Design & Construction of Micropiles

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ABSTRACT: This paper discusses the micropile classification, design concept, problems associated with the common installation methods, construction control and performance of this piling system. Micropiles can be designed as soil frictional piles and rock socketed piles either under tension or compression. Discussions will be addressed on the strain compatibility between the steel reinforcements and the grout under high carrying working load and the effects of grout in rock socketed micropile with permanent casings in the overburden soil. The strain compatibility problem of micropile, although important, is usually overlooked by many engineers. However, the inherent conservatism in the micropile design may obscure this compatibility problem. Construction control is another important aspect to warrant success of the micropiling system. Case histories on the construction problems, such as jointing of the reinforcements, disturbance to the subsurface materials induced by the different installation techniques and control of grouting operation, will be presented and discussed. Results of static pile load tests of micropiles are also presented. Generally, rock socketed micropiles usually experience very small residual settlement as compared to other piling systems.

1.0 Introduction

Micropiles were conceived in Italy in the early 1950's in response to the demand for innovative techniques for underpinning historic building and monuments that has sustained damage with time. The micropile systems used today are evolution from the basic small-diameter, cast-in-place pile developed by Dr. Fernando Lizzi called “palo radice”. A typical micropile construction involves the drilling the pile shaft to the required depth, placing the steel reinforcement, initial grouting by tremie and placing additional grout under pressure where applicable. A typical construction sequence is shown in Figure 1.

In Malaysia, micropiles are widely recognised as a common remedial option for underpinning structures with foundation problems and as well as a common foundation option. This is primarily due to local geological factors such as karstic features in limestone formation and performance factors for speedy foundation construction. The sizes of micropiles constructed in Malaysia vary from 100mm to 350mm carrying load from 150kN to 2,800kN respectively

Other advantages of micropiles are high carrying capacity, less site constraint problems and self sustained operation. This piling system is therefore attractive to both the client and the foundation designer. Apart from the light and compact drilling rigs, other ancillary equipment, like grout mixer and grout pump, is very compact in size. The only disadvantage of micropiles is the relatively high cost as compared to other piling systems except for the case of shallow pile termination depth.

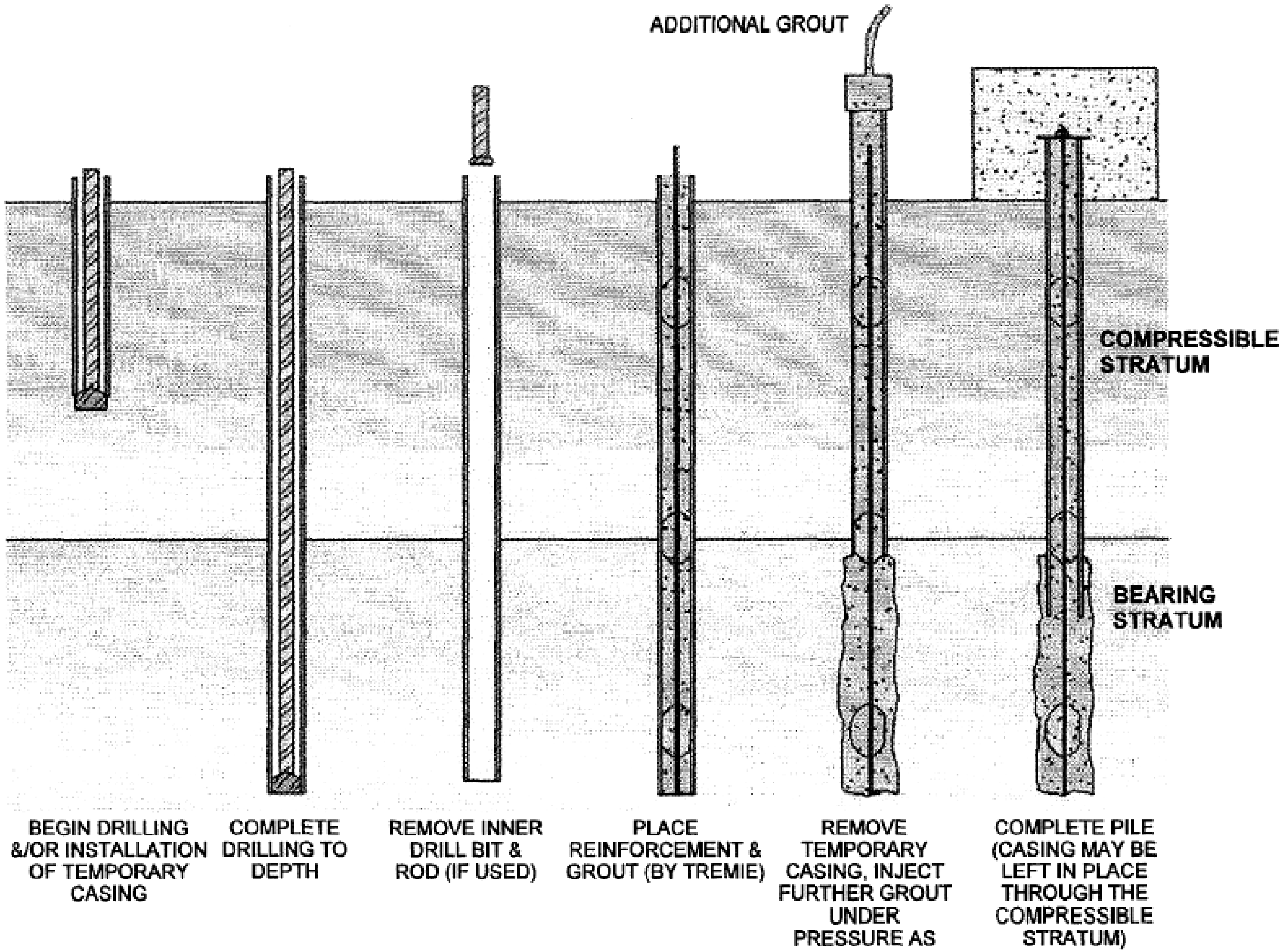


Figure 1: Typical Micropile Construction Sequence

2.0 TYPES OF MICROPILES AND DRILLING METHODS

2.1 Micropile Classification

Micropiles are generally classified firstly according to design application and grouting method. The design application dictates the function of the micropile while the grouting method defines the grout/grout bond capacity.

In the design application, there are two general types of application. The first type is where the micropile is directly loaded either axially or laterally and the pile reinforcement resists the majority of the applied load. Examples of such application are shown in Figure 2. This type of pile is used to transfer structural loads to deeper, more competent or stable stratum and may be used to restrict the movement of the failure plane in slopes. The loads are primarily resisted by the steel reinforcement structurally and by the grout/grout bond zone geotechnically.

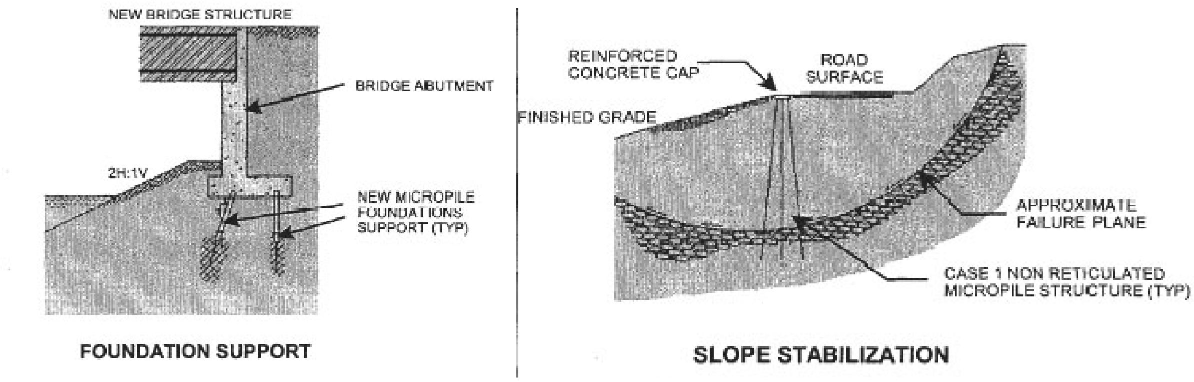


Figure 2: Directly Loaded Micropiles Examples

Second type of design application is where the micropile reinforces the soil to make a reinforced soil composite that resist the applied load and known as reticulated pile network. Examples are shown in Figure 3. This application of micropile serves to circumscribe and internally strengthen the reinforced soil composite.

The method of grouting is generally the most sensitive construction control over grout/ground bond capacity and varies directly with the grouting method. The second part of the micropile classification is based primarily on the method of placement and pressure under which grouting is used during construction. The classification is shown schematically in Figure 4.

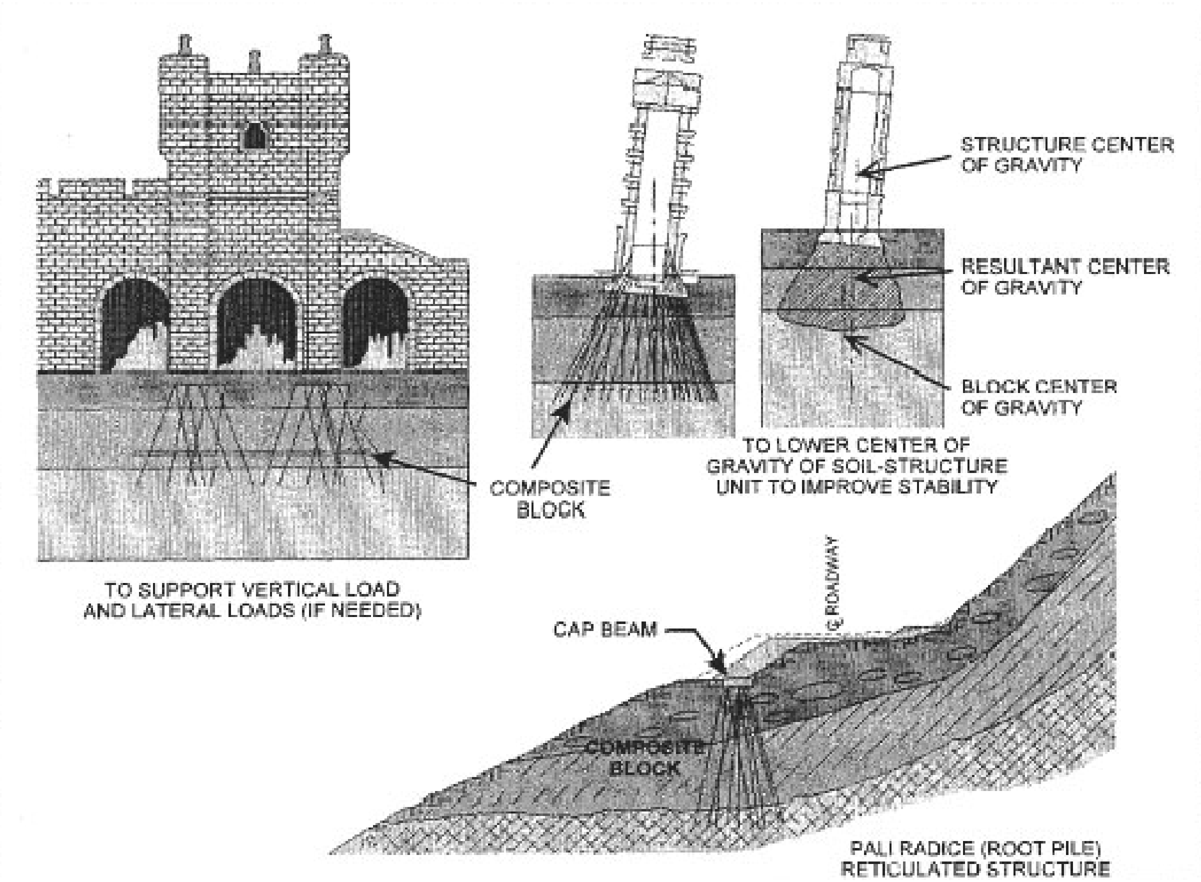


Figure 3: Reticulated Pile Network Micropiles Examples

Type A: Type A classification indicates that grout is placed under gravity head only. Sand-cement mortars, as well as neat cement grouts, can be used because the grout column is not pressurised.

Type B: Type B indicates that neat cement grout is placed into the hole under pressure as the temporary steel drill casing is withdrawn. Injection pressures typically range from 0.5 to 1 MPa, and are limited to avoid hydro-fracturing the surrounding ground or causing excessive grout takes, and to maintain a seal around the casing during its withdrawal, where possible.

Type C: Type C indicates a two-step process of grouting: Primary grout is placed under pressure of 1.0 – 2.0 MPa, causing hydrofracturing of surrounding ground. Prior to the hardening of the primary grout (typically 15 to 25minutes), secondary grout is injected usually via tube á manchette. This method is sometimes referred to as IGU (Injection Globale et Unitaire)

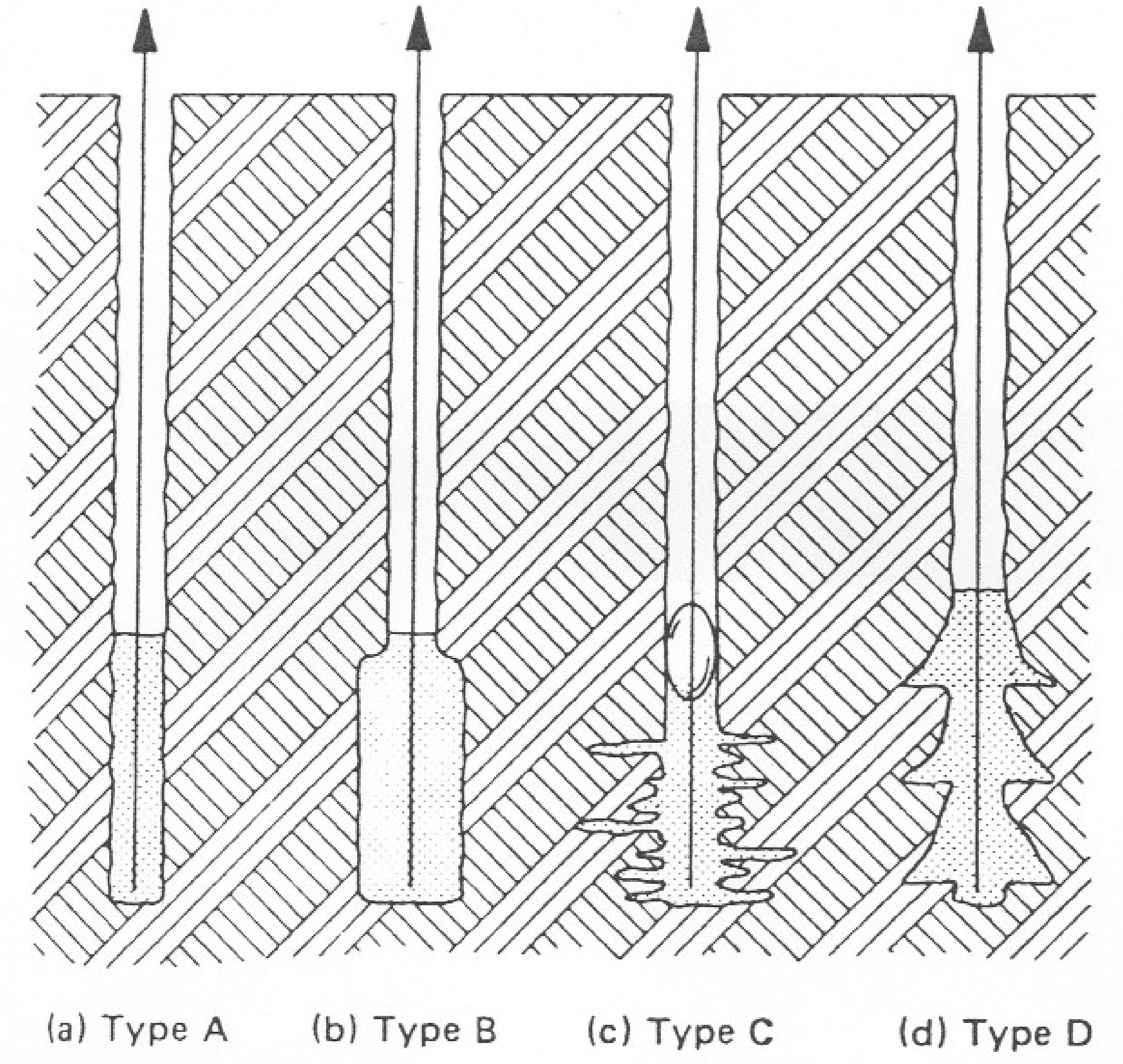


Figure 4: Micropile Classification based on Grouting Method

Type D: Type D indicates a two-step process of grouting similar to Type C with modifications to the secondary grouting. Primary grout is placed under pressure and after hardening of the initially placed grout, additional grout is injected via tube á manchette at a pressure of 2 to 8 MPa. A packer may be so that specific levels can be treated several times, if required.

2.2 Drilling Techniques

The drilling method is selected on the basis of causing minimal disturbance to the ground and nearby sensitive structures and able to achieve the required drilling performance. In all drilling methods, drilling fluid is used as a coolant for the drill bit and as a flushing medium to remove the drill cuttings. Water is the most common drilling fluid compared to other drilling fluid such as drill slurries, polymer, foam and bentonite. Another type of flushing medium is using compressed air, which is commonly used in Malaysia.

Generally, there are six main drilling techniques and their principles and range of application are shown in Table 1.

Single-tube advancement - external flush (wash boring): By this method, the toe of the drill casing is fitted with an open crown or bit, and the casing is advanced into the ground by rotation of the drill head. Water flush is pumped continuously through the casing, which washes debris out and away from the crown. The water-borne debris typically escapes to the surface around the outside of the casing, but may be lost into especially loose and permeable upper horizons. Care must be exercised below sensitive structures in order that uncontrolled washing does not damage the structure by causing cavitation.

Air flush is not normally used with this system due to the danger of accidentally over pressurizing the ground in an uncontrolled manner, which can cause ground disturbance. Conversely, experience has shown that polymer drill flush additives can be very advantageous in certain ground conditions, in place of water alone. These do not appear to detrimentally affect grout-to-soil bond development as may be the case with bentonite slurries.

Rotary Duplex: With the rotary duplex technique, drill rod with a suitable drill bit is placed inside the drill casing. It is attached to the same rotary head as the casing, allowing simultaneous rotation and advancement of the combined drill and casing string. The flushing fluid, usually water or polymer flush, is pumped through the head down through the central drill rod to exit from the flushing ports of the drill bit. The flush-borne debris from the drilling then rises to the surface along the annulus between the drill rod and the casing. At the surface, the flush exits through ports in the drill head. Although any danger with duplex drilling is less than when using the single-tube-method, air flush must be used with caution because blockages within the annulus can allow high air pressures and volumes to develop at the drill bit and cause ground disturbance.

Rotary Percussive Duplex (Concentric): Rotary percussive duplex systems are a development of rotary duplex methods, whereby the drill rods and casings are simultaneously percussed, rotated, and advanced. The percussion is provided by a top-drive rotary percussive drill head. This method requires a drill head of substantial rotary and percussive energy.

Rotary Percussive Duplex (Down-the-Hole Hammer): Originally sold as the Overburden Drilling Eccentric (ODEX) System, this method involves the use of rotary percussive drilling combined with an eccentric under-reaming bit. The eccentric bit undercuts the drill casing, which then can be pushed into the oversised drill hole with much less rotational energy or thrust than is required with the concentric method just described. In addition, the drill casing does not require an expensive cutting shoe and suffers less wear and abrasion.

The larger diameter options, of more than 127 mm in diameter, often involve the use of a down-the-hole hammer acting on a drive shoe at the toe of the casing, so that the casing is effectively pulled into the borehole as opposed to being pushed by a top hammer. Most recently, systems similar to ODEX, which is now sold as TUBEX, have appeared from European and Japanese sources. Some are merely mechanically simpler versions of TUBEX. Each variant, however, is a percussive duplex method in which a fully retractable bit creates an oversized hole to ease subsequent casing advancement.

Double Head Duplex: With the double head duplex method, a development of conventional rotary duplex techniques, the rods and casings are rotated by separate drill heads mounted one above the other on the same carriage. These heads provide high torque (and so enhanced soil-and obstruction-cutting potential), but at the penalty of low rotational speed. However, the heads are geared such that the lower one (rotating the outer casing), and the upper one (rotating the inner drill string) turn in opposite directions. The resulting aggressive cutting and shearing action at the bit permits high penetration rates, while the counter-rotation also discourages blockage of the casing/rod annulus by debris carried in the exiting drill flush. In addition, the inner rods may operate by either purely rotary techniques or rotary percussion using top-drive or down-the-hole hammers. The counter-rotation feature promotes exceptional hole straightness, and encourages penetrability, even in the most difficult ground conditions.

Hollow-Stem Auger: Hollow-stem augers are continuous flight auger systems with a central hollow core, similar to those commonly used in auger-cast piling or for ground investigation. These are installed by purely rotary heads. When drilling down, the hollow core is closed off by a cap on the drill bit. When the hole has been drilled to depth, the cap is knocked off or blown off by grout pressure, permitting the pile to be formed as the auger is withdrawn. Such augers are used mainly for drilling cohesive materials or very soft rocks.

Drilling Method	Principle	Common Diameters	Typical Maximum Depths	Remarks
Singe-tube advancement: (a) Drive drilling (b) External flush	Casing with "lost point" percussed without flush. Casing, with shoe, rotated with strong water flush.	50 - 100 mm 100 - 250mm	30 m 60 m	Obstructions or very dense soil problematical. Very common for anchor installation. Needs high torque and powerful flush pump.
Rotary Duplex	Simultaneous rotation and advancement of casing plus internal rod, carrying flush.	100 - 220 mm.	70 m	Used only in very sensitive soil/site conditions. Needs positive flush return and high torques (internal flushing only)
Rotary percussive concentric duplex	As rotary duplex, except casing and rods percussed as well as rotated	89 - 175mm	40 m	Useful in obstructed/rocky conditions. Needs powerful top rotary percussive hammer.
Rotary percussive eccentric duplex (ODEX/TUBEX)	As rotary duplex, except eccentric bit on rod cuts oversized hole to ease casing advance.	89 - 300 mm	60 m	Expensive and difficult system for difficult overburden.
"Double head" duplex	As rotary duplex, except casing and rods may rotate in opposite directions.	100 - 150mm	60 m	Powerful, new system for fast, straight drilling in very difficult ground. Needs significant hydraulic power.
Hollow-stem auger	Auger rotated to depth to permit subsequent introduction of grout and/or reinforcement through stem.	100 - 400mm	30 m	Obstructions problematical; care must be exercised in cohesionless soils Prevents application of higher grout pressures.

Table 1: Drilling Methods

Various forms of cutting shoes or drill bits can be attached to the lead auger, but heavy obstructions, such as old foundations and cobble and boulder soil conditions, are difficult to penetrate economically with this system. In addition, great care must be exercised when using augers: uncontrolled penetration rates or excessive “hole cleaning” may lead to excessive spoil removal, thereby risking soil loosening or cavitation in certain circumstances.

In Malaysia, two most common drilling techniques used are the wash boring and rotary percussive duplex. The wash boring is slow in drilling through hard materials but causes less disturbance to the surrounding soil. Rotary percussive duplex (ODEX system) has the advantages of fast penetration and good verticality but has excessive vibration and blowing out of excessive earth materials. Therefore, rotary percussive duplex technique is favourable in most site conditions except in sensitive ground with adjacent structures. Wash boring drilling technique best suits for drilling at sensitive ground and remedial works for foundations under distress.

2.3 Grouting

Grouting operations have a major impact on the micropile carrying capacity and the details of the grouting vary somewhat throughout the world, depending on the origins of the practice and the quality of the local resources. In general, the grout mixture consist of cement, water and in certain cases additives such as sand and superplasticizers may be added to achieve the required working conditions.

The critical importance of the grouting operation is underlined by the fact that the placed grout is required to serve a number of purposes:

- It transfers the imposed loads between the reinforcement and the surrounding ground.
- It may form part of the load-bearing cross section of the pile.
- It serves to protect the steel reinforcement from corrosion.
- Its effects may extend beyond the confines of the drill hole by permeation, densification, and/or fissuring.

The grout, therefore, needs to have adequate properties of fluidity, strength, stability, and durability. The need for grout fluidity can mistakenly lead to the increase in water content; this has a negative impact on the other three properties. Of all the factors that influence grout fluidity and set properties, the water/cement ratio is the most dominant. Figure 5 illustrates why this ratio is limited to a range of 0.45 to 0.50, although even then, additives may be necessary to ensure adequate workability for ratios less than 0.40. Commonly, non-shrink grout specification requires the addition of expanding grout admixture. Non-shrink grout is important as the grout volume shrinks during setting and reduces the grout/rock bond which can severely impair the micropile capacity. Therefore, expanding grout admixture are added into the grout mix to provide an expansion of about 1 – 4% to compensate against the grout shrinkage and maintain the grout/rock bond.

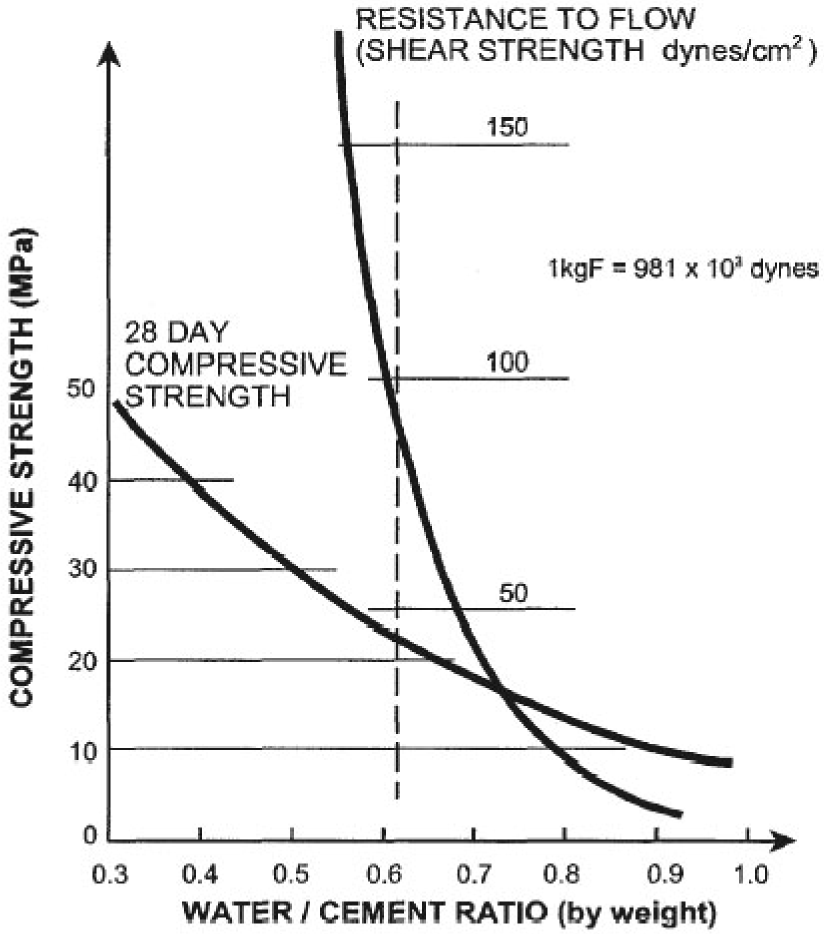


Figure 5: Effect of Water Content on Grout Compressive Strength and Flow Properties (Barley & Woodward, 1992)

2.4 Grouting Equipment

In general, any plant suitable for the mixing and pumping of fluid Cementitious grouts may be used for the grouting of micropiles. The best quality grouts, in terms of both fluid and set properties, are produced by high-speed, high-shear colloidal mixers (Figure 6) as opposed to low-speed, low-energy mixers, such as those that depend on paddles (Figure 7). Mixing equipment can be driven by air, diesel, or electricity, and is available in a wide range of capacities and sizes from many manufacturers.

For grout placement, lower pressure injection (say, to 1 MPa) is usually completed using constant pressure, rotary-screw type pumps, while higher pressure grouting, such as for Type C or D micropiles, usually requires a fluctuating pressure piston or ram pump. Collodial mixers are generally preferred as they can break up the cement lumps and ensure uniformity of the grout mixture.

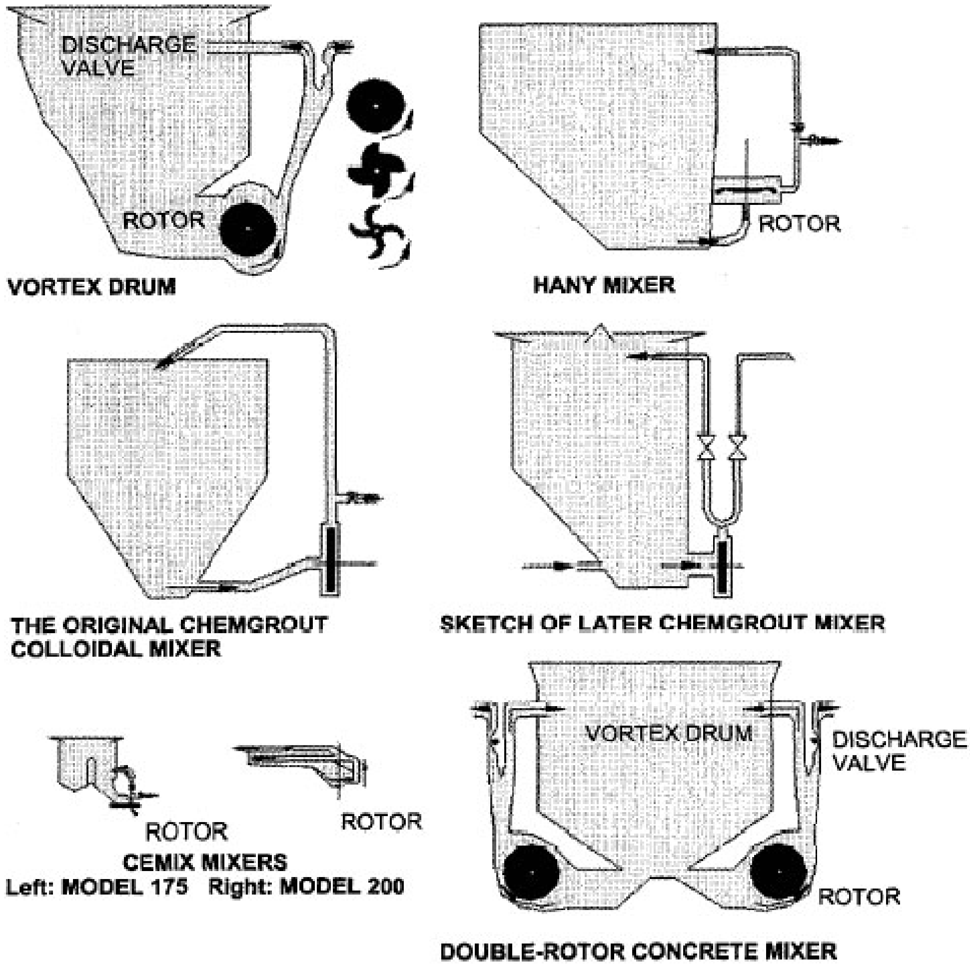


Figure 6: Various Types of Colloidal Mixers (Armour etal., 2000)

2.5 Grout Mixing

The measured volume of water is usually added to the mixer first, followed by cement and then aggregate or filler if applicable. It is generally recommended that grout be mixed for a minimum of two minutes and that thereafter the grout be kept in continuous slow agitation in a holding tank prior to being pumped to the pile. Only in extreme cases, for example where exceptionally large takes are anticipated should ready-mix grout supply be required. The grout should be injected within a certain maximum time after mixing. This “safe workability” time should be determined on the basis of on- site tests, as it is the product of many factors, but is typically not in excess of one hour.

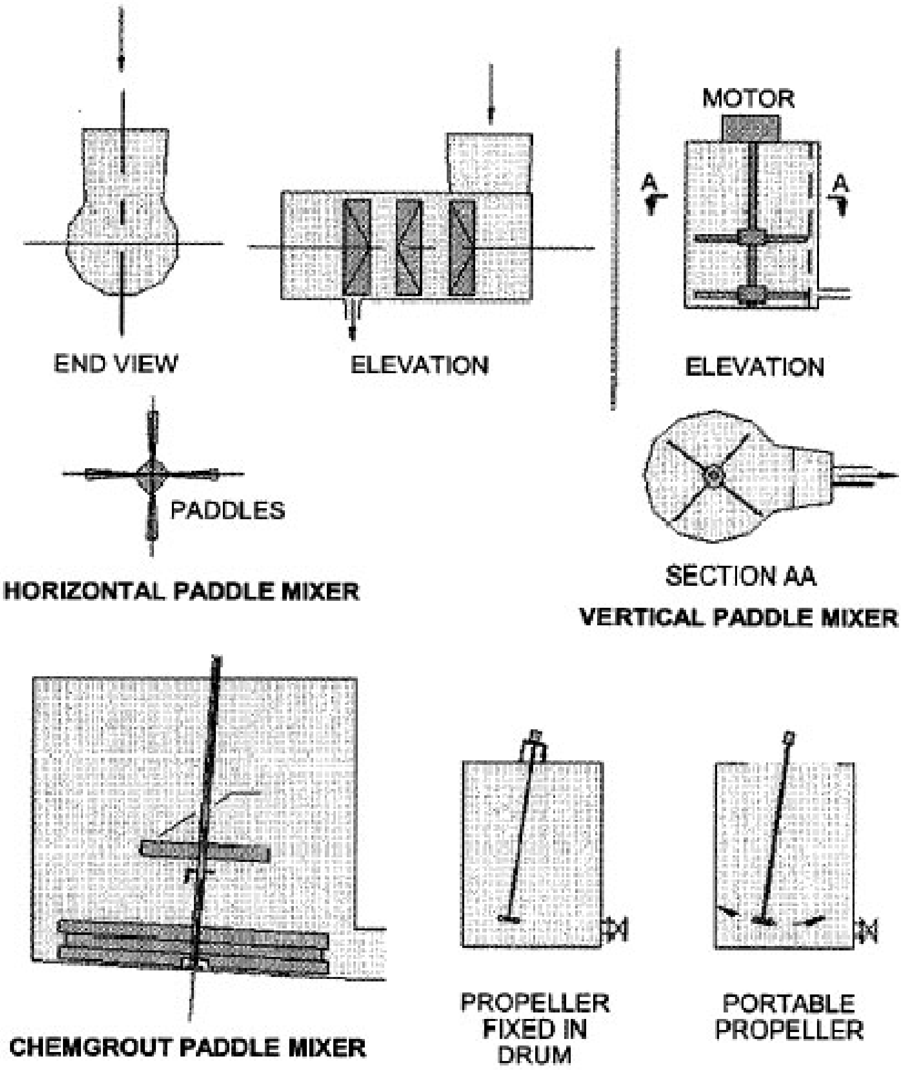


Figure 7: Various Types of Paddle Mixers (Armour etal., 2000)

2.6 Grout Placement Techniques

2.6.1 Gravity Fill Techniques (Type A Micropiles)

Once the hole has been drilled to depth, it is filled with grout and the reinforcement is placed. Grout should always be introduced into the drill hole through a tremie pipe exiting at the bottom of the hole. Grout is pumped into the bottom of the hole until grout of similar quality to that being injected is freely flowing from the mouth of the borehole. No excess pressure is applied. Steps are taken to ensure that the quality of grout is maintained for the full length of the borehole. This type and phase of grouting is referred to as the primary treatment.

Gravity fill techniques tend now to be used only when the pile is founded in rock, or when low-capacity piles are being installed in stiff or hard cohesive soils, and pressure grouting is unnecessary (Bruce and Gemme, 1992).

2.6.2 Pressure Grouting Through the Casing (Type B Micropiles)

Additional grout is injected under pressure after the primary grout has been tremied, and as the temporary casing is being withdrawn. The aim is to enhance the grout/soil or rock bond characteristics. This operation can be limited to the load transfer length within the design-bearing stratum, or may be extended to the full length of the pile where appropriate.

Pressure grouting is usually conducted by attaching a pressure cap to the top of the drill casing (this is often the drilling head itself) and injecting additional grout into the casing under controlled pressure. Grout pressures are measured as close to the point of injection as possible, to account for line losses between pump and hole. Commonly, a pressure gauge is mounted on the drill rig and monitored by the driller as a guide to rate of casing withdrawal during the pressurization phase. Alternatively, if a grouting cap is used and the casing is being extracted by means other than the drill rig (e.g., by hydraulic jacks), it is common to find a pressure gauge mounted on the cap itself. Line losses are inevitable in the system, but contractors typically record the pressure indicated on the pressure gauge without the correction, reasoning that such losses are compensated by the extra pressure exerted by the grout column due to its weight in the borehole.

The effective injection pressures (typically 20 kPa per meter of depth in loose soils and 40 kPa per meter of depth in dense soils) are dictated by the following factors:

- The need to avoid ground heave or uncontrolled loss of grout.
- The nature of the drilling system (permissible pressures are lower for augers due to leakage at joints and around the flights).
- The ability of the ground to form a “seal” around the casing during its extraction and pressure grouting.
- The need to avoid “seizing” the casing by flash setting of the grout due to excessive pressure, preventing proper completion of the pile.
- The “groutability” of the ground.
- The required grout/ground bond capacity.
- Total pile depth.

The injection of grout under pressure is aimed at improving grout/ground skin friction, thus enhancing the load-carrying capacity of the micropile. Extensive experience with ground anchors has confirmed the effect of pressure grouting on ultimate load-holding capacity.

When carrying out pressure grouting in granular soils, a certain amount of permeation and replacement of loosened soils takes place. Additionally, a phenomenon known as pressure filtration occurs, wherein the applied grout pressure forces some of the integral mixing water out of the cement suspension and into the surrounding soil. This process leaves behind a grout of lower water content than was injected and is thus quicker setting and of higher strength. It also causes the formation of cake-like cement paste along the grout/soil interface that improves bond. In cohesive soils, some lateral displacement, compaction, or localised improvement of the soil can occur around the bond zone, although the improvement is generally less well marked than for cohesionless soils.

Pressure grouting also appears to cause a recompaction or redensitication of the soil around the borehole and increases the effective diameter of the pile in the bond zone. These mechanisms effectively enhance grout/soil contact, leading to higher skin friction values and improved load/displacement performance. Such pressure grouting may also mechanically improve the soil between piles.

2.6.3 Post-grouting (Type C and D Micropiles)

It may not be possible to exert sufficiently high grout pressures during the casing removal stage. For example, there may be ground hydro-fracture or leakage around the casing. Alternatively, some micropile construction methods may not use or need a temporary drill casing, and so pressure grouting of the Type B method is not feasible. These circumstances have led to the development of post-grouting techniques, whereby additional grout can be injected via special grout tubes some time after the placing of the primary grout. Such grouts are always neat cement-water mixes (for the ease of pumpability) and may therefore have higher water contents than the primary grout. It is reasoned that excess water from these mixes is expelled by pressure filtration during passage into the soil, and so the actual placed grout has a lower water content (and therefore higher strength).

This post-grouting method is primarily used in the Type C and Type D micropile classification:

Type C: Neat cement grout is placed in the hole as done for Type A. Between 15 and 25 minutes later, and before hardening of this primary grout, similar grout is injected once from the head of the hole without a packer, via a 38- to 50-mm diameter preplaced sleeved grout pipe (or the reinforcement) at a pressure of at least 1 MPa.

Type D: Neat cement grout is placed in the hole as done for Type A. When this primary grout has hardened, similar grout is injected via a preplaced sleeved grout pipe. Several phases of such injection are possible at selected horizons and it is typical to record pressures of 2 to 8 MPa, especially at the beginning of each sleeve treatment when the surrounding primary grout must be ruptured for the fast time. There is usually an interval of at least 24 hours before successive phases. Three or four phases of injection are not uncommon, contributing additional grout volumes of as much as 250 percent of the primary volume.

Variations on the technique exist. The post-grout tube can be a separate 25 mm or 38 mm diameter sleeved plastic pipe (tube-a-manchette) placed together with the steel reinforcement (Figure 8), or it can be the reinforcement tube itself, suitably sleeved (Figure 9). In each of these cases, a double packer may be used to grout through the tubes from the bottom sleeve upwards.

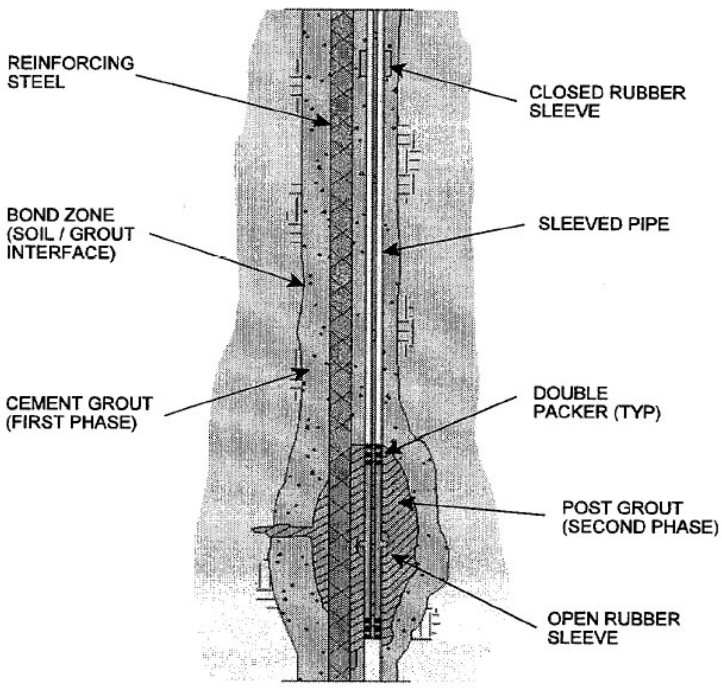


Figure 8: Principle of the Tube á Manchette Method of Post-Grouting

2.7 Reinforcement

Generally, there are three types of reinforcement for micropiles and consist of single reinforcing bar, reinforcement bars or steel pipe. The most commonly used in Malaysia is the rolled structural steel and the steel pipe. Reinforcement bars is primarily deformed high-tensile strength steel bar and is typically placed in groups to increase the structural capacity. They are available up to 40mm in diameter with yield strength of up to 460 MPa. Steel pipe is mainly used ex-oil API (American Petroleum Institution) pipe which are high tensile strength steel pipe. Available sizes ranges from 60mm to 300mm in diameter with typical yield strength of 552 MPa for grade N80. Figure 10 shows the typical sections of micropiles for the two reinforcement systems.

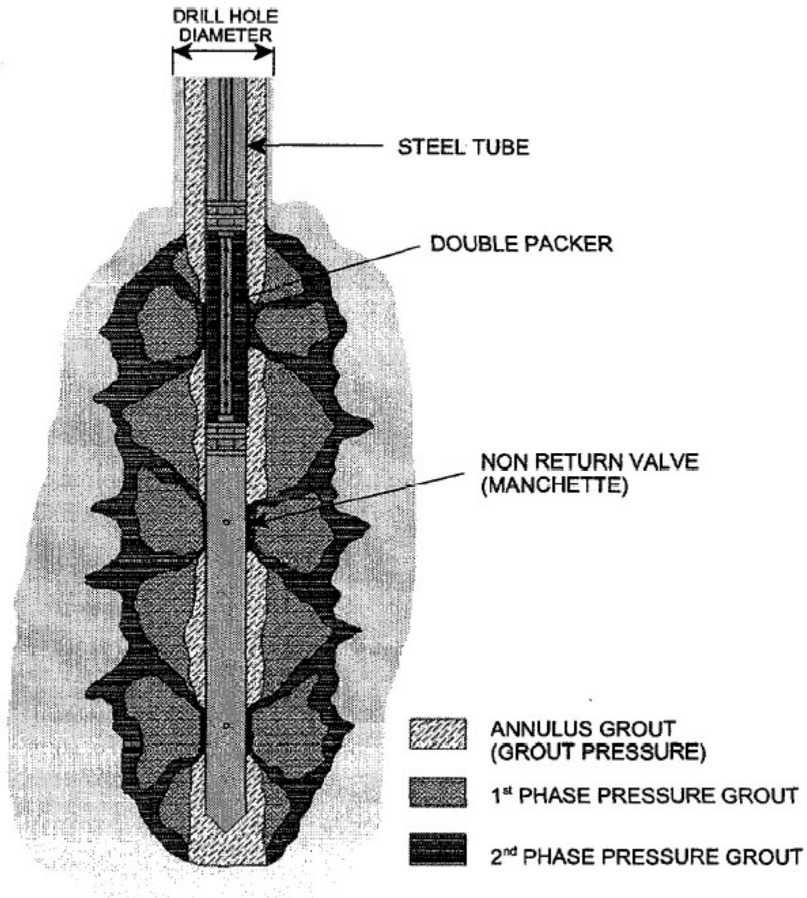


Figure 9: Use of Reinforcement Tube as a Tube á Manchette Post-Grouting

API pipe system is generally recommended for compression piles as it provides good lateral stability of the pile under axial compression load. Reinforcement bars are common for tension piles because of its resemblance with ground anchorage. Some designers also use reinforcement bars for compression piles with provision of sufficient helical links to avoid buckling of reinforcement bars and for piles in ground with good lateral support.

Other type of reinforcement includes the steel reinforcing bars that have a continuous full-length thread such as GEWI pile (Figure 11). The bar is continuously ribbed thread rolled and is available in diameters ranging from 19mm to 63mm with yield strength up to 550 MPa. The thread on the bars ensures grout to steel bond and as well as allow the bar to be cut at any point and joined with a coupler to provide full tension/compression capacity.

However, care needs to be taken for micropiles to carry large lateral load or high bending moments as the small pile cross-section will limit the development of sufficient shear and bending resistances. It can be economically inefficient option for such purpose.

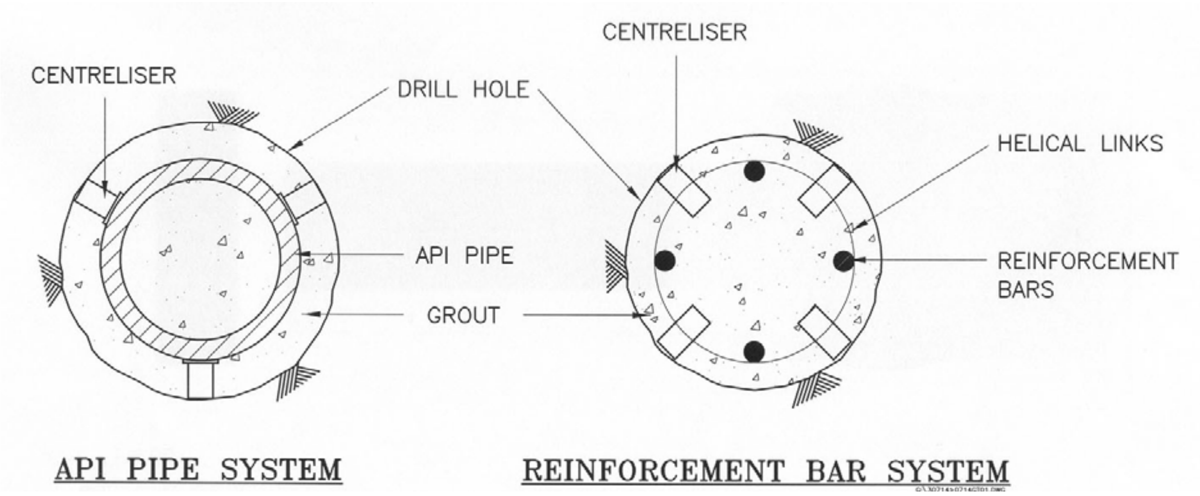


Figure 10: Typical Cross Sections of Two Common Micropile System

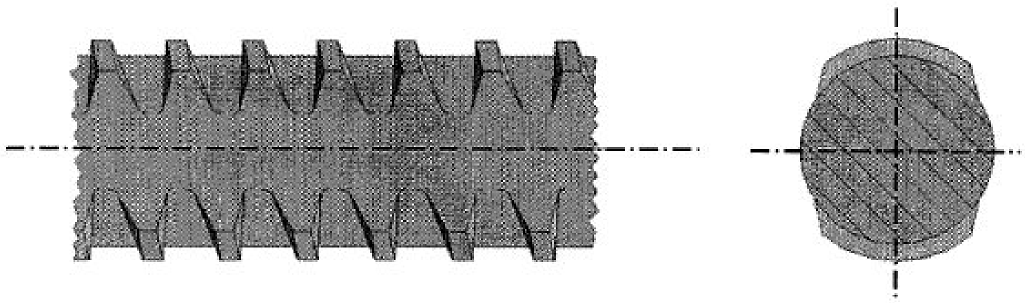


Figure 11: Details of Continuously Threaded Dywidag Bar (Armour etal., 2000)

3.0 DESIGN CONCEPT

Micropiles can be designed as rock socketed piles in rock formation and friction piles in weathered rocks or soils to carry either compression load or tension load. All micropiles are designed to transfer load through the shaft friction over a length of pile shaft to the founding medium. End bearing at the pile tip is generally negligible for the reasons of small base bearing areas, in which the axial load cannot be effectively transferred to the base. This design philosophy also inherently demands a founding medium with sufficient thickness to carry the imposed load from the micropile. If there is a cavity below the pile toe or the pile is socketed into a boulder, there will be some transfer of the load to the surrounding sound material by arching effect or to spread the load to the underneath soils. If large pile group is involved in these founding conditions, care needs to be taken to avoid punching shear failure of the rock slab or bearing failure of soils underneath the boulder causing excessive settlement under the entire pile group.

There is no specific design standard for micropile design, however, relevant design standards for each design components can be referred to in the pile design. These standards are BS 449, BS 8081, BS 8110 and BS 8004. However, working stress approach is still widely adopted for the pile designs in view of the compatibility between the structural and the geotechnical designs.

Minimum factors of safety (FOS) for both structural and geotechnical capacities are 2.0 as recommended in BS 449, BS 8004 and BS 8081, and are well accepted in the local practice. The FOS of 2.0 is also allowed in BS 8081 for anchors if full-scale field tests are available to validate the designs.

3.1 Structural Design

In practice, the design compressive stress in the steel reinforcements is limited to 50% of the yield strength. The pile capacity is normally derived from the allowable structural capacity of the reinforcements in the preliminary design. Other components, such as the grout and additional reinforcement bars can be included to enhance the allowable structural capacity. However extra care needed to ensure its effectiveness during construction.

For load transfer at the reinforcement/grout interface, an average ultimate reinforcement/grout bond stress with the appropriate safety factor, says 2.0, is used to derive the required bond length. As for anchorage designs, British Standard BS 8081 is commonly referred to select the ultimate bond stress. Table 2 shows the ultimate bond stresses between the cementitious grout of minimum compressive strength of 30,000kPa and reinforcement with different contact surface conditions.

Ultimate Bond Stress	Contact Surface Conditions
1,000kPa	Clean and Plain Bar or Wire
1,500kPa	Clean and Crimped Wire
2,000kPa	Clean Deform Bar
3,000kPa	Locally Noded Strand

Table 2: Ultimate Bond Stress Between Grout and Reinforcement (BS 8081)

The minimum required bond length should be 3m. This is to provide allowance for the uncertainties on the load distribution along the reinforcement/grout interfaces, as the actual stress distribution is uneven. Woods and Barkhodari (1997) have presented the load distribution at the interfaces as shown in Figure 13. The calculated reinforcement/grout bond length should be checked against the grout/rock or soil bond length as discussed later. The longer bond length should be taken as final bond length. However, one can also optimise the grout/rock bond length to match the other bond length by varying the size of drillhole.

If empty cavity or very soft slime zone is encountered, the buckling load should be considered for necessary downgrading of pile capacity in compression. The famous Euler formulae shown below can be used to calculate the buckling load depending on the end constraints. Figure 12 shows the possible end constraints for buckling piles under different cases.

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

where P_{cr} = Buckling load (kN)
E_p = Young modulus of equivalent pile section (kN/m2)
I_p = Moment inertia of equivalent pile section (m4)
L = Length of pile column without lateral support (m)
K = 1.0 for pinned ends, 0.25 for fixed ends (for the cases of cavity or slime zone), 0.7 for one fixed end and one pinned end (for the case of soft clay)

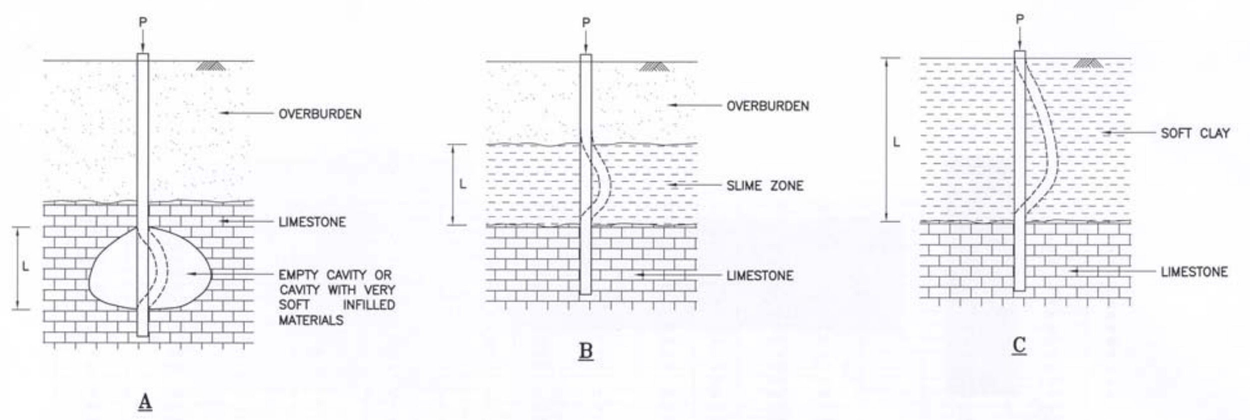


Figure 12: Buckling Modes of Micropiles

For micropiles through soft deposits and socketed into bedrock, the buckling load should be checked using the Winkler spring model as lateral support on pile under buckling mode. Al Although BS 8004 has suggested that buckling of a pile is not a concern for the pile in the soils with undrained shear strength larger than 20kPa, this may not be applicable for micropile because of its large slenderness ratio and high carrying load.

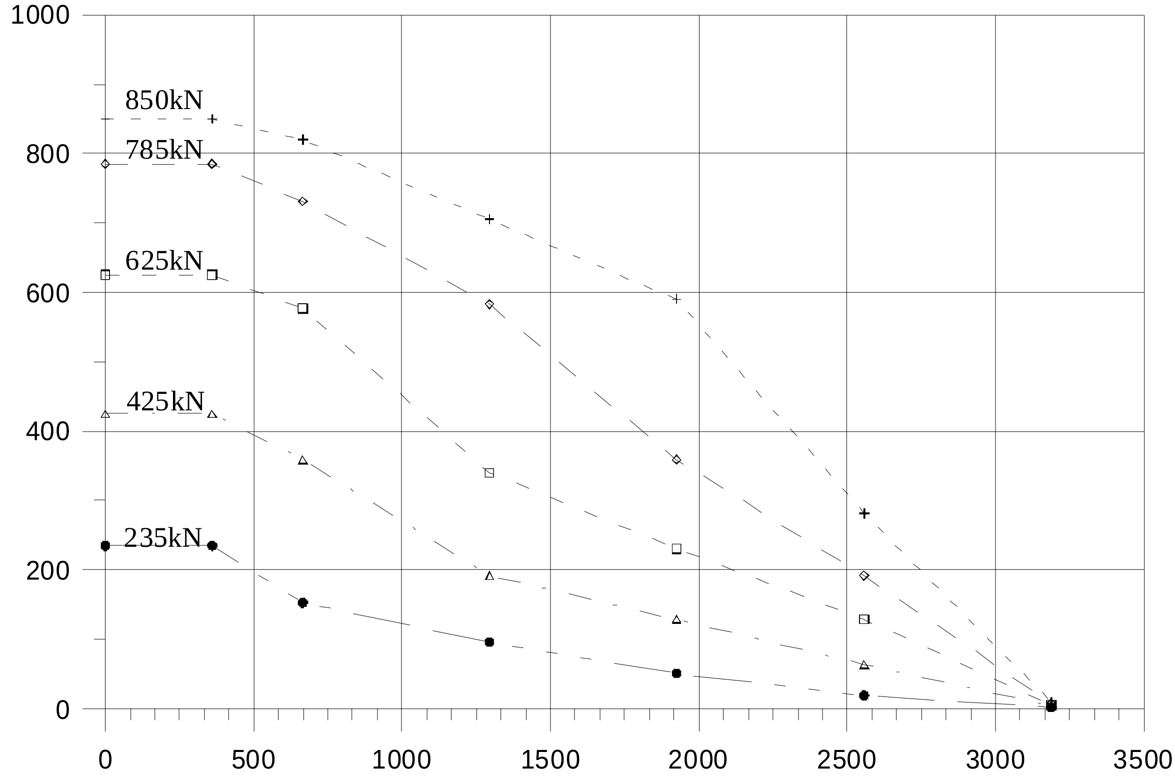
BS 8081 requirements for cementitious grout mix design are summarised and tabulated in Table 3.

Requirements	Range of Limits
Total Sulphate Content	≤ 4% of cement in the grout
Total Chloride Content	≤ 1% of cement in the grout
Bleeding of Grout at 20°C	≤ 2% of the volume 3 hours after mixing and have a maximum of 4% of the volume in less permeable soil
Water/Cement Ratio	0.35 ~ 0.6 in normal soil conditions ≤ 0.45 for low permeable soil

Table 3: Requirements for Cementitious Grout Mix Design

The cement content for grout mix depends on the sulphate content and is recommended in BS 8081 (Table 10) in the range of 250 to 380kg/m^3 for 5 categories of sulphate contents. BS 8004 demands the minimum cover of 40mm for foundation piles. The minimum spacing between reinforcements for grout is 10mm. In practice, anti-shrinkage additives are recommended to reduce grout shrinkage.

Strain compatibility between the unconfined grout and the reinforcements has seldom been considered in the design. In view of the relatively high design axial stress (50% of the yield stress of the reinforcement) is usually adopted for the reinforcement, hence the primary load carrying element in micropile is the reinforcement instead of grout. This is acceptable for pile section above founding level from the structural point of view. However, when reaching the load transfer stratum, the grout in the annulus between the reinforcement and founding medium, as a bonding medium, plays an important role of transferring axial load from the reinforcement to the supporting medium. Therefore the grout must be in good integrity and intact to transfer the load. If the grout failed in crushing due to excessive compressive stress before the reinforcement reaching the design axial stress, progressive debonding at the grout/reinforcement interface is then expected, hence increasing the elastic deformation at the debonded pile segment and reducing the load transfer efficiency at the grout/soil interface. This is particularly dangerous for micropiles with the bars system under compressive load as the bar reinforcements will buckle due to insufficient confinement by the crushed grout.



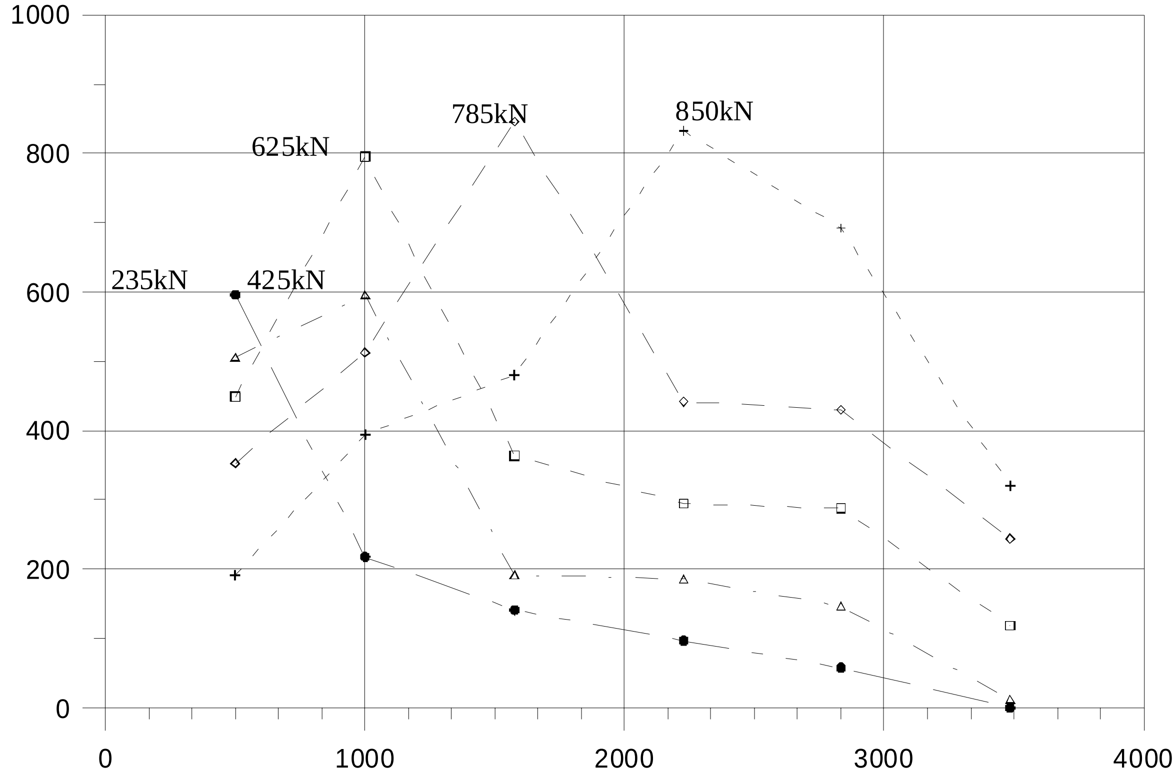


Figure 13: Load Distribution of Anchorage (Wood & Barkhordari, 1997)

Strain compatibility between the two load transfer elements, namely the reinforcements and grout can pose uncertainties to the design. The following analysis will demonstrate the compatibility problem for the micropile with inadequate lateral confinement to the pile.

Assuming :

Yield strength of steel reinforcement (API),	f_y = 552,000kPa
Young Modulus of steel reinforcement,	$E_S = 210\text{x}10^6$ kPa
Characteristics strength of grout,	f_{cu} = 30,000kPa
Young Modulus of grout, E	$_g = 28\text{x}10^6$ kPa

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

$$\varepsilon_s = \frac{\sigma_s}{E_s} = \frac{0.5\text{x}552{,}000\ \text{kPa}}{210\text{x}10^6} = 1.314\text{x}10^{-3}$$

For strain compatibility, the grout shall have the same strain with the reinforcement and will be under the compressive stress calculated as follows:

$$\sigma_g = E_g \varepsilon_s = 28\text{x}10^6\, kPa\ \text{x}\, 1.314\text{x}10^{-3} = 36800 kPa$$

The calculated compressive stress based on linear elastic theory obviously exceeds the compressive strength of the grout (30,000kPa). Although the compressive strain limit for concrete is well recognised to range from 2.0×10^{-3} to 3.5×10^{-3}, it is believed that yielding of grout at the above calculated compressive strain level may occur. Two failure mechanisms can be expected as the strain of the reinforcement reaching the strain limit of grout. First is the crushing of grout body under excessive compression. Second is the yielding at reinforcement/grout interface. It is the authors' opinion that the second failure mechanism will likely to happen, because the adhesion of most normal material is always lower than the cohesion, which is an indication of grout strength.

The ultimate bond stresses given in Table 2, which are significantly lower than the grout strength, can substantiate this view. Once the overstressing occurs, the yielding of grout/reinforcement interface will propagate to deeper depth until the stress level in the grout under lateral confinement drops below the limit. In the paper by Neoh (1996), has indicated the peak side shear propagating downwards with increasing axial load in three instrumented test piles. The phenomenon of grout bursting can be observed near the pile top during failure load test if the pile head not protected by steel casing. Yielding of the interface is expected to be insignificant for micropiles socketed in sound rock. This is because the confinement provided by sound rock and the axial strain in the micropile attenuates very rapidly with depth at the rock socket. The design implications of the interface yielding are largely elastic shortening, reduction of effective composite section and unsatisfactory load transfer at the yielding portion of pile to the ground. If one is to design frictional piles in soils, care to be taken to minimise the yielding. Similar concept can be applicable to tension piles. The effective solutions to this problem for piles under compression are as follows:

1. Reduce the pile axial stress to an acceptable strain limit of grout by downgrading of pile capacity or increase the reinforcements,
2. Provide permanent steel casing to confine the grout as higher strength and stiffness are experienced in full confinement of any material.

Stiffening effect of grout being confined by permanent casing or API pipes or close helical links for reinforcement bars can substantially reduced the elastic deformation of the micropiles under axial load. However, if inner surface of the API pipe contains grease or other debonding agent, the inner grout will have virtually no contribution to the stiffening effect.

Differential settlement due to elastic shortening of micropiles with various pile lengths at different pile caps should also be considered. In association with this problem, pile cap analysis is also required to check the foundation load distribution among the long and short piles within the same pile cap.

Checking of elastic shortening on the micropiles is essential in order to comply with the specified settlement performance. In the case of long pile, downgrading of pile capacity or additional piles may be required for the compliance of settlement and bearing capacity criteria.

3.2 Geotechnical Design

Ultimate bond stress at the grout/rock or soil interface is dependent on strength of grout, rock or soil strength, quality of contact interface resulting from drilling, cleaning and grouting operations. The ultimate rock/grout bond stress for various rock types can refer to Table 25 of BS 8081. Allowable design bond stresses at grout/rock interface and the unconfined compressive strength for mudstone, shale and sandstone. Seidel and Harberfield (1995) present a pile socket design approach in rocks and hard soils in relation to the two critical factors, namely shaft roughness and socket diameter. Neoh (1996) indicates that working bond stress of 650 kPa is commonly adopted for rock socket design in limestone areas in Malaysia. However, the recorded bond stress could be as high as 1600kPa in limestone. For soil friction piles, the conventional design method for cast-in-situ bored piles based on SPT-N (Standard Penetration Test) values can be used to derive the ultimate side shear stress, i.e. $f_{s,ult} = 2.0 \times \text{SPT-N (kPa)} \leq 120\text{kPa}$ (Chang and Broms, 1991). Toh et al (1990), and Buttling and Robinson (1987) also present the bored pile designs for sedimentary soils and residual soils respectively.

For rock socketed piles, the contribution of the overburden soil to the pile shaft is usually small and ignored due to difficulty in assessing the mobilised shaft resistance under the working conditions and comparatively lower shaft resistance in the soil. Even the overburden shaft friction could be mobilised, the relative movement between the pile shaft and the overburden soil will be significant at the top due to the overall elastic shortening of piles, and reducing towards the rock socket. In most cases, the soil strength increases with depth, therefore, the mobilised friction will be at the upper weak soil layer and hence is insignificant. This simplification of design is valid for short pile. Significant contribution of shaft friction from the overburden is observed for long pile as reported by Neoh (1996). Analytical methods, such as elastic method (Puolos and Davis, 1968), finite element method (Osterberg and Gill, 1973), and load-transfer method (Coyles and Reese, 1966) with established database in local soils, can be used to estimate the mobilised shaft friction in the overburden soils for long rock socketed piles. Another simple approach is to apply an overall FOS to the estimated ultimate shaft friction in the overburden soils to arrive at the mobilised shaft friction under working load. However, validation of the FOS with fully instrumented test piles is required for the various ground conditions.

Chan and Ting (1996) present a new design approach to improve the pile capacity and performance of friction piles in soil, weathered and fractured rock by pressure grouting. Two methods of grouting, namely IRS (Injection, Repetitive and Selection – multi-point and multi-stage grouting) and IGU (Injection, Global and Unitary) are introduced. The IRS method yields a better improvement than the IGU due to the stringent grouting procedure for IRS method and is recommended for medium to stiff clayey materials and for fractured and karst formation. Design charts of ultimate shaft resistance in relation to SPT-N and Menard Limit Pressure (Pl) are presented for both grouting methods. Troughton and Stocker (1996) observe an increase of 50% to 60% of ultimate pile capacity for a series of trials on the 570mm-diameter shaft grouted piles. Grout mix with water cement ratio of 0.4 to 0.5 and grouting pressure up to 8,000kPa were used in the grouting operation for the trials. A general trend of the test results indicates that the ultimate shaft friction increases with reducing pile size for both cohesive and cohesionless soils. This probably explains the dilatation effect more pronounced in small piles. Pressure grouting improvement is useful in remedial works for foundation piles under distress and found on weak to medium strong soils with site constraints.

Unlike in bored pile, soft toe is not an issue for micropiles although micropiles are generally designed based on grout/rock or grout/soil friction. The contact area of the micropile reinforcement is very small compared to the drilled hole and this allows the reinforcement penetrate the layer of soft materials at the pile toe and in contact with the hard stratum. Moreover, the capacity of the micropile is designed based on the reinforcement in contrast to bored pile where both the reinforcement and concrete is taken into design considerations. Therefore, load tests on micropiles are usually performed to it's intended working load.

4.0 PILING PROBLEMS ASSOCIATED IN THE INSTALLATION METHODS

Recent years, rotary percussive duplex or percussion hammer (down-the-hole hammer) is gaining popularity in the local micropiling industry. This is because of its robustness, fast penetration, good drillhole verticality, neat operation (no spilling of stabilising fluid over the working area apart from groundwater) and good drillhole protection by advancing steel casings with the hammer simultaneously. The problems with the percussion hammer are the vibration and blowing out of excessive earth material, particularly in cohesionless soils. This is because the percussion hammer is drilling ahead of the protection casing, hence, flushing out of excessive materials forming voids. If the ground is sensitive to vibration, for instance, densification of loose sandy soil, ground movements are to be anticipated. Figure 14 shows the process of percussion drilling and blow out of excessive material.

This effect of excessive ground disturbance from the use of percussion hammer can be demonstrated in the monitoring results of a bridge structure founding on sandy stratum, in which micropiling works with percussion hammer technique were carried out near the structure as illustrated in Figure 15. Substantial movements on the bridge structure were observed while the micropiling works were carrying out. As the piling works stopped, the movements stabilised. Another possible explanation for these movements is that the percussion technique blowed out excessive material and creates voids larger than the size of the protection casing during drilling operation. The collapse of voids can induce overall ground movements and propagate to the surface and the adjacent areas. Subsequently in the project, the drilling method was changed to rotary drilling with drilling fluid to reduce the ground movements. The comparison of boreholes carried out before and after the piling works indicates the potential soil loosening as shown in Figure 16.

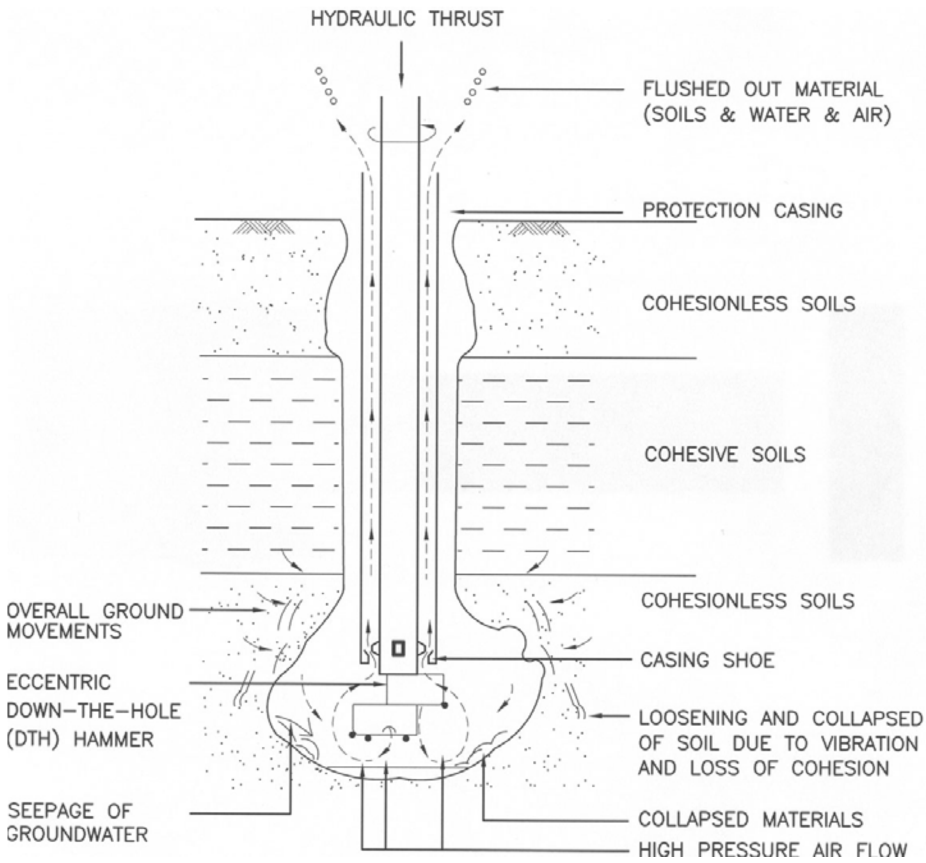


Figure 14: Process of Rotary Percussion duplex (Percussion Drilling)

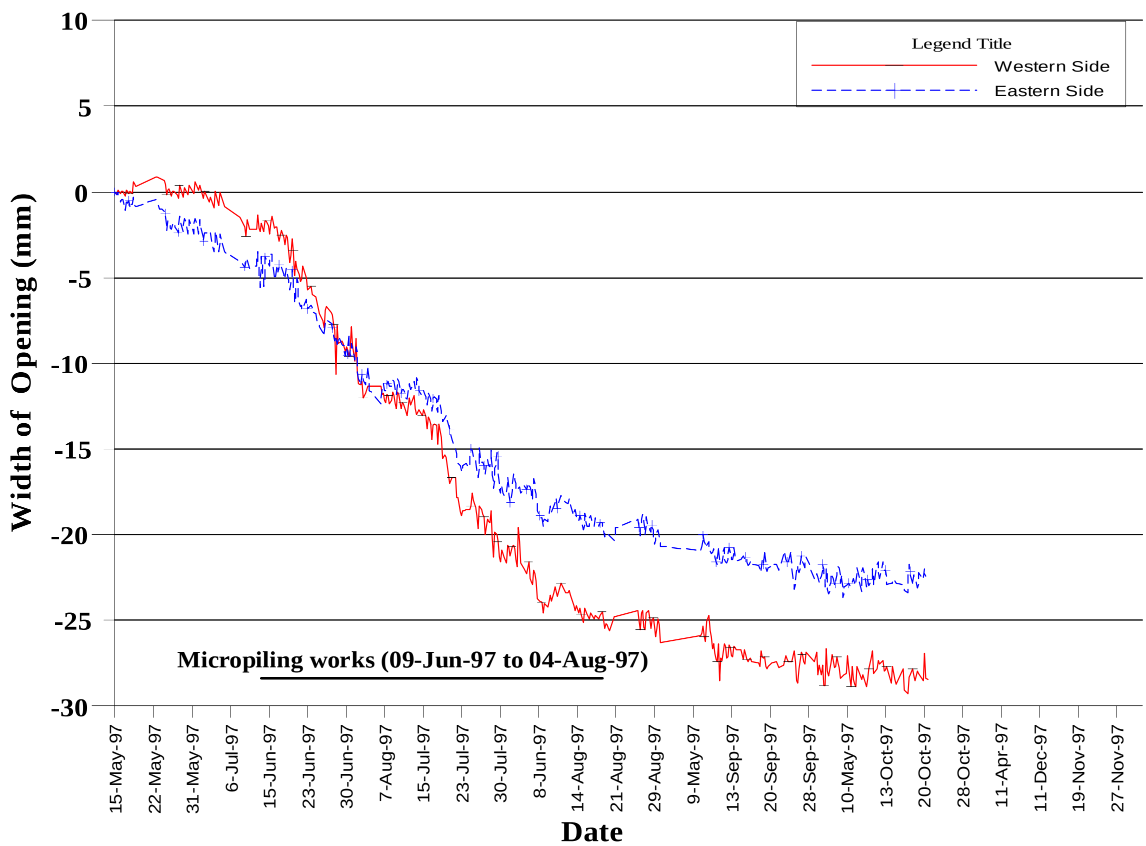


Figure 15: Opening of Bridge Expansion Joint at Abutment

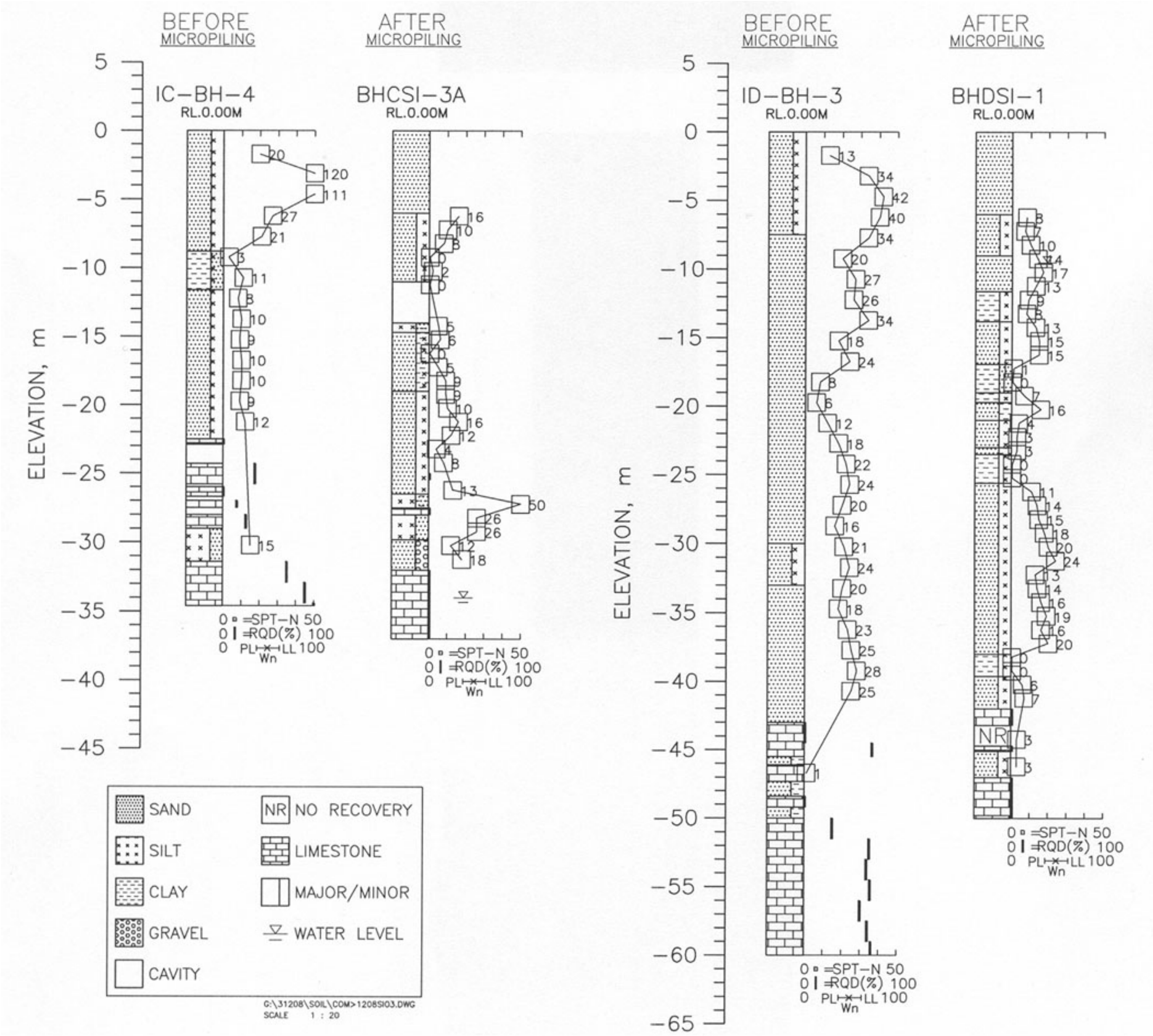


Figure 16: Comparison of Borelogs Before and After Micropiling

5.0 CONSTRUCTION CONTROL

The success of micropiles highly relies on the quality of pile installation particularly the installation method. The construction controls to ensure a successful pile installation are given as follows:

1. As it is very difficult to determine the rock conditions for every pile, hence, visual inspection on the rock chipping by experienced supervising personnel is useful in determination of the degree of weathering, indicative rock strength, rock mass structures and/or karst features. Recording of the socket penetration rate, and calibrated to the borehole information, and the hydraulic pressure applied on the drill shafts can provide indication of rock quality. Change of water level or stabilising fluid may indicate existence of cavity, solution channels and permeable layer where excessive grout loss is anticipated. Change of hydraulic pressure or sudden drop of drill shaft may also indicate karst features, boulders or hard pans.
2. Measures should be taken to avoid drillhole collapse by means of temporary protection casing or /and stabilising fluid.
3. Grouting should be carried out immediately after cleaning of drillhole by flushing the drillhole with clean water.
4. Permanent casing can be used to minimise excessive grout loss. Alternatively, using of rapid hardening grout to seal the flow channel could be considered.
5. Proper connection ensuring both ends of the pipes in full contact for coupler and threaded joints and sufficient lapping of reinforcement bars is important to ensure efficient load transfer between the reinforcement. At coupling or reinforcement lapping, it is recommended to stagger the coupling or lapping to avoid weak section.
6. Centralisers of reinforcements are important elements to assure adequate grout cover for the bonding of interfaces.
7. Excessive welding on high yield steel reinforcement should be avoided as heat can alter the chemical and physical properties of the material.
8. Grease or coating on reinforcement should be removed to ensure good bonding. However, cleaning of the debonding material at the inner surface of the pipes is very difficult.
9. Provision of holes should be allowed at the tip of API pipe to facilitate grouting between the drillhole and API pipe.

6.0 PERFORMANCE OF TEST PILES

Two test pile results are presented here to demonstrate the performance of the micropiles carrying working load of 1200kN in different founding formations. The two test piles (TP-B22 and TP-C27) are 250mm diameter, and are reinforced by Grade N80 API pipe of 177.8mm outer diameter and 10.36mm thick. The details of the two test piles are shown in Figure 19. The upper 13.5m pile section enlargement is to strengthen the flexural rigidity of piles undertaking significant lateral load and bending moment. The permanent casing extended to the completed weathered granite and additional reinforcements (4Y40 bars) of 24m long at the socket portion in test pile TP-B22 are to reduce the elastic strain and avoid potential grout crush or interface debonding respectively. The performance of the test piles is tabulated in Table 4 and Figures 17 and 18. The two test piles results are almost identical except residual settlement during unloading, although one is rock socket pile (TP-C27) and the other is soil friction pile (TP-B22). There is almost no residual settlement for test pile TP-C27 during the first cycle of unloading.

Test Pile	Pile Head Settlement			
	1×WL (1200kN)	First Unloading	2×WL (2400kN)	Second Unloading
TP-B22	6.7mm	2.6mm	20.4mm	4.5mm
TP-C27	6.3mm	0.3mm	20.1mm	1.5mm

Table 4: Summary of Test Piles Results

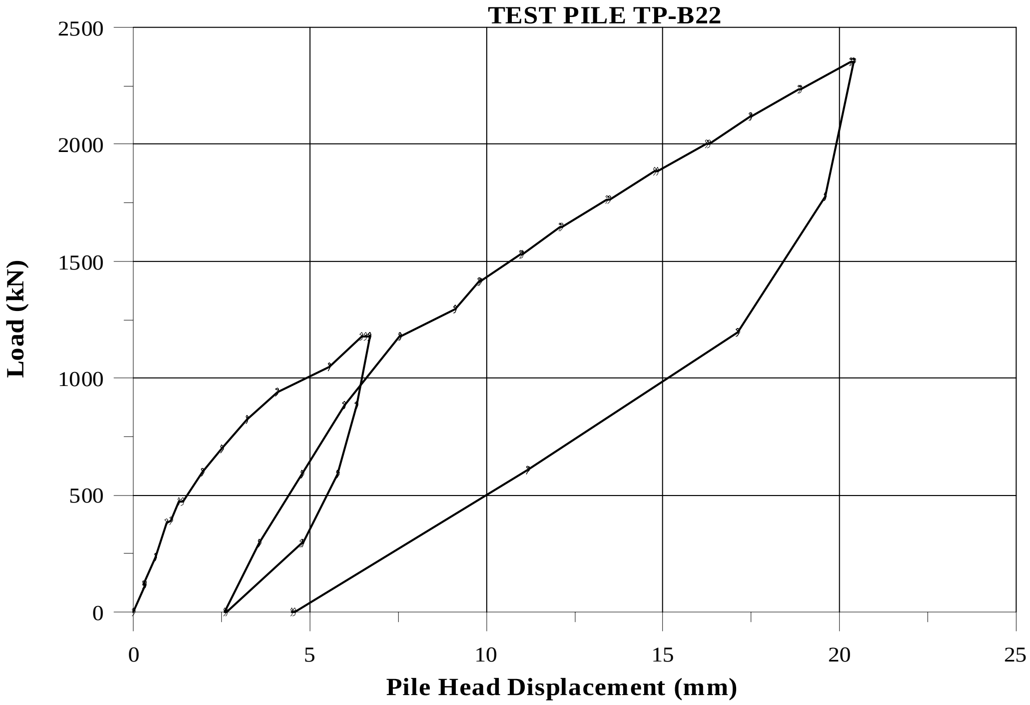


Figure 17: Load Settlement Results for Test Piles TP-B22

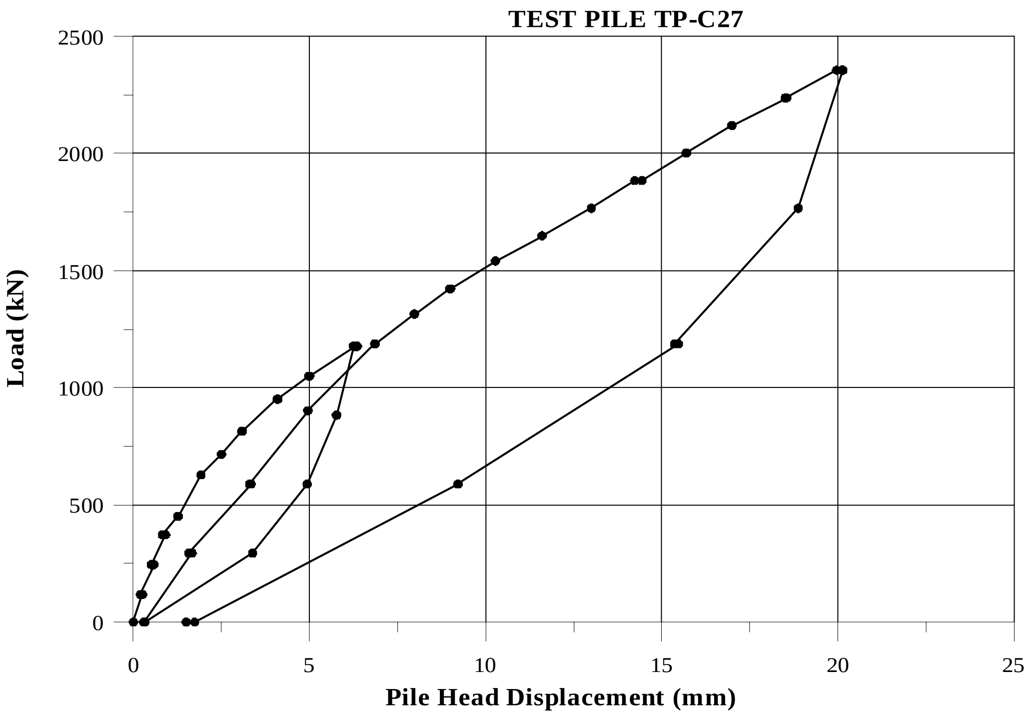


Figure 18: Load Settlement Result for Test Pile TP-C27

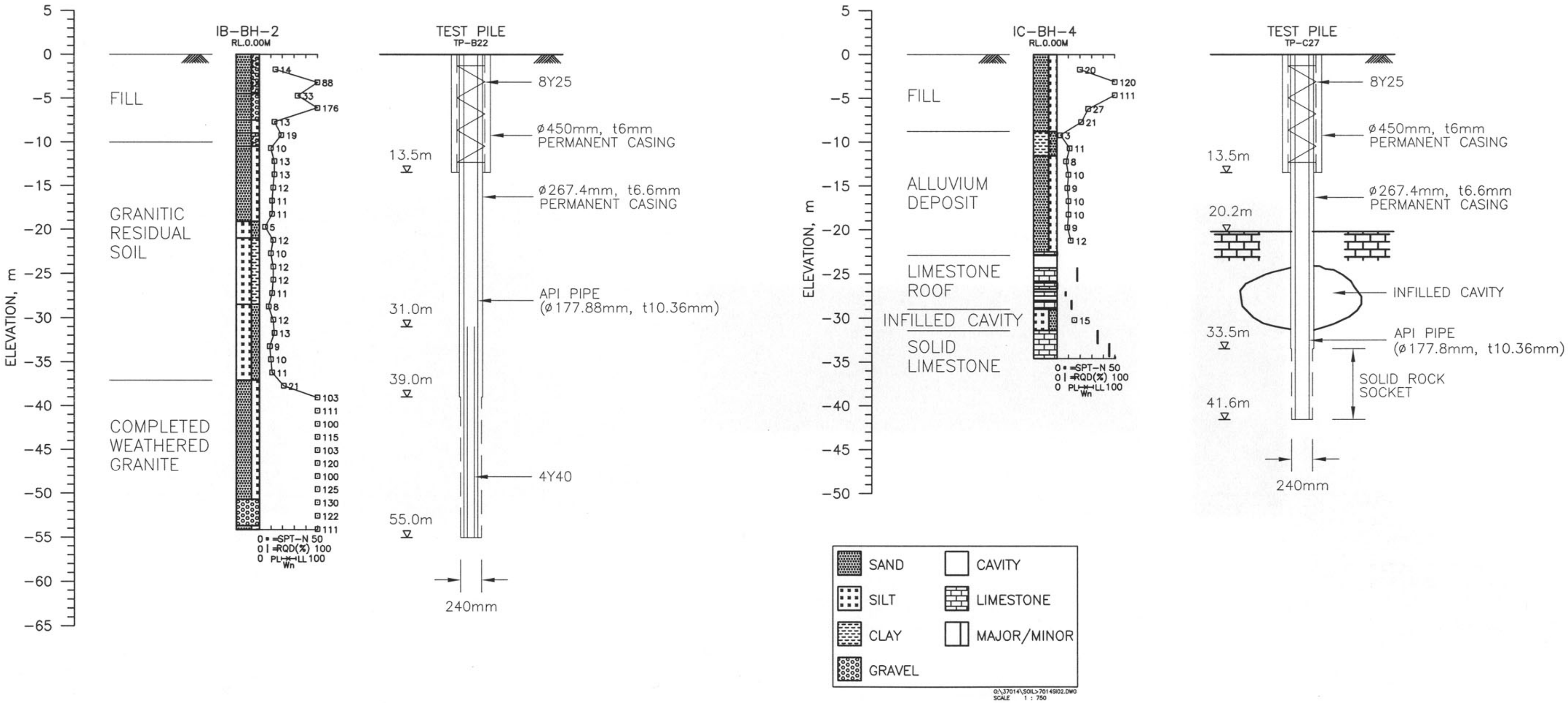


Figure 19: Details of Test Piles and Subsurface Conditions

7.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are summarised on the applications of micropiles, design aspects, construction methods and control and associated problems,.

1. Micropiles can be used as normal foundation piles and compensation piles for remedial works, especially in area with site constraints. Micropiles can be designed as either rock socketed piled or soil friction piles. API pipe system provides good compression performance in terms of lateral stability and vertical movements. Tension piles can be economically reinforced by bars system. Micropiles can be a costly option to support lateral load and huge bending moment.
2. Percussion drilling technique can be applied in most micropile construction except in sensitive ground, particularly in cohesionless soils. In the case of sensitive ground, rotary drilling is highly recommended with temporary casings or/and stabilising fluid.
3. Factors of safety for both geotechnical and structural designs should be at least two.
4. Buckling load should be checked in soft overburden and very soft of loose infilled cavities.
5. Elastic deformation of micropile is generally large due to high stress utilised on the reinforcements, especially for long piles. Downgrading of pile capacity and additional piles may be required to reduce the differential elastic settlement between the short piles and long piles. Pile group analysis is also required to check the load distribution among the short piles and long piles.
6. Stain compatibility between reinforcements and grout for micropiles in soils should be examined. Suggestions to reduce the elastic strain in the reinforcement by down-grading pile capacity or increasing the steel content for the pile section.

Note: The sample specifications for bored piling, testing of bored piling and checklist for construction of bored pile are attached in Appendix for further reference. Many other specifications, checklists and technical papers prepared by Gue & Partners Sdn Bhd can be downloaded from our website at ***www.gueandpartners.com.my***.

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APPENDIX A

Sample Specification for Micropile

MICROPILE SPECIFICATION

1.0 GENERAL

The Contractor shall supply, install and test micropiles shown on the drawings or specified herein in accordance with the specification

The Contractor shall allow for all necessary operations including cutting through concrete slabs, scaffolding, platforms, handling equipment, tools machinery etc necessary for the expeditions handling of the work.

1.1 Setting Out

The Contractor shall be required to employ an approved Licensed Surveyor who will set up the positions of the piles as shown in the pile layout plans of the detailed design. The Contractor will be responsible for the accuracy of location and positioning of each pile. Any errors in setting out and any consequential loss to the Employer will be made good by the Contractor to the satisfaction of the Engineer.

The Contractor shall preserve the pegs set out by the Surveyor. Should any peg be displaced or lost it must be replaced by a Licensed Surveyor to the approval of the Engineer. Upon completion of all piling works, the Contractor shall produce as-built Drawings showing the positions of all piles as installed. The positions of piles shall be verified by a Licensed Surveyor.

1.2 Tolerances

Position

The pile heads shall be positioned as shown on the Drawings within a maximum deviation of 40mm in either direction from correct centre point.

Verticality

For bored cast-in-situ piles, the maximum permitted deviation of the finished pile from the vertical at any level is 1 in 150. The contractor shall demonstrate to the satisfaction of Engineer the pile verticality is within the allowable tolerance.

Correction

Should piles be installed outside these tolerances affecting the design and appearance of the structure, the Contractor shall propose and carry out immediate remedial measure to the approval of the Engineer.

1.3 Person in Charge

The piling work is to be carried out by full time operators and supervisory staff who must be experienced in the installation of the proposed type of piles.

The Contractor shall submit to the Engineer for approval, written evidence to show that the persons who will be engaged in the works have had such experience.

1.4 Piling Equipment and Accessories

The equipment and accessories must be capable of safely, speedily and efficiently installing piles to the design requirements at the project site.

Sufficient units of equipment and accessories must be provided to keep to the agreed construction schedule.

1.5 Sequence of Installation of Working Piles

The Engineer reserves the absolute right and the Contractor shall recognise such right to direct the installation of working piles in any sequence the Engineer deems necessary for the satisfactory completion of the works.

8.1.1 2.0 SCOPE OF WORKS

The contract comprises the provision of all labour, materials, tools, plant etc necessary for the following work :

a. Supply and installation of pile foundations to carry the loads as specified in the drawings.

b. Stripping and cutting the piles to cut off levels specified and preparation of the pile head as shown.

c. Carrying out standards load test as specified.

3.0 MATERIALS

3.1 Reinforcement

The type of reinforcement to be used, the diameter and/or thickness, grade, yield strength and stress shall be as specified or as shown on the Drawings.

8.1.1.1

8.1.1.2 3.2 Grout

Unless otherwise specified, the grout shall be non-shrink cement grout. The grout mix design such as the water-cement ratio, the minimum cement and grout strength at 7 and 28 days shall be as specified and shown on the Drawings.

Grout shall be tested in accordance with BS 1881 and BS 4550. Maximum bleed shall be limited to 5%.

If admixtures are used, details of admixtures shall be submitted to the Engineer for approval before commencement of works. The use of the admixture shall fully comply with the manufacturer's instructions.

If the grout cube as tested failed to satisfy the criteria as prescribed in Specification and drawings, the piles constructed using this batch of grout shall be rejected. The Contractor shall undertake all necessary additional and consequential remedial/compensatory work to the approval of the Engineer.

4.0 SITE AND ADJACENT PROPERTIES

8.1.1.3

8.1.1.4 4.1 Subsoil Data

The soil investigation report is included in the tender documents only for information and guidance to the tenderers, and shows the approximate nature of the strata as known to the Engineer. The Employer and Engineer shall not be liable for the accuracy of the data given and the Contractor may carry out his own soil investigation to obtain additional information.

8.1.1.5 4.2 Site Visit

The tenderer is advised to visit the site to acquaint himself with the site conditions and no claims for inadequate information regarding site conditions will be entertained at a later date.

The system or systems put forward by the tenderer shall be well known. The adequacy of any system and its approval shall be at the discretion of the Engineers.

4.3 Underground Services And Adjacent Property

The Contractor shall take care to ensure the safety of underground services and adjacent properties during the installation of micropiles. The contractor will be liable to any claims of damage to the piling operations.

5.0 DRILLING OPERATIONS

5.1 Diameter of Piles

The diameter of piles shall not be less than the specified/designed diameter at any level throughout its length.

8.1.1.6 5.2 Drilling

The Contractor shall submit to the Engineer details of drilling equipment and drilling procedure for approval before commencement of works. Drilling operations shall be carried out in accordance with the relevant requirements as follows:

(a) Boring near recently Cast Piles

Piles shall not be bored next to other piles which have recently been cast less than 24 hours or contain unset grout, whichever longer to avoid damage to any of these piles.

(b) Stability of Drill holes

It is held that the Contractor has allowed in the unit rate of the pile for the implementation of all necessary measures, including the provision of all materials, labour and plant, for maintaining the stability of the sides of boreholes during Micropile installation and successful completion of the piles. The Contractor shall submit his proposed methods for agreement prior to commencement of boring operations.

Irrespective of the presence of ground water, the sides of all borehole shall be kept intact and no loose material shall be permitted to fall into the bottom of the boreholes. The Contractor's boring equipment shall be able to sink a steel casing to support the sides of all boring.

If the sides of boreholes are found to be not stable, temporary steel casing shall be driven into stable stratum. The borehole shall be filled with drilling fluid to a level sufficiently to stabilise the boreholes.

If ground water is found in any hole in sufficient quantity or gushing out as to affect boring operations or excavations and removal of soil from the boreholes, or the sides of boreholes collapse, then a steel casing of appropriate size and length in conjunction with stabilising fluid or other alternatives of sufficient strength shall be used to support the sides of the borehole and permit boring operations to proceed smoothly and safely. The proposed drilling fluid mix must be submitted to the Engineer for approval.

Excavations shall not be exposed to the atmosphere longer than is necessary and shall be covered at all times when work is not in progress. Pile excavated shall be casted within 24 hours unless otherwise agreed by the Engineer.

In the event of a rapid loss of drilling fluid from the borehole excavation and caused instability of bore, the excavation shall be backfilled without delay or other appropriate and approved remedial measures taken by the Contractor like installing temporary casing prior to resuming boring at that location. The cost of redrilling of the hole shall be borne by the Contractor.

(c) Stability of bore by Temporary Casing Method

Where the use of a temporary casing is required to maintain the stability of a bore, the bottom of casing shall be kept a minimum of 1 metre or more below the unstable strata to prevent the inflow of soil and the formation of cavities in the surrounding ground.

Temporary casings shall be thin walled mild steel cylindrical casing. The dimensions and quality of the casing shall be adequate to withstand without damage or distortion all handling, construction and ground stresses to which they will be subjected. The casings shall have an internal diameter not less than the specified pile diameter. They shall be free of significant distortion, of uniform cross-section throughout each continuous length and free from internal projections and encrusted grout which might prevent the proper formation of piles. The joints of casings shall be watertight.

If temporary casings are damaged during installation in a manner which prevents the proper formation of the pile, such casings shall be withdrawn from the bore before grout is placed, repaired if necessary, or other action taken as may be approved to continue the construction of the pile.

(d) Rock Coring

Rock coring shall means coring of sound bedrock. Coring of rock other than two items specified below shall not be considered as coring in rock, and will only be considered as boring in soil.

(i) Rock socket length
(ii) Cavity roof

Coring of inclined rock surface, limestone pinnacles, cavities and soil below boulder/floater shall be considered as boring in soils.

(e) Inspection of Pile Excavation

Where practicable, all pile excavations shall be inspected for their full length before grouting. The Contractor shall provide all the apparatus necessary for the inspection. In the course of inspection any loose or soft material in the borehole which is likely to affect the performance of the pile shall be removed to the satisfaction of the Engineer.

6.0 GROUTING OPERATIONS

6.1 Mixing and Placing Grout

The Contractor shall provide details of the method and equipment used in grout mixing. Further information such as grouting pressure, grouting procedure, grouting equipment and technique employed in grouting underwater shall also be furnished for approval.

Grout shall be mixed on Site and shall be free from segregation, clumping and bleeding. Grout shall be pumped into its final position in one continuous operation as soon as possible and in no case more than half an hour after mixing.

Micropile shall be grouted in one continuos process. If there is significant loss of grout, the Contractor may choose to carry out pre-grouting in stages as necessary to prevent further loss of grout for the construction of micropile. Method statement of pre-grouting including details of equipment, materials and procedures have to be reviewed and approved by the Engineer. If after the process of pre-grouting and re-drilling of the hole is required. The Contractor has to bear the cost and time of the pre-grouting and re-drilling.

8.1.1.7 6.2 Grout Falls

The lost of flushing mediums of either water or drilling mud drilling will demonstrate potential excessive grout loss or falls. Depending on its seriousness, the Contractor can decide to carry out a water tightness test to decide whether pregrouting is required. The cost and time of the test will be borne by the Contractor. Pregrouting an re-drilling shall be carried out if results of the test shown that leakage exceeds 5L/min at an excess head of 0.1 Mpa, measured over a period of 10 minutes.

7.0 CONSTRUCTION OF PILE HEADS

7.1 Lengthening of Piles

Where lengthening is required, the pile reinforcement unit shall be connected on Site to the details shown on the Drawings.

Other means of jointing reinforcement shall be to the approval of the Engineer.

7.2 Cutting and Preparation of Pile Heads

Pile heads shall be constructed to the details as shown on the Drawings.

8.1.1.8 8.0 STANDARDS

All materials shall be of the best quality and new. All piling work shall be executed in accordance with the approved designs prepared by the Contractor and to the approval of the Engineer.

8.1.1.9 8.1 Standard Load Tests

Load test of two (2) times the working loads shall be carried out on piles designated by the Engineers and in accordance with BS 8004. The number and location of test piles shall be at the discretion of the Engineer. The Contractor shall submit a detailed proposal of the load tests to the Engineer and shall obtain his approval in writing before carrying them out. On completion of the test, the Contractor shall submit to the engineer the results including graphs showing load and settlement versus time and settlement versus load.

The test procedure shall be as specified in Specification.

Failure to standard load test shall be as specified in Specification.

9.0 TEST REPORT

The report shall contain the following : -

a. Pile designation, date completed, weather condition, pile length, pile size, volume of grout intake, time of drilling at intervals not greater than 4m and time to grout the pile.

b. Description of the apparatus used for testing, loading system and procedure for measuring settlement.

c. Field data

d. Time/Settlement Curve

e. Load/Settlement Curve

f. Remarks explaining unusual events or data and movement of piles.

g. Calibration certificates of dial gauges and pressure gauges.

h. The format of record shall be approved by the Engineer.

8.1.1.10
8.1.1.11 10.0 DAMAGED OR DISPLACED PILES

Should the deviation exceed the tolerance provided in this specification, the contractor shall submit this remedial proposal for the approval for the Engineer. Failing this, the faulty pile shall be replaced by additional piles as necessary in positions as determined by the Engineer at no cost to the Employer. The cost of modification to pile cap etc., if any, shall be borne by the Contractor. The same will also apply to any piling work rejected by the Engineer for not truly constructed and installed in accordance with the specification.

Where a pile has been damaged during installation, testing or by other causes, the damaged pile shall be considered and treated as a faulty pile and should be replaced by additional piles as approved by the Engineer at the Contractor's expense.

10.1 Forcible Correction Not Permitted

Where piles have not been positioned within the specified limits no method of forcible correction will be permitted.

8.1.1.12
8.1.1.13 11.0 PAYMENT

8.1.1.14 11.1 Unconcreted (Empty) Bore

The unit rate of the pile shall be deemed to include whatever empty bore above the cut-off level of the pile and re-drilling after pre-grouting. No claims will be considered for any empty bore and re-drilling, and the Contractor shall allow in tender for the cost of these processes due to his sequence of construction.

8.1.1.15 11.2 PAY LENGTH

For all proposed pile, the Contractor shall be paid only for the length of installed pile measured from toe of the pile to the cut-off level. The same applies for the grout. Pre-grouting, grout loss, over drill, re-drilling will have to be borne by the Contractor and shall be deemed to have included in the rate.

8.1.1.16 12.0 PILING RECORDS

Complete piling records shall be kept by the Contractor during pile installation. The Contractor shall submit the following in duplicate to the Engineer:

a. Records of all piles as the work proceeds.

b. Upon completion, a record of the work as carried out and as-built drawing.

The format of the record shall be approved by the Engineer.

The record shall contain all information required by the Engineer which includes the following where applicable :

- reference number and position of pile
- type and dimension
- date of boring and nature of strata where each pile is bored
- details of equipment used
- ground level and base of excavation level
- total penetration
- length and position of cavity/cavities in each pile
- penetration in rock
- time of drilling at intervals not exceeding 5m
- details of all splicing or jointing operations, locations of sleeves, etc.
- details of grouting operation for tremie grouting and time tables
- weather
- top level of pile immediately after completion
- errors in position and inclination
- amount of grout and the pressure used
- size and position of boulder/boulders in each pile
- detailed drilling speed (m/min)
- description of drilled material

8.1.1.17
8.1.1.18 12.1 As-Built Drawings

After completion of the piling, the Contractor shall submit an as-built drawing. This drawing shall be prepared by Registered Licensed Surveyor. It should include the following:

a. Size and type of piles

b. Eccentricities in both directions

c. Depth of penetration of each pile or reduced level of tip of each pile and cut-off level of each pile.

APPENDIX B

Sample Procedure for Supervision for Micropile Construction

SUPERVISION OF MICROPILE CONSTRUCTION

INTRODUCTION
A check list for supervision of micropile construction.

DESK STUDY

Study the following documents and clarify with the Project Engineer :-
1) Method Statement on Pile Installation
2) Specifications for Materials and Testing (Reinforcement, Grout, Additives, Stabilising Fluid)
3) Construction Drawings

Review and comment on the following construction records :
1) Boring Record (Borelog with time taken for every m of drilling)
2) Grouting Record (Grout mix, Cube Strength, Grout Intake)

ON SITE CONSTRUCTION SUPERVISION

Check the following items in the submitted records :

Boring Record
1) Change of Soil/Rock Formation/Karstic features(Cavity, Overhang, Floater, pinnacle)/boulders/Loss of Water/Artesian Water with respect to Depth (preferably in RL).
2) Drilling Rate in Soil/Rock (Time taken at convenient intervals but not more than 3m).
3) All relevant levels (Working Platform, Top Casing level, Water level, Bedrock level, Socket level, Pile Base level, Connection level, etc).
4) Pile Reference Number.
5) Starting/Completion Date.
6) Drilled hole Diameter, Pile Dimension and Capacity.
7) Theoretical Volume and Actual Volume of Grout.

Grouting Record

1) Grout Mix.
2) Strength.
3) Additive.
4) Grouting Pressure (if any).
5) Starting/Completion Time.
6) Grout Intake with Time.
7) Grout Loss.

Records to be submitted not later than at noon of the next working day after pile has been installed.

Piles Record

1) Record on each activity such as drilling, installing API pipe, grouting of the individual pile.

Visual Inspection on Materials at site :

Conditions of Reinforcement and Pipe

1) Visual Defects (Rusty, Distorted, etc).
2) Thread at Connection.
3) Coupling Element.
4) Protection Coating, Painting or Greasing (inside/outside the API pipe).
5) Dimension of Reinforcement (OD, thickness).
6) Spacers/Centralisers

Cement Grout and Additives

1) Expired Date of Cement.
2) Expired Date of Additives.

Conditions of Casings (Permanent/Temporary)

1) Visual Defects (Rusty, Distorted, etc).
2) Dimension of Casing. (ID and OD)
3) Connection for Welding/Coupling.

Water quality for Grouting

1) Source of supply. (Portable water)

On Site Supervision on Installation Process :

1. Check the deviation of installed micropiles in accordance to specification.
2. Make sure both ends of API pipes completely fitted in the coupling element and good contact between two connected ends of the API pipes.
3. Check mixing and density of stabilising fluid before grouting and after desilting.
4. Make sure no debonding substance on the reinforcements or inside/outside the API pipes and casings. API pipes have been degreased on both sides.
5. Check dimension of spacing between the centralisers along the API pipe.
6. Confirm the base is clean by depth measurement in the boreholes before grouting.
7. Ensure the cleaning of socket by flushing process is effective.
8. Collect representative API pipe specimens and grout cube samples for strength testing before installation.
9. Check pile installation sequence.
10. Measure the length and mark on the tremix hose before inspecting in borehole.
11. Only allow opening for flow of grout to the annulus at the bottom of the API pipe (not along the pipe)
12. Overflow grout should be the same in mixing/density as in the mixer.
13. Collect soil/rock samples.

Report of Anomalies :

1. Water gushing out of hole.
2. Water loss during drilling.
3. Sudden loss of grout.
4. Collapse of hole.

Testing On Materials :

1. API pipe: minimum 3 specimens for each batch of supplied reinforcement or every 40 piles (which ever is larger) or as directed by the Engineer
2. Grout: minimum 9 cubes for each batch of mixing or grout for 7-day, 14-days and 28-day strength testing. For test piles, extra 2 cubes are to be taken for 1-day and 3-day strength testing for early strength for load testing.