite: Core boulders are common features in this formation. This feature can be easily observed from the outcrops or along river. Therefore, it is important to identify proper founding stratum for the foundation piles during the subsurface investigation. This can be overcome by careful assessment of the weathering profile interpreted from the deep boring exploratory holes.

2.4 Verification of Bored Piles Capacity :

For the verification of bored pile capacity, maintained load test is the normal mean specified by most practicing engineers. In certain cases where detailed interaction behaviours between the pile and the foundation formations are of interest to the designer for design refinement and value engineering, full scale instrumented test pile equipped with multi-level strain gauges.

3. Structural Requirements of Bored Pile :

Following are some brief guidelines for structural design of bored piles:

a) Allowable structural capacity of bored piles

Allowable structural capacity of bored piles = 0.25 x fcu x Ac

Where:

fcu = concrete cube strength at 28 days (Grade 30 to 35 is most common)

Ac = cross-sectional area of the pile

b) Cover for reinforcement

Cover for reinforcement = (40mm + values in Table 3.4, BS8110: Part 1)

c) Reinforcement

For bored piles in compression only, the structural capacity is derived from the concrete strength alone and some nominal reinforcement is sometimes provided to prevent damage during construction. However, for bored piles supporting bridges where there will be bending moment and shear force acting on the piles, then the bored piles can be designed like beam. Length of the reinforcement can be curtailed until the influence depth of the flexural effect. Hanging the steel cage without the lower supporting steel reinforcements has been successfully carried out. However, for ease of construction, minimum steels are sometimes provided right to the bottom of the bored pile to support the upper steel cage during concrete casting.

3.1 Verification of Concrete Quality for Bored Pile (Integrity Tests) :

Besides verification of capacity, concrete quality of bored piles is also an important aspect of design and construction of bored piles. Concreting for bored piles is usually carried out using tremie (self-compacting) concrete. Some general recommendations on tremie concrete are summarised below:

a. The concrete should be cohesive, rich in cement (i.e. not less than 400 kg/m3) and of slump not less than 150 mm.
b. The sides of the borehole have to be stable. This may be achieved by maintaining an adequate head of fluid or by the provision of a temporary casing of the necessary length.
c. The tremie pipe should be water tight throughout its length and have a hopper attached at its head by a watertight connection.

d. The tremie pipe should be large enough in relation to the size of aggregate. For 20 mm aggregate the tremie pipe should be of diameter not less than 150 mm, and for larger aggregate tremie pipes of larger diameter are required.

e. The tremie pipe should be lowered to the bottom of the boreholes allowing ground water to rise inside it. It is essential to prevent the tremie concrete from mixing with water in the tremie pipe and to this end a plug or other devise should be used.

f. The tremie pipe should always be kept full of concrete and should penetrate well into the concrete in the borehole with an adequate margin of safety against accidental withdrawal if the pipe is surged to discharge the concrete.

g. The pile should be concreted wholly by tremie and the method of deposition should not be changed part way up the pile, to prevent the laitance from being entrapped within the pile.

h. If the time taken to form large piles is likely to be excessive, the use of set retarding admixtures should be considered, particularly in the case of high ambient temperatures.

i. All tremie pipe should be scrupulously cleaned before use.

j. When drilling muds such as bentonite suspension are used, the fluid at the pile base should be checked for contamination before concreting to ensure that it will be readily displaced by the rising concrete.

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