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DESIGN AND INSTALLATION OF MICROPILES AND GROUND ANCHORS FOR OHL SUPPORT FOUNDATIONS

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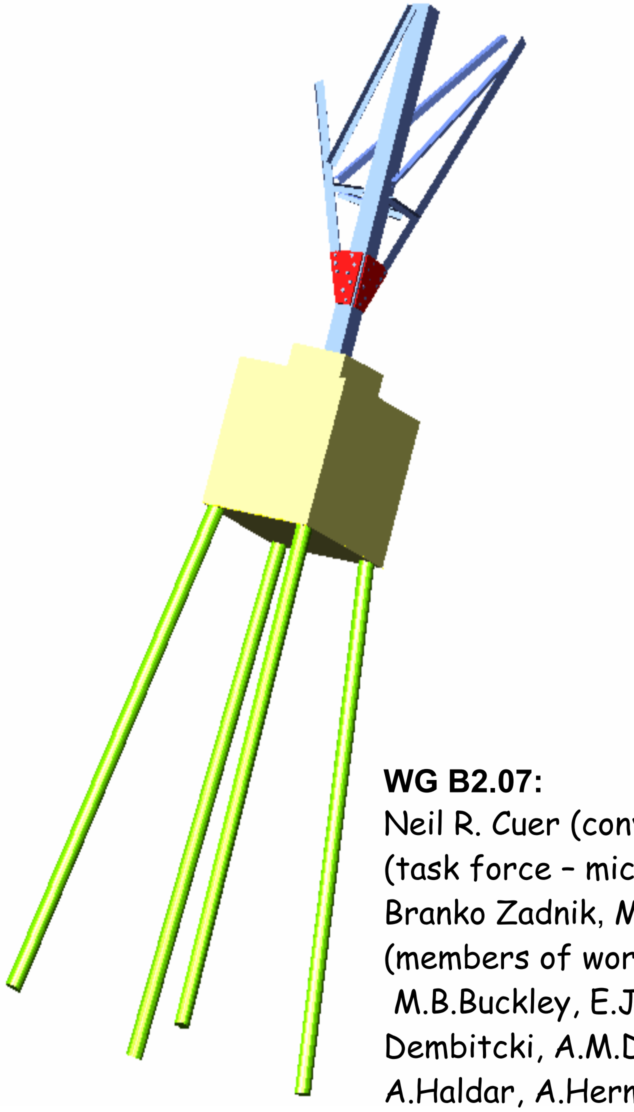

WG B2.07:
Neil R. Cuer (convenor),
(task force – micropiles and anchors)
Branko Zadnik, Michael Pietschke
(members of working group)
M.B.Buckley, E.J.O'Connor, E. Dembitcki, A.M.DiGioia Jr, J.Dufour, A.Haldar, A.Herman, M.Leva, K.Sahla, J-P. Siversten, M.J.Vanner.

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SYNOPSIS

This report was prepared by a task force drawn from WG07 'Foundations of Cigre Study Committee 22 and provides a guide to the design and installation of Micropiles and Ground Anchors for Overhead Line Support Foundations.

Micropiles are small diameter cast-in-place piles with a diameter less than 300 mm (or an equivalent with a noncircular cross section) and can be installed vertically or with an inclination up to 20 degrees to the horizontal surface, they are principally loaded by axial forces. If micropiles are used only to resist tension loading they can also function as ground or rock anchors. Nevertheless micropiles are non-prestressed foundation elements no matter whether they are used as piles or as ground anchors.

Drilled micropiles can be installed in a wide range of soils from non-cohesive, poorly graded granular soils, to cohesive plastic clays, also in rocks and even through existing concrete structures. Drilled micropiles are a viable solution to underpinning or structural support projects.

Micropiles can replace conventional piles under most circumstances, and are especially economical where there are difficult ground conditions or where there is limited or difficult access or work space. The drilling/driving equipment can be adapted to operate with low headroom, low weight and small dimensions.

The preliminary design of micropile is considered in Section 1 of this report, while Section 2 covers the determination of geotechnical site characteristics. Further guidance on the site investigation and corresponding soil classification is given in Section 6. Details of the ground characteristics, the concepts of failure of the micropile and the determination of the holding capacity of the micropile are given in Sections 3 and 4 respectively. The site requirements and details of the installation workmanship are given in Section 4. Examples of application of Micropiles and ground anchors to OHL support foundations are Section 5.

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SCB2 WG07 Task Force Members: B. Zadnik (S) and M. Pietscke (DE)

During the preparation of this report, WG07 comprised the following members:
N.R. Cuer (Convenor), E.O' Connor (Secretary), M.B. Buckley (IE), E. Dembicki (PL),
J. Dufour (FR), A.M. DiGioia Jr. (USA), A. Haldar (CA), A. Herman (BE), M. Leva (IT),
K. Sahla (FI), J-P. Sivertsen (NO), M.J. Vanner (UK), G. Paterson (AU).

1. Basic principles and applications of micropiles and anchor foundations

1.1 Introduction

In the last 20 years the application of small diameter (micro) piles to transmission line support foundations has become more frequent.

This report provides an overview about the design and installation of these types of foundations.

In accordance with [1] the following definitions are given:

Micropiles are small diameter cast-in-place piles with a diameter less than 300 mm (or an equivalent with a noncircular cross section).

Micropiles can be installed vertically or with an inclination up to 20 degrees to the horizontal surface, they are principally loaded by axial forces.

If micropiles are used only to resist tension loading they can also function as ground or rock anchors. Nevertheless micropiles are non-prestressed foundation elements no matter whether they are used as piles or as ground anchors.

Micropiles consist of a central steel tendon surrounded by concrete, cementious or resin based grout, which is usually placed under pressure.

The load transfer is bond/adhesion tendon – grout and friction/shear grout – soil (rock).

The excavation for the micropile can be produced by drilling, driving or vibration.

Depending on the installation method and type of bearing element the following types of micropiles are used in practice:

<u>Cast-in-situ concrete piles</u>

< installed by drilling, with a steel reinforcing cage or a single reinforcing bar embedded in concrete or cementious grout.

<u>Composite piles</u>

< installed by driving a prefabricated steel bearing element into a cementious grout or concrete filled hole or driving an element with an enlarged base directly into the ground with accompanied or subsequent grouting.
< installed by setting the steel bearing element into a hole which is subsequently grouted.
< installed by drilling down the bearing element (steel tube) itself with accompanied grout injection.

Drilled micropiles can be installed in a wide range of soils from non-cohesive, poorly graded granular soils, to cohesive plastic clays, also in rocks and even through existing concrete structures. Drilled micropiles are a viable solution to underpinning or structural support projects.

Micropiles can replace conventional piles under most circumstances, and are especially economical where there are difficult ground conditions or where there is limited or difficult access or work space. The drilling/driving equipment can be adapted to operate with low headroom, low weight and small dimensions.

All these advantages were seen also in transmission line foundation works, especially taking into account environmental aspects or the upgrading of existing foundations.

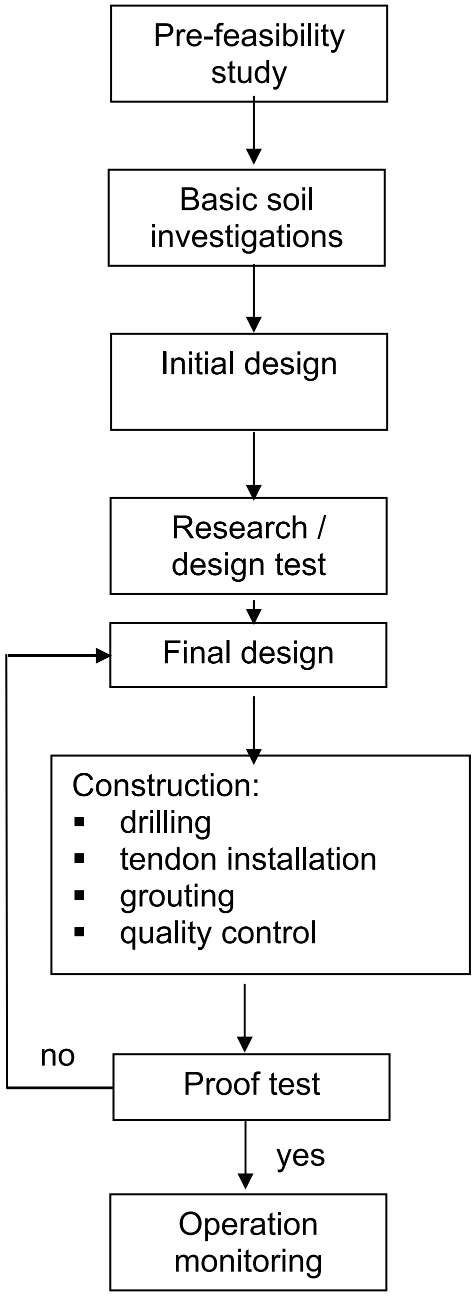
1.2 The preliminary design of micropiles

1.2.1 General

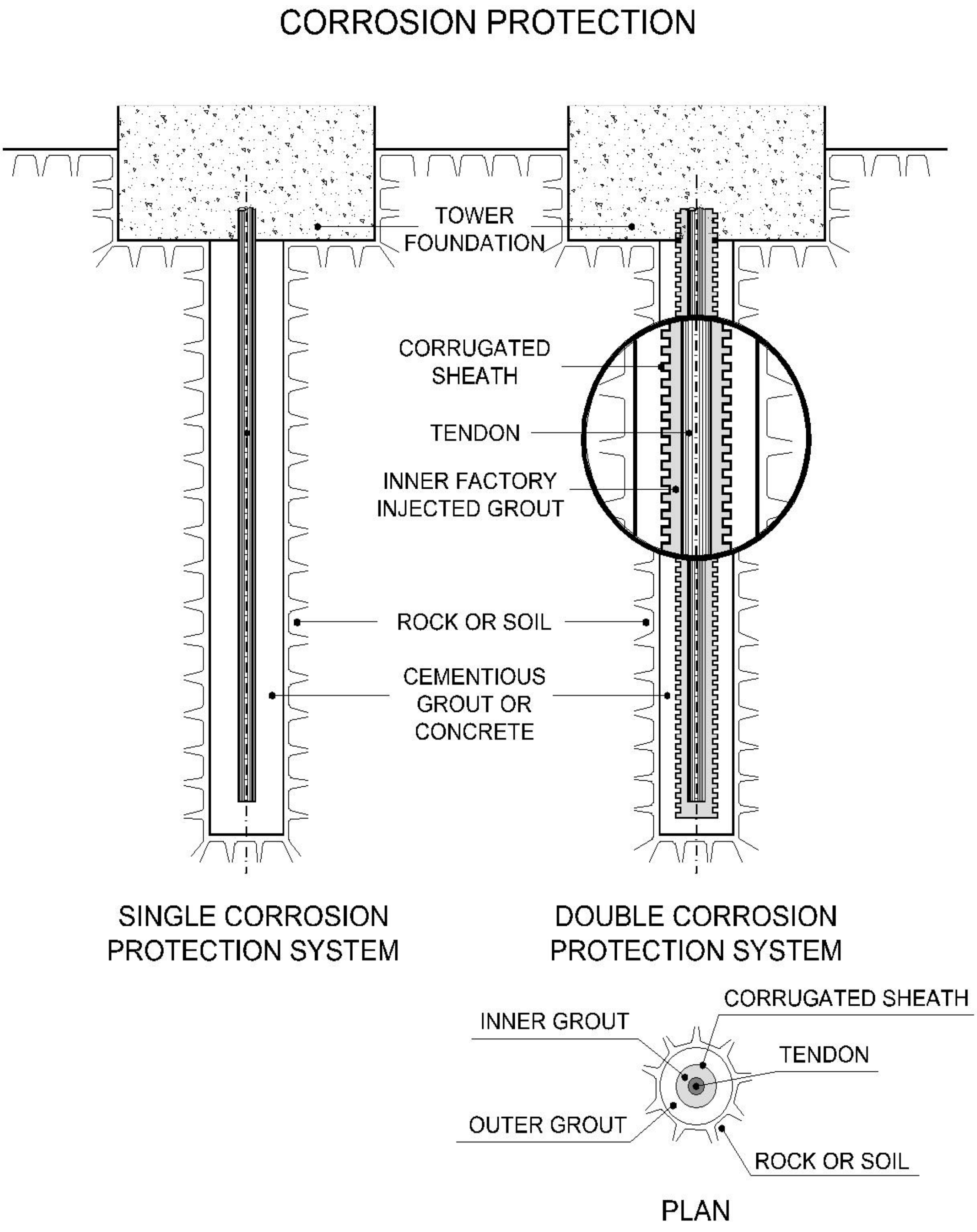
Preliminary design of micropiles may be confined to simple determination of the fixed length of the pile or ground anchor, under known or assumed soil conditions. This may be sufficient for determining the suitability of the proposed foundation, both technically and economically. However, the following objectives will need to be considered:

- Ground investigations;
- Selection of the configuration and inclination of the micropile as a part of the support foundation;
- Identify the tendon type and size;
- Estimate the length of the micropile on the basis of known soil characteristics;
- Check the overall micropile – foundation stability;
- Select the corrosion protection system;
- Specify a testing programme;
- Specify the inspection/investigation procedures during construction of the micropile;
- Monitoring of the micropile foundation through the whole life of the support.

1.2.2 Flow chart of design procedures for micropile foundation



1.2.3 Two basic types of micropile foundations according to the corrosion protection system



2 Geotechnical site characterization

2.1 Introduction

2.1.1 General

Geotechnical site characterization is the process of defining the subsurface soil/rock layers and their properties.

Because of soil complexity, comprehensive characterization at a particular site could require a detailed and costly testing program. Therefore, empirical correlations between the laboratory and/or in-situ tests and the desired soil properties are used extensively in estimating the design soil properties. In current practice, in-situ measurements and other available site information are interpreted by the design engineer, along with any laboratory test results, to obtain the design soil properties. This interpretation is largely subjective and requires sound engineering experience and judgment.

Site investigations are usually carried out to confirm the feasibility of a project as a whole, but adequacy in this respect does not necessarily ensure the feasibility of a proposed micro pile installation. Whereas, adequate data may be available to indicate the feasibility and advantages of micropile foundation, this data nonetheless may be insufficient to permit their economic design and construction. This shows the importance of a detailed knowledge of the ground.

In general, an investigation program will include the following stages:

a) Initial site reconnaissance and field survey;
b) main field and laboratory geotechnical investigations;
c) investigations during construction.

The main points of such an investigation programme are described by following items. For further detailed description see also section 6.

2.1.2 Site reconnaissance and field survey

This initial desk top stage is undertaken to determine the extent of the subsequent site investigation. Available ground and physical condition of the site are studied at this stage. The following four groups of data are of the main interest:

Site topography, which includes the collection and analysis of various maps and photographs. Site topography is useful in obtaining a preliminary plan and longitudinal profile.

Site geology. Initial studies of geologic and soil survey maps are undertaken to determine the general ground conditions. Experience with the local geology can be useful in achieving a higher level of geologic information to the site.

Ground water conditions are essential general information and can be obtained by preliminary tests at the site. Information on hydrology and meteorology can be obtained from other available sources. Observation of surface water and vegetation growth is useful in assessing potential drainage problems. Potential aggressiveness of ground water may relate to corrosion problems.

Site history. Includes the details and records of existing underground structures and the intentions for developing new sites in the area of the future micropile location. Site inspection at the initial conception of the project will ensure that the installation of the micropiles can be scheduled with regard to wayleave conditions and the interaction with existing structures.

2.1.3 Site investigation

The function, geometry and operational characteristics of micropiles relate to the ground conditions. Minor variations in ground conditions must be given greater attention because of the higher sensitivity of micro piles to variations in soil properties/parameters changes compared with conventional foundations.

The number and locations investigated by borings, probes or in situ tests and the depth to which they must be extended will be determined with regard to the type of micropile, site location, the number of support sites to be considered, and data available from previous investigations.

Additional test borings should be drilled where sloping ground exists, or where there is potential for land slide activity to occur.

Sampling. Available sampling techniques are well documented. Samples are taken by standard tube penetrometer, Shelby tube, or NX rock coring to obtain material for identification and testing. Samples should be taken from each stratum at maximum intervals of 1.5 m in thick strata. Intermediate disturbed samples suitable for simple classification tests should also be obtained.

Ground water. Determination of the ground water conditions is essential for the overall design and construction. All observations of the water conditions during boring or drilling should be recorded in the investigation process.

Long term ground water condition can be measured with the use of piezometers.

Additional requirements. The extent of field investigation relates to the importance of the overhead line and associated risks in its execution. In-situ pull out tests are required according to the national standards to verify the design proposals.

2.1.3.1 Laboratory investigation

Soil properties. Soil properties relevant to micropile design are:
a) Angle of internal friction;
b) cohesion;
c) particle size (in cohesionless and mixed soils);
d) in situ density;
e) permeability;
f) liquid and plastic limits;

g) unconfined compressive strength.

Rock properties.

a) Modulus of elasticity;
b) uniaxial compressive strength;
c) existing interfaces between various strata;
d) presence of water in the joints.

Chemical analysis.

Suitable tests are usually carried out on a routine basis to assess the overall corrosion hazard. Where an aggressive or corrosive environment exists, a comprehensive chemical analysis is mandatory to determine:

a) the aggressiveness of the ground water with respect to cement;
b) the aggressiveness of the soil with respect to the metal.

2.1.4 Investigation during construction

Irrespective of whether there is any variation in the ground conditions, QA requirements would demand that a daily record of the drilling is kept. This record should include variations in strata levels, ground types, and conditions that may require design changes and different installation procedures. The performance of the test piles should also be within the scope of site investigation, and should be analyzed with respect to the field and laboratory data.

2.2 Ground characteristics and concept of failure

In general, theories and design methods assume that mass of soil will fail along slip lines or shear planes. The relevant forces are introduced in a stability analysis. Two basic load transfer mechanisms cause the ground resistance to be mobilized as the micropile undergoes displacement under load. The first is side shear (adhesion or friction), followed by end-bearing where suitable configuration exists and when sufficient movement occurs. Accordingly, micropiles can fail in localized shear as long as the continuity of the surrounding ground is not disturbed. General failure occurs when the shear planes are fully mobilized or under significant deformations progressively reaching the ground surface.

In general, the analysis of the load resistance of micropiles must consider the following:

a) Mechanism of failure as load is transferred from one medium to another in the micropile – soil system;
b) Ground characteristics at failure;
c) Area roughness and configuration of potential failure interfaces;
d) Stress conditions (type, magnitude, direction) occurring along the failure interface when failure initiated.

Usual design practice dictates selection of micropile components and the analysis of potential failure modes with an appropriate factor of safety, consistent with the actual known strength or associated degree of risk.

2.2.1 Failure of ground – grout bond

2.2.1.1 Basic assumptions and considerations

Provided the micropile is of sufficient depth, failure is likely to occur at the grout – soil/rock interface prior to failure occurring in the soil mass. For a conventional straight sided, cylindrical micropile, a convenient assumption is that the shear resistance is mobilized at the interface of the borehole, and is uniformly distributed along the length. Under these conditions, the total shear resistance developed at the interface is a function of fixed dimensions of the micropile and applied load.

Experimental and theoretical work has shown that the shear resistance between the ground and grout is more complex than the foregoing idealized model. This complexity gives rise to essentially a nonuniform bond distribution, and in the final stage, in an inaccurate design. However, it is reassuring to know that where certain sections of the fixed zone are overloaded and shear failure is imminent, other sections begin to receive and resist the load so that equilibrium is reestablished.

Bond resistance (adhesion or friction) is also known to depend on the soil properties. An increase in the relative density of sand generally increases the angle of internal friction, which in turn increases the friction resistance at the interface. For cohesive soils, increase in stiffness or decrease in plasticity usually implies higher shear strength, with a corresponding improvement in the bond capacity.

Other factors that have considerable effect on the bond resistance relate to field operations. For example, the use of rotary percussive hammers to advance a casing in sand increases the normal stress, and this improves friction. In cohesive soil drilling without casing or with casing using flushing water tends to have softening effects and thus reduce shear. Post-grouting generally improves the load capacity in proportion to the magnitude of the post-grouting pressure.

2.2.1.2 A simple theoretical expression for bond

For conventional straight sided cylindrical micropiles, the average shear stress τ along the grout – ground interface can be related to the applied load P by the simple expression:

$$P = \pi D L \tau \qquad \text{eq. 1}$$

where D is the outer diameter of the grout cylinder and L is the fixed length. This approach is valid under the following assumptions:

1. The transfer of the load from the grout to the ground occurs uniformly over the fixed length.
2. The borehole and the fixed length have the same diameter.

3. Failure occurs by sliding at the ground – grout interface for a smooth borehole, or by shearing along a zone adjacent to the interface for a rough borehole (failure along the weaker shear zone, which may be at the interface or away from it).
4. Debonding does not occur at the ground – grout contact area.
5. There are no discontinuities or weak planes that can alter the process of failure.

The total shear resistance at the interface is the summation of two components: adhesion and friction. Thus, the shear stress τ is expressed as:

$$\tau = c_a + \sigma_n \tan\delta \qquad \text{eq.2}$$

where: c_a = adhesion between ground and grout

σ_n = normal effective stress on the anchor zone

δ = friction angle between the soil and grout

Where shear strength tests are carried out on representative rock or soil samples, all factors from eq.2 are grouped into a single parameter. In this case, the allowable bond stress is estimated from shear strength test values with an appropriate factor of safety, normally not less than 2.

2.3 Empirical data on ground properties

Current practice in determining soil properties from in-situ and/or laboratory measurements are based on empirical models and the quality of the estimate is dependent on the data base supporting the model. Frequently, the model is based on limited data, and therefore it may be biased towards a specific soil type; accordingly, it may not be appropriate for soils at other locations.

Data given in this section are generally from published literature, e.g. technical journals, conference proceedings, and research reports. The interpretation of data depends on the knowledge and experience of the design engineer. Published design bond values are summarized in Table 1, for a wide range of igneous, metamorphic, and sedimentary rocks. The factor of safety correlates ultimate and working bond stress based on uniform bond distribution.

TABLE 1: Published Rock-Grout Bond Values for Design

No	Rock Type	Working Bond strength (N/mm^2)	Ultimate Bond strength (N/mm^2)	Factor of Safety	Source
Igneous					
1	Medium hard basalt		5.73	3 – 4	India - Rao (1964)
2	Weathered granite		1.50 - 2.50		Japan - Suzuki et al (1972)
3	Basalt	1.21 - 1.38	3.86	2.8 - 3.2	Britain - Wycliffe-Jones (1974)
4	Granite	1.38 - 1.55	4.83	3.1 - 3.5	Britain - Wycliffe-Jones (1974)
5	Serpentine	0.45 - 0.59	1.55	2.6 - 3.5	Britain - Wycliffe-Jones (1974)
6	Granite and basalt		1.72 - 3.10	1.5 - 2.5	USA - PCI (1974)
Metamorphic					
7	Manhattan schist	0.70	2.80	4.0	USA - White (1973)
8	Soft shales		0.21 – 0.83		
9	Slate and hard shale		0.83 - 1.38	1.5 - 2.5	USA - PCI (1974)
Calcareous Sediments					
10	Limestone	1.00	2.83	2.8	Switzerland - Losinger (1966)
11	Chalk – Grades I-III (N = SPT in blows/0,3 m)	0.005N	0.22 - 1.07 0.01N	2.0 (temporary) 3.0 - 4.0 (permanent)	Britain - Littlejohn (1970)
12	Tertiary limestone	0.83 - 0,97	2.76	2.9 - 3.3	Britain - Wychliffe – Jones (1974)
13	Chalk limestone	0.86 - 1.00	2.76	2.8 - 3.2	Britain - Wychliffe – Jones (1974)
14	Soft limestone		1.03 - 1.38	1.5 - 2.5	USA - PCI (1974)
15	Dolomitic limestone		1.38 - 2.07	1.5 - 2.5	USA - PCI (1974)
16	Dolomite	< 0.50		2.8	Slovenia – Zadnik (1998)
Arenaceous Sediments					
17	Hard coarse - grained sandstone	2.45		1.75	Canada - Coates (1970)
18	Weathered sandstone		0.69 - 0.85	3.0	New Zealand - Irwin (1971)
19	Well-cemented mudstones		0.69	2.0 - 2.5	New Zealand - Irwin (1971)
20	Bunter sandstone	0.40		3.0	Britain - Littlejohn (1973)
21 22	Bunter sandstone (UCS > 2.0 N/mm^2)	0.60		3.0	Britain - Littlejohn (1973)
23	Hard fine sandstone	0.69 - 0.83	2.24	2.7 - 3.3	Britain - Wycliffe-Jones (1974)
24	Sandstone		0.83 - 1.03	1.5 - 2.5	USA - PCI (1974)
Argillaceous Sediments					
25	Keuper marl		0.17 - 0.25 (0.45 c_u)	3.0	Britain - Littlejohn (1970)
26	Weathered marl		0.17 - 0.25		
26	Andesite tuff		0.30	2.5 – 2.9	Slovenia – Zadnik (1998)
27	Weak shale		0.35		Canada - Golder Brawner (1973)
28	Soft sandstone and shale	0.10 - 0.14	0.37	2.7 - 3.7	Britain - Whycliffe-Jones (1974)
29	Soft shale		0.21 - 0.83	1.5 - 2.5	USA - PCI (19784)
General					
30	Competent rock (where UCS > 20 N/ mm^2)	Uniaxial compressive strength - 30 ($\leq$1.4 N/mm^2)	Uniaxial compressive strength - 10 ($\leq$4.2 N/mm^2)	3	Britain - Littlejohn (1970)
31	Weak rock	0.35 - 0.70			Australia - Koch (1972)
32	Medium rock	0.70 - 1.05			
33	Strong rock	1.05 - 1.40			
34	Wide variety of igneous and metamorphic rocks	1.05		2	Australia - Standard CA35 (1973)
35	Concrete		1.38 - 2.76	1.5 - 2.5	USA - PCI (1974)

Typical values for the ultimate skin friction for small diameter injection piles are given in Table 2.

TABLE 2: Typical limit skin friction values for small diameter injection piles

Type of soil	Compression piles	Tension piles
	(N/mm^2)	(N/mm^2)
Medium gravel and coarse gravel	0.20	0.10
Sand and gravelly sand	0.15	0.08
Cohesive soil	0.10	0.05

Note: Above table based on values quoted in DIN 4128

The permissible skin friction values are obtained by dividing the limit skin friction given in Table 2 by safety factors. Typical values of the safety factors are 2.0 for compression piles, 2.0 for tension piles inclined between 0° and 45° to the vertical.

Data for the pre-calculation of injection anchors were also given by Ostermayer [17]. Figure 1 gives an overview on ultimate capacity of injection anchors in noncohesive soils depending on the load transfer (injection-) length:

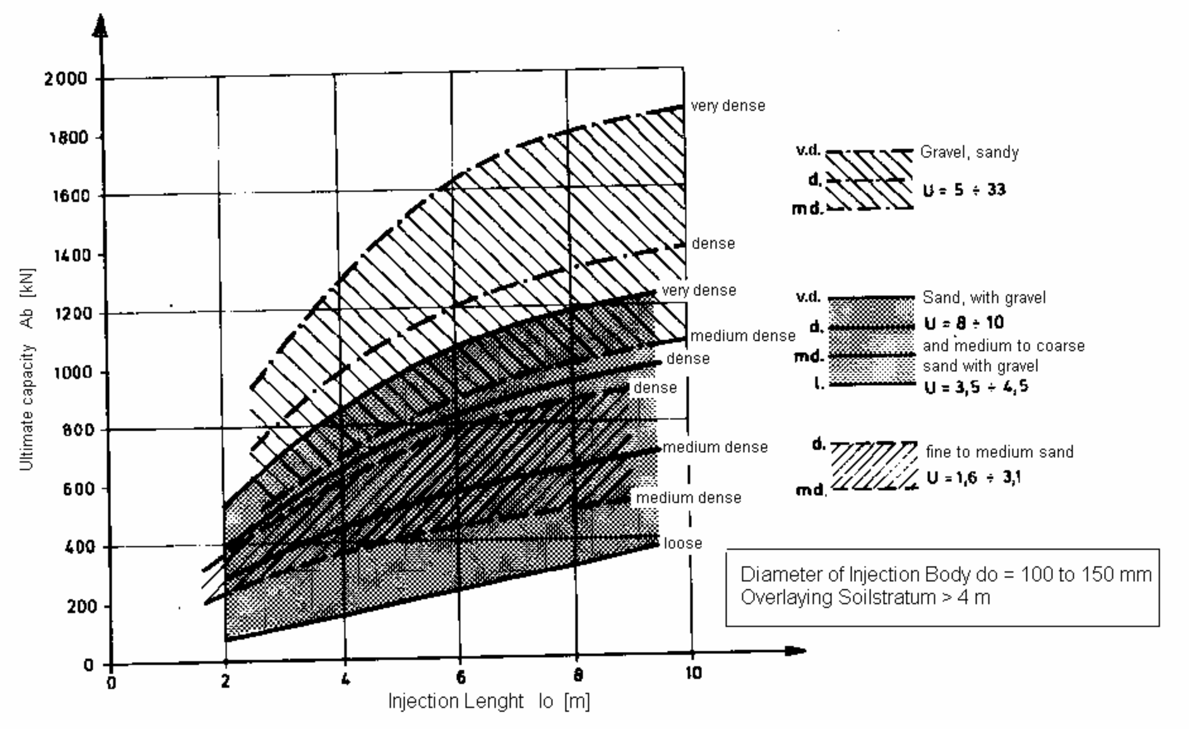


FIGURE 1: Ultimate capacity of injection anchors in non-cohesive soil

From Figure 1, it can be seen, that:

1. Ultimate capacity increases very much with increasing density.
2. For the same density the ultimate capacity increases with the mean grain size.

3. Ultimate capacity increases under-proportionally with the increase of injection length. Especially in dense soils the increase of the ultimate capacity for injection length over 6 m to 7m is small due to the progressive failure along the injected body. Greater injection length in this kind of soil is therefore not economical in common cases.
4. For common diameters of injection bodies between 100 mm and 150 mm the influence of the diameter on the ultimate capacity is negligible (decrease of bond strength with increase of injection diameter).
5. The influence of the overlaying stratum thickness is negligible if a minimum value of about 4 m is exceeded.

Frequently the density of the soil is determined by penetration tests, correspondingly Figure 2 [17]. Therefore Figure 2 gives the ultimate capacity of injected anchors depending on values of the standard penetration test (SPT) or heavy dynamic probing (DPH) (see DIN 4095).

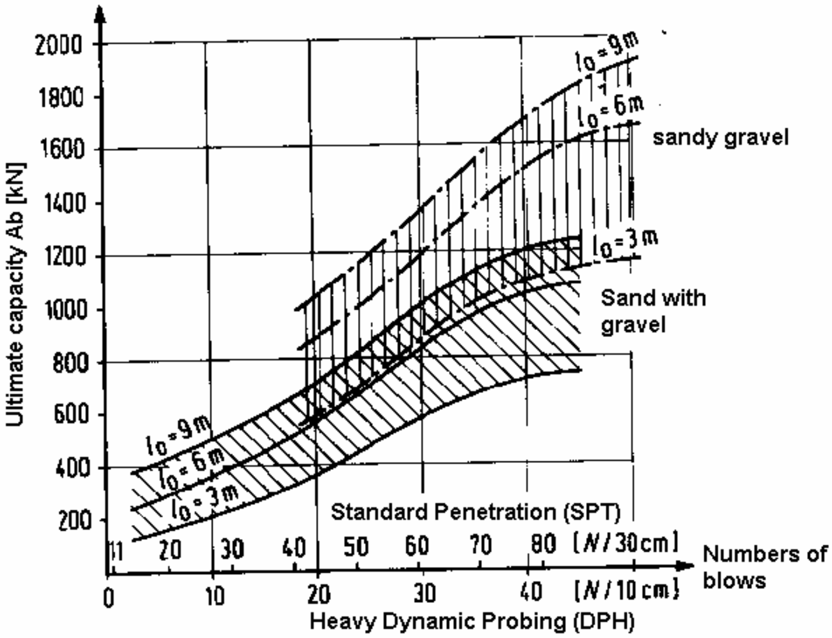


FIGURE 2: Ultimate capacity of injection anchors in non-cohesive soil depending on penetration tests

Figure 3 [17] shows mean values of ultimate bond strength related to the real surface of injection body in cohesive soils depending on the injection length for anchors with (fig. 3a) and without (fig. 3b) re-injection.

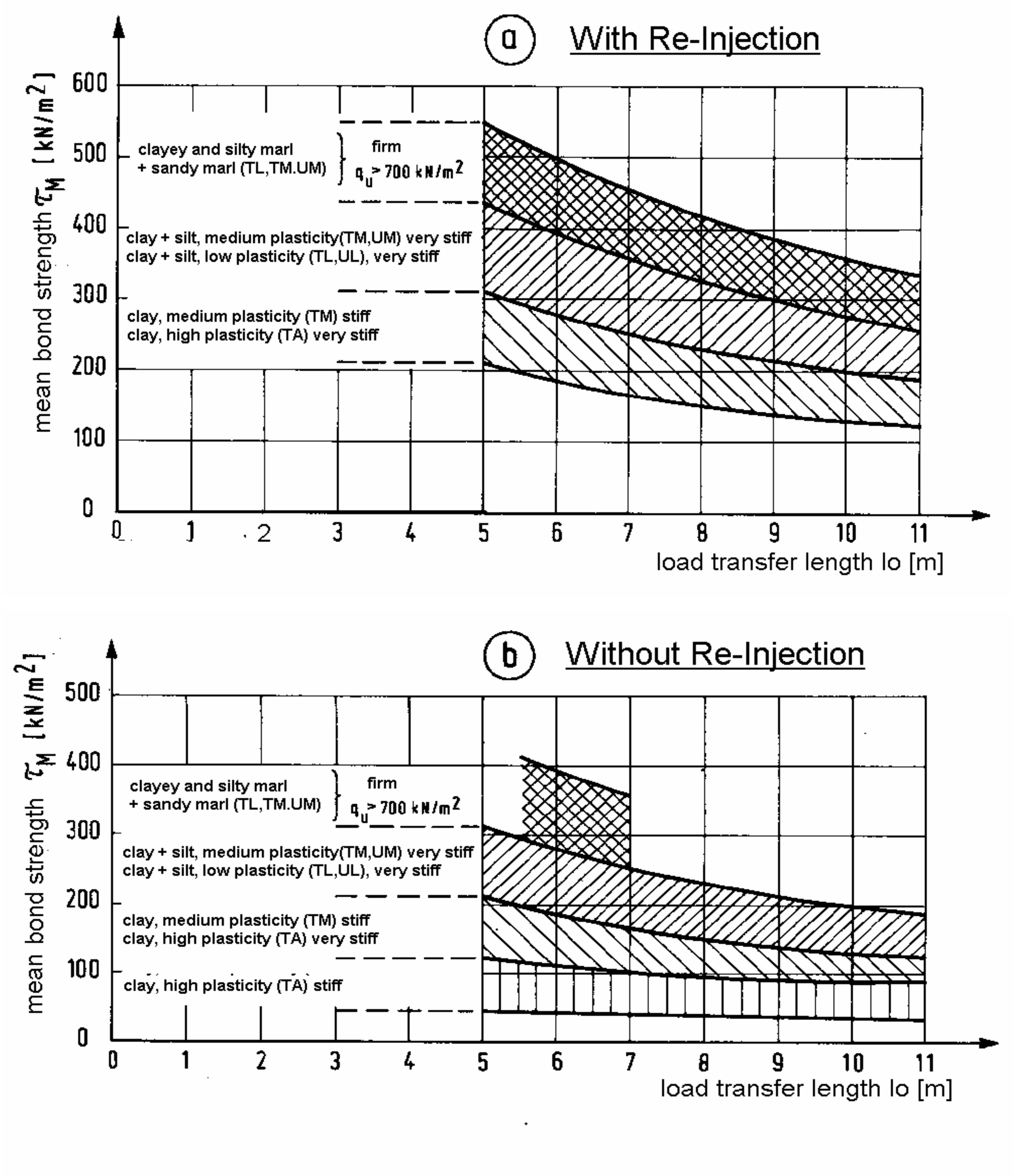


FIGURE 3: Ultimate values of mean bond strength for anchors in cohesive soils (with and without re-injection)

From Figure 3, it can be seen, that:

1. The bond strength increases with decreasing plasticity and increasing consistency, e.g. stiff clays with high plasticity have lower bond strength values than firm cemented marl.

2. The mean value of bond strength is independent from the injection length l_0 for values of about 100 kN/m², which means that the ultimate strength increases proportional to l_0. For values higher than 100 kN/m² the bond strength values decrease with increasing injection length (progressive failure), that means that the ultimate strength increases under-proportionally to l_0.
3. For common diameters of injection bodies between 100 mm and 150 mm the influence of the diameter on the ultimate bond strength is negligible, which means that the ultimate strength increases with increasing diameter.
4. Re-injection can lead to a relevant increase of the bond strength.

The increase of the ultimate anchor strength by re-injection is achieved in some cases by increasing the bond strength (interlocking in the soil) on one hand and by enlarging the diameter of the injection body on the other hand.

For the transition region from very firm (hard) soils to weathered rock Littlejohn (1970) proposed the following empirical relationship for ultimate bond strength:

$\tau = 0{,}45 * c_u$ to $0{,}60 * c_u$ - for marl and very firm clays

$\tau = 10 * N_{30}$ [kN/m²] - for lime marl

with: c_u = undrained shear strength

N_{30} = number of blows (SPT) per 30 cm

3 Determining micropile/anchor holding capacity

3.1 General

In OHL tower foundation analysis, there is practically no difference between an anchor and a micropile. From both a theoretical and a practical viewpoint, understanding the load transfer mechanism is essential for the design of a foundation where the micropile/anchor is the main structural element. The structure must have an adequate safety factor and has to satisfy implicit economic criteria. This topic must be analyzed both in a theoretical context and confirmed by empirical data. However, it is necessary to distinguish and identify the limitations of our present knowledge.

Micropile/anchor transfer theories are often based on idealized assumptions, and where the conditions are different from the foreseen ones, the results can be misleading and questionable. Therefore, it is recommended to check the design according to different design rules available based on full-scale tests and general field experiences. The effects of construction techniques and the quality of construction work on micropile/anchor pullout capacity are quite obvious. Thus, the micropile/anchor design is supplemented by a mandatory testing program to confirm the load carrying capacity of this type of foundation.

3.2 Micropile/anchor carrying capacity and serviceability

The carrying capacity of an anchor or a micropile is defined as the force which will cause visible deformations, i.e. large displacements of the micropile/anchor body with regard to the surrounding soil. In other words, this is the force in the direction of anchor axis, expressed in a magnitude large enough to cause a settlement at the compression load or a pullout at the tension load of an unacceptable measure. A carrying capacity may also be defined as the force causing failure of the steel tendon, bond failure (slippage at the tendon - grout interface), shear failure along the micropile-ground contact surface and/or plasticity of the soil under the micropile base at the compression. This can be described by the following expression:

$$N = N_b + N_0 \qquad \text{eq. 3}$$

where:

N = carrying capacity
N_b = compression carrying capacity of the micropile base
N_0 = friction carrying capacity of the micropile-ground contact surface.

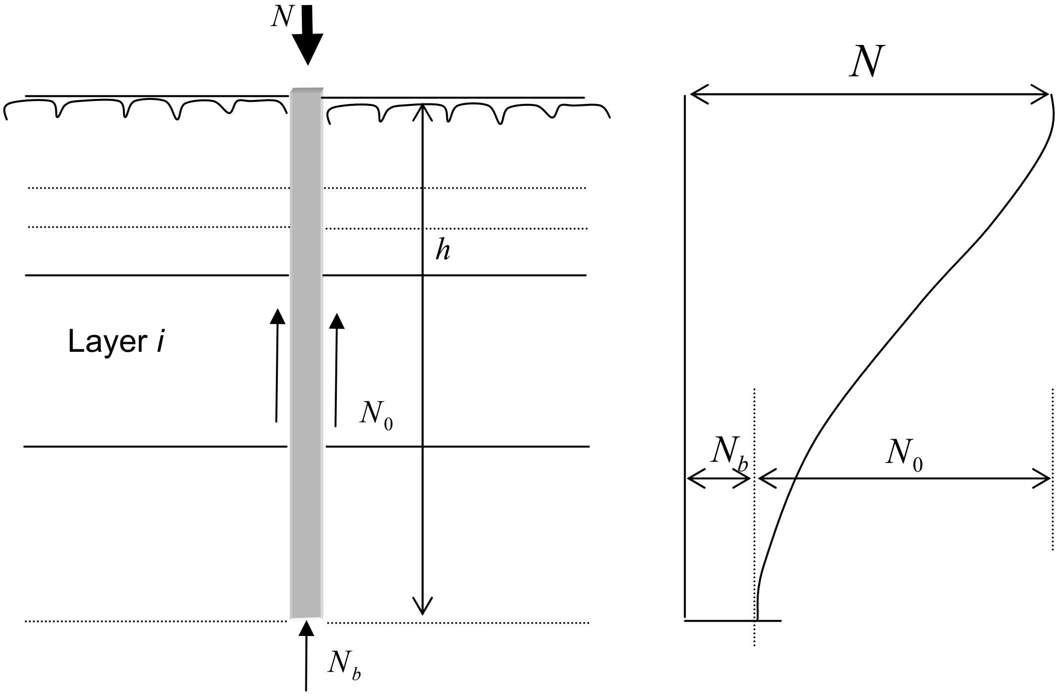


FIGURE 4: Load carrying mechanism of a micropile under compression

h = length of the micropile

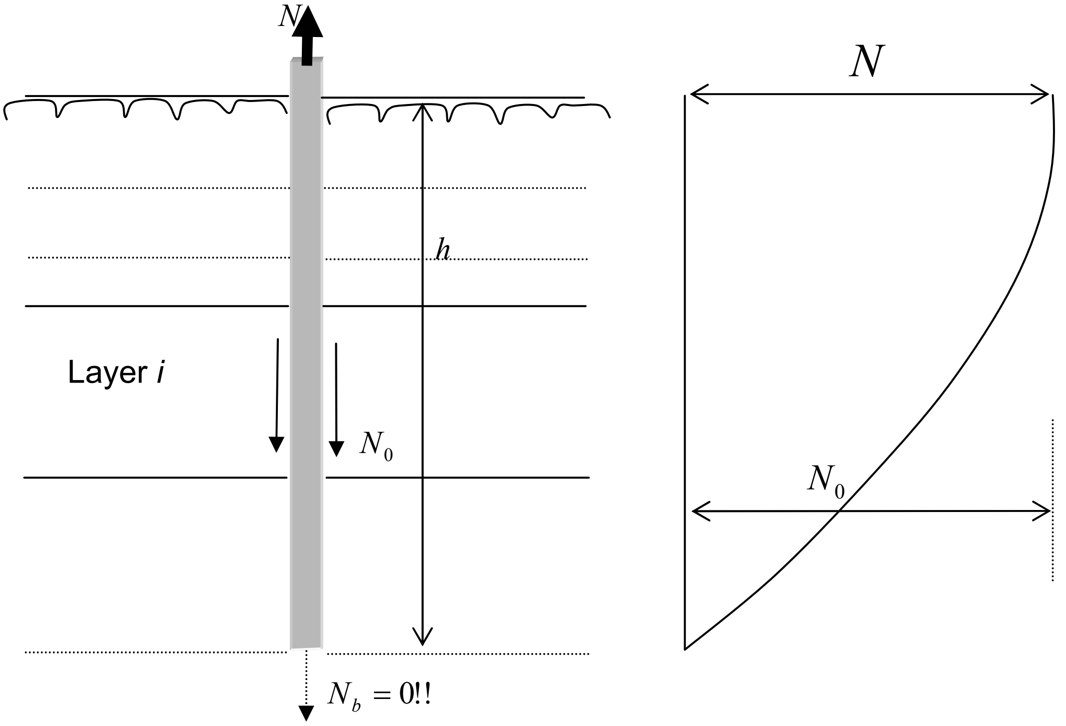


FIGURE 5: Load carrying mechanism of a micropile under tension

In case of compression loads the micropile base will carry a portion of the load (Figure 4); whereas, in case of tension loads the loads only act on the grout - ground surface (Figure 5). On the basis of known geometrical characteristics and ultimate values of the soil mechanical parameters, it is possible to assess the complete carrying capacity of the micropile.

The parameters of equation 3 are expressed as follows:

$$N_b = F_b . p_u \qquad \text{eq. 4}$$

where:

F_b = area of the micropile base,
p_u = ultimate stress under the micropile base.

and:

$$N_0 = F_0 . \tau_0 \qquad \text{eq. 5}$$

where:

F_0 = area of the micropile surface
τ_0 = friction on the micropile and ground contact surface

The ultimate holding capacity depends on the ultimate values of the base bearing capacity N_b and on the ultimate shear stress values on the grout-ground surface of the micropile N_0. These ultimate values are only exceeded in case of large displacements. Correspondingly, it is a rule that the friction on the grout-ground surface exceeds the ultimate value before the compression stress under the micropile base reaches its maximum value. This means that the micropile acts in the first phase as a friction pile and later, with increasing axial load in the second phase, the base becomes increasingly engaged as the carrying capacity element.

For foundations in weak soil layers it is necessary to check the compression-loaded piles on buckling. Micropiles are very slender structures and in soils with an undrained shear strength lower than 10 kN/m^2, under consideration of deformations, safety against buckling should be verified according to the second order theory. Neither the cement grout nor any lateral support provided by the soil should be taken into account. However, by verifying resistance against buckling, the effective bending rigidity may be calculated from the cross section of the load bearing member and the cement grout inside a plastic corrugated sheathing (if such is the construction of the micropile).

Vertical pressure in the foundation soil (p_u) under the micropile base should be calculated according to the basic rules of soil mechanics. The ultimate value of compression stresses at great depth has to be considered, which means that at such depth no extrusion of surrounding soil is possible and that plastification of the soil material is in progress.

Friction t_0 is a result of adhesive stresses on the grout – ground surface of the micropile. Consequentially, it depends on many parameters including the installation method. For driven piles there are different friction values manifested in comparison with drilled piles. With increasing depth friction rises to its limit value. The determination of a friction coefficient (τ_0) on the surface of the micropile is generally more complicated than the determination of the allowable compression stress (p_0) under the base of the micropile. In case of micropiles, the bearing capacity of the base is usually neglected due to its relatively small area. Normally, it is supposed that the friction force will comprise the whole bearing capacity.

In case of tension loads acting on the micropile, the friction force on the grout - ground surface gets an opposite sign as in case of compression loads.

In everyday practice, the holding capacity can be calculated by defining the micropile allowable load (S_{allow}). It is defined as:

$$S_{allow} = \frac{N}{F_S} \qquad \text{eq. 6}$$

where:

F_S = safety coefficient in the range of 1.0 to 5.0.

3.3 Bearing capacity of a micropile under horizontal loads

The foundation of an OHL tower is exposed to vertical tension (Figure 5) and/or compression (Figure 4) forces and simultaneously, to the horizontal forces (Figure 6). The direction of a horizontal force depends, among other factors also on the wind direction and has to be treated in + or – direction. Basically we assume that the horizontal resultant force acting on the tower will distribute proportionally to the complete tower foundation. However, in case of separate foundations, including micropile foundations, we have to check the horizontal resistance of a single leg foundation. A suitable anchorage depth (t_1) into the well-compacted soil layers has to be provided. Different analytical methods may be used to determine the deflection of the pile exposed to the horizontal load (H) at the top. In case of a micropile foundation for steel lattice towers which are very sensitive to differential vertical and/or horizontal leg displacements, the deflection (Δ) becomes a leading parameter for assessment of the foundation bearing capacity. Based on practical experiences, the displacements in the range of up to 5 mm in the horizontal direction are acceptable. Field tests are recommended in order to confirm the analytical values.

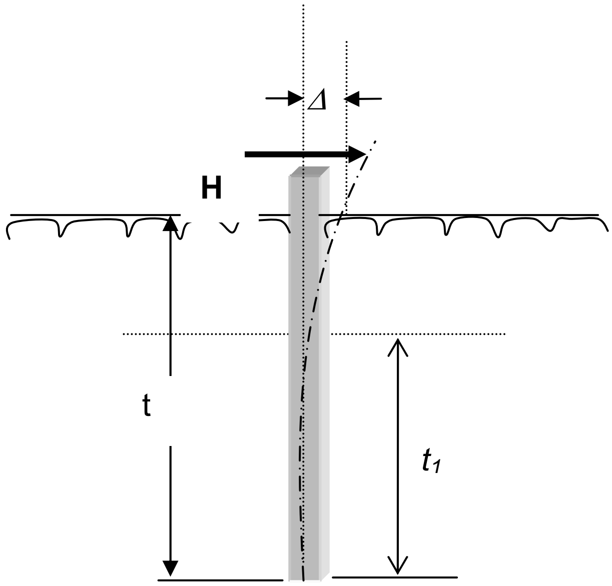


FIGURE 6: Bearing capacity of a micropile under horizontal loads

Micropiles are very flexible structures with a diameter of d ≤ 300 (200) mm and length in the range of 7 and more meters, depending on the project requirements. The allowable deflection (Δ) at the top of the pile may be attained only in very sound (rock) soil. In normal situations, there should be a reinforced concrete cap connected to the top of the micropile which will take over all horizontal loads and transmit them into the surrounding soil. In very weak soils, a concrete frame would be a successful solution of this problem.

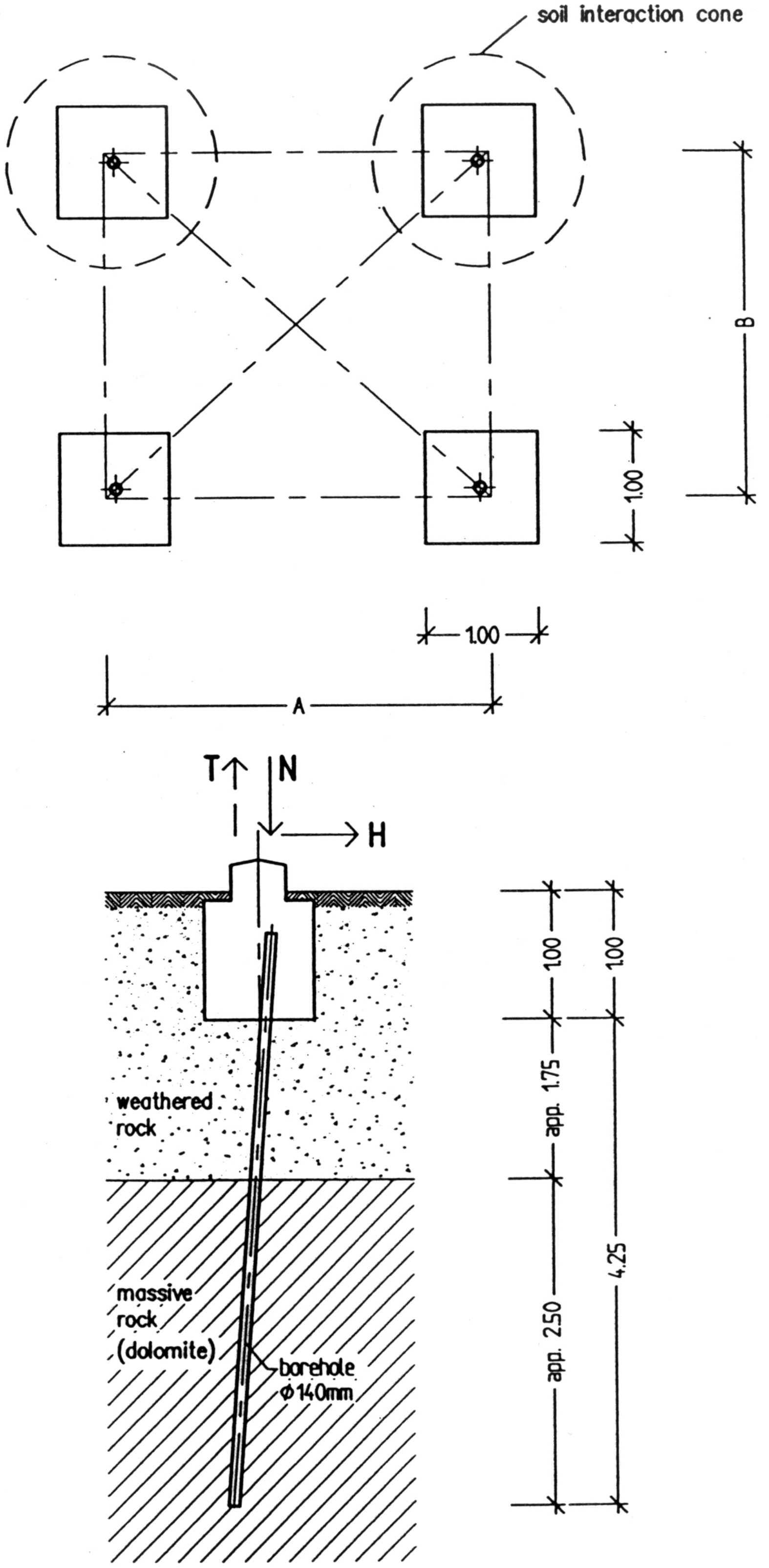


FIGURE 7: Micropile rock foundation, reinforced concrete block head

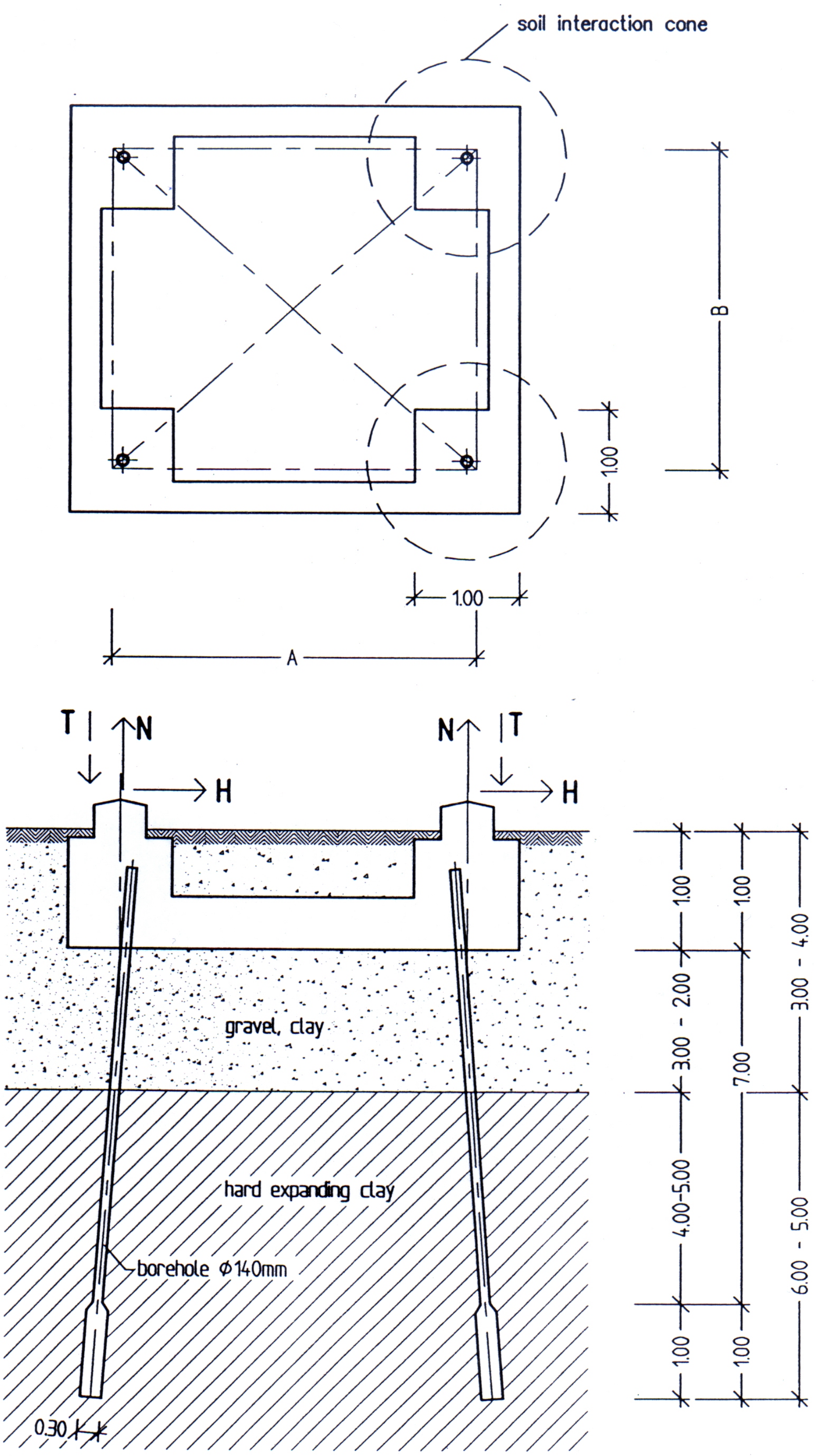


FIGURE 8: Micropile soil foundation, reinforced concrete frame head

Where more micropiles/anchors are used on each leg of the tower, the horizontal force acting on the head of the foundation may be directed to a single anchor using simple geometry rules.

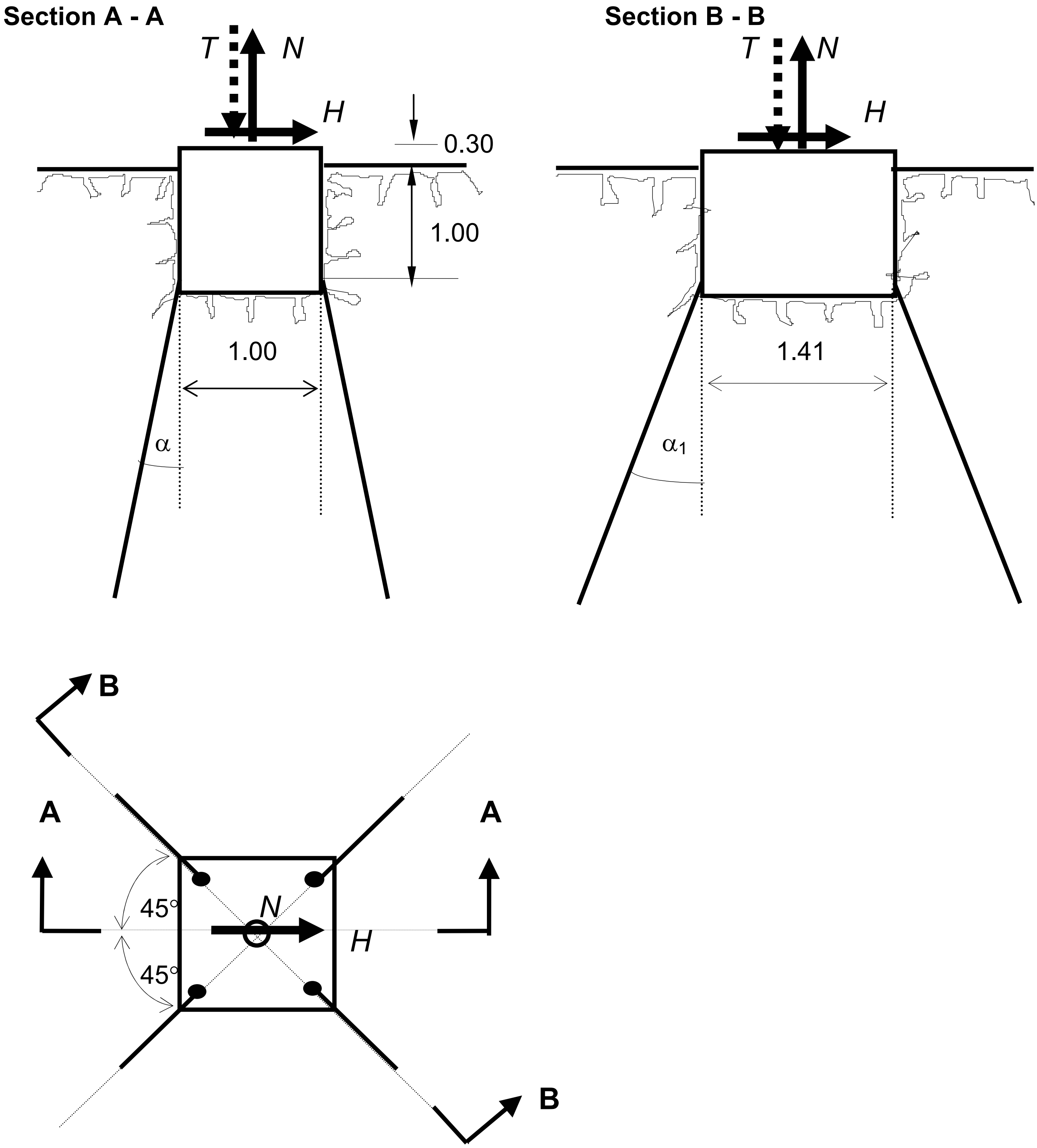


FIGURE 9: Four anchors per leg of a tower

3.4 Methods of bearing capacity determination

The bearing capacity of a micropile can be determined in accordance with different national codes and practice:

1. On the basis of experiences gained from similar practical cases;
2. On the basis of the data collected by statical or dynamical ground penetration;
3. By the help of dynamic formulas;
4. On the basis of load test results;
5. By static equations using soil bearing capacity data

3.4.1 Experiences gained in foundation construction

The allowable force of the micropile can be determined by using data from previously installed micropiles of similar types in similar ground conditions. Such data can be introduced in the early stage of design procedures.

3.4.2 Penetration tests

Data collected by standard penetration tests or dynamic penetration tests show the basic nature of the soil resistance of the geotechnical profile. The soil resistance is one of the factors needed to assess the allowable force in a micropile.

3.4.3 Dynamic formulas for calculation of a micropile holding capacity

The micropiles holding capacity calculation can be performed by an empirical approach to dynamic formulas derivation. These formulas are namely very useful in case of driven piles foundation design. Energy of the pile driver is used to penetrate the pile into the soil. The relation is shown in eq. 11:

$$N = \frac{W.H}{s.K_u} \qquad \text{eq. 11}$$

where:

W = weight of the pile-driver
H = height of falling of the pile-driver
s = penetration of the pile at each impact
K_u = coefficient greater than 1.0 (2.0 to 15.0), given in codes

The allowable carrying force of the pile is given by:

$$S_{allow} = \frac{N}{F_S} \qquad \text{eq. 12}$$

where F_S is safety factor in the range of 2.0 to 3.0 and depends on the heterogeneity of the ground and importance of the structure.

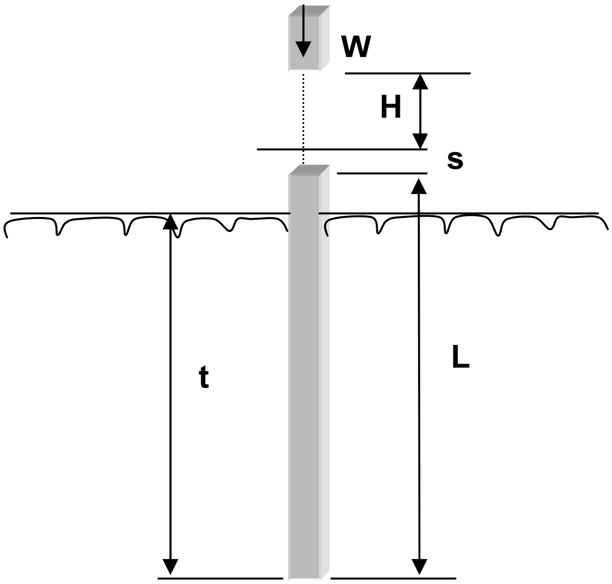


FIGURE 10: Determination of holding capacity N of pile driven into the ground

3.4.4 Static approach to determining load carrying capacity for a micropile

In the every day design load carrying capacity (N) and/or allowable force (S_{allow}) of the micropile are determined by static formulas. Soil shear parameters cohesion (c) and the internal angle of friction (φ) are very important in these equations.

In general, we have two groups of equations. The first one is used to determine load carrying capacity of the micropile (N) by the measured values of shearing parameters. The second group is used to determine allowable force (S_{allow}) on the micropile by reduced values of these parameters.

The allowable load carrying capacity of the micropile is defined as follows:

$$S_{allow} = S_b + S_0 \qquad \text{eq. 15}$$

with

$$S_b = F_b * p_{allow} \qquad \text{eq. 16}$$

$$S_0 = \sum (F_{t_i} . t_{allow_i}) \qquad \text{eq. 17}$$

where:

F_b = area of the micropile base

p_{allow} = allowable compression load carrying capacity under base

F_{t_i} = area of micropile body surface in soil layer i.

t_{allow} = average value of friction on soil - grout contact in soil layer i.

The allowable compression load carrying capacity under the micropile base is given by the following equation:

$$p_{allow} = c_m.N_c + K_0.q.N_q + \gamma.r.N_\gamma \qquad \text{eq. 18}$$

$$c_m = \frac{c}{F_c} \qquad \text{eq. 19}$$

where:

r = radius of micropile base

c_m = mobilized cohesion of soil with safety coefficient F_c in the range of 2.0 to 3.0, usually 2.5.

K_0 = coefficient of lateral pressure at rest ($K_0 = (1 - \sin\varphi)$).

q = vertical soil pressure (p_v).

γ = soil unit density

N_c, N_q, N_γ = coefficients of soil carrying capacity which depends on mobilized angel of internal friction of soil layer.

Mobilized angel of internal friction is given by:

$$tg\varphi_m = \frac{tg\varphi}{F_\varphi} \qquad \text{eq. 20}$$

F_φ = safety coefficient in the range of 1.2 to 1.8, usually 1.5.

TABLE 3: Orientation values of soil carrying capacity coefficients [8]

φ (°)	φ_m (°)	N_c	N_q	N_γ
15	10	25	2,7	-
22	15	38	5,5	-
29	20	75	12.0	4
35	25	150	25.0	8

The intermediate values can be determined by linear interpolation.

Friction (t_i) on the core surface in the depth (h_i) represents the sum of mobilized cohesion and shear resistance along the micropile axis:

$$t_i = c_{m_i} + p_{0_i}.tg\varphi_{m_i} \quad \text{(kN/m}^2\text{)} \qquad \text{eq. 21}$$

$$p_{0i} = \gamma_i.h_i.K_{0_i}$$

where: i = index of the soil layer which is being analyzed
h_i = depth of layer

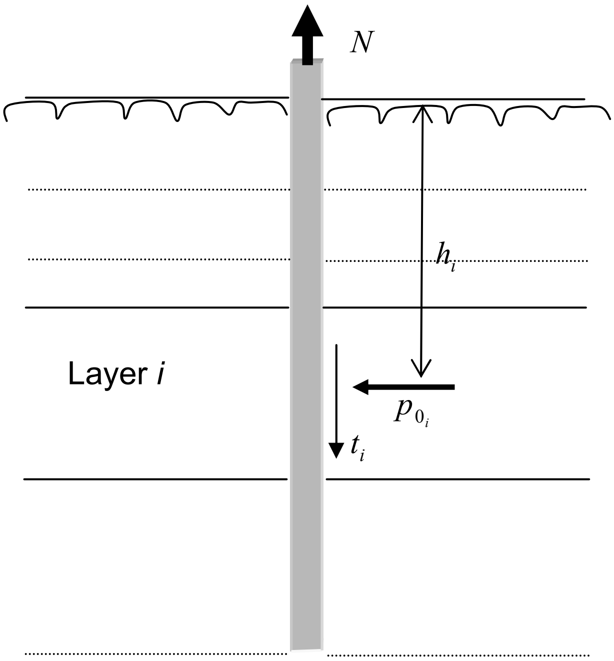


FIGURE 11: Forces on the micropile in the layer i

3.4.5 Experimental tests of the micropiles

The experimental load test is the most reliable method of determining a micropile holding capacity (N). In this method, all parameters which have influence on the holding capacity of the micropile are included. The ground strata, depth of the pile and influence of the construction method are some of the critical parameters that are thus incorporated into the procedure for evaluating micropile holding capacity. The only deficiency of this method is that the results obtained are exact only for the tested pile. In case of a micropile group acting in very heterogeneous soil conditions, the testing results relating to one micropile may be risky for the other piles.

The micropile holding capacity test is executed as a design test used to provide data in the foundation design procedures, or, after the construction start, as a test confirming the design values of a safety coefficient F_S.

$$S_{allow} = \frac{N}{F_s} \qquad \text{eq. 13}$$

$$F_S = \frac{N}{S_{allow}} \qquad \text{eq. 24}$$

The basic principle of a load test lies in the measurement of the displacements during the definite increments of loading, the load increments being applied at specific time intervals. The maximum load is achieved after a defined number of steps. Each loading phase is followed by an unloading phase. The loading and unloading time intervals are specified by national codes or previous experience.

The relationship between the force and the displacement is shown in the diagram below. Understanding this diagram as well as explaining of the results obtained represents a problem. In general, the diagram has a form of a non-linear line and it is very difficult to determine the force which should represent the maximum load carrying capacity (N) of a micropile. For this purpose, different methods are used in practice. One method is to pre-define the allowable displacement and read the value of load carrying capacity *N* on the vertical axis fixed by the experimental curve. The second method defines *N* by the intersection of two tangent lines to the initial and final stages of curve (Figure 12).

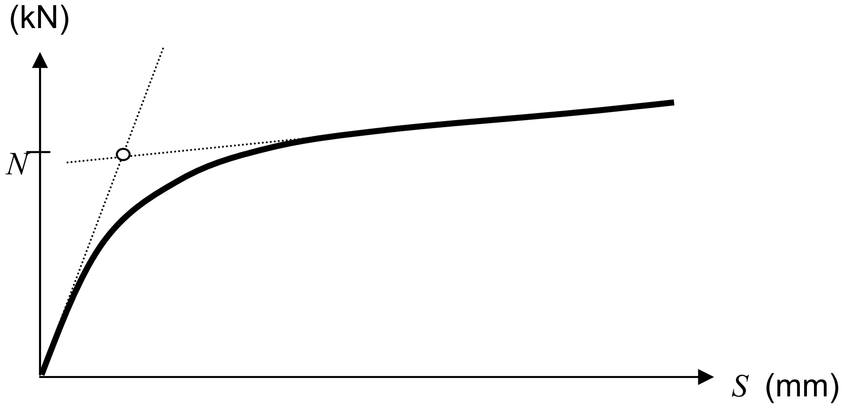


FIGURE 12: Graphical method of determining load carrying capacity (N) of a micropile

Although the testing of piles is usually covered by national / international standard, these generally have a bias towards normal civil engineering practice, i.e. compressive load testing. However, in the case of transmission support foundations, the critical loading is usually uplift (tension) and therefore, it is recommended that consideration is given to the use of IEC 61773 (Testing of Foundations for Structures, IEC, 1996). Both design and proof tests would normally be required.

Design tests encompass: establishing the geotechnical parameters, e.g. the grout / ground bond strength, etc., verification of the design parameters, e.g. the overall length of the pile, verification of the installation procedure or proving the design of the pile against the specified contractual requirements, e.g. load – resistance capacity and/or displacement criteria. Design tests are always undertaken on micropiles that are specially installed and do not form part of the final production (working) foundations.

Proof tests are usually required to check the quality of the installation, for the production (working) foundations, or where there is a wide variation in the assumed ground conditions and hence in the micropile load – resistance capacity. Proof tests are always undertaken on production foundations to a defined percentage of their ultimate capacity, with a specific limit on their displacement. The actual number of micropiles to be tested will depend on the quality assurance requirements for the contract; typically 10% of the total number of micropiles installed is required. However, as previously mentioned, if there is a wide variation in the assumed ground conditions, the percentage tested could increase to 100% of the micropiles installed.

If specific tests are required on the water tightness of the drilled hole and hence the volume of grout injected in the installation of the micropile, reference should be made to relevant standards for the design of ground anchors, e.g. EN 1537 (Execution of special geotechnical works: Ground Anchors: 2000).

3.4.6 Stability of ground mass

The analysis of micropile/anchor load capacity and modes of failure should be supplemented by an analysis of the stability of the foundation ground mass above the micropile. The uplift capacity determination is based on a wedge mechanism; whereby, the pullout loads are in equilibrium with the weight of a specified ground body. Where this body is situated below the water table, the submerged weight should be used. The idealized shape of the soil mass is combination of a cone and cylinder, as shown in Figure 13, or, in case of a group of anchors, of a truncated cone and cylinder configuration as shown in Figure 14. In this case, the maximum uplift resistance does not include the shear resistance along the failure surface.

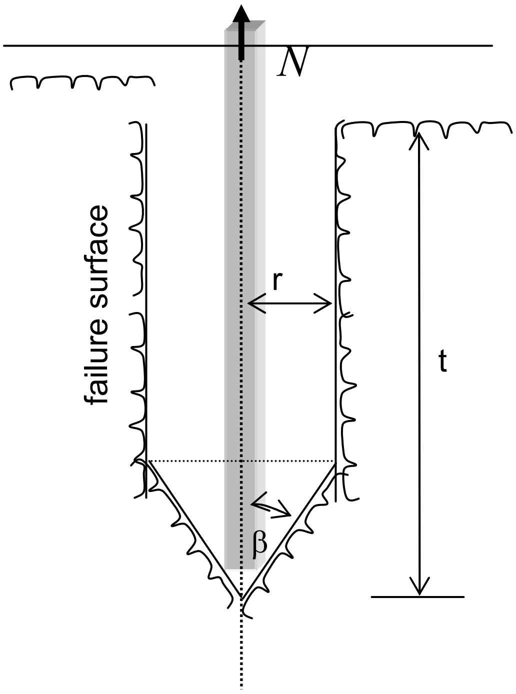


FIGURE 13: Idealized shape of a mobilized soil mass in pullout failure

In Figures 13 and 14 the parameters are as follows:

r = radius of the influenced cylinder in the range up to 5d, where d is the diameter of the micropile.
β = vertex angle of failure surface in range of 2/3 φ to φ
α = angle of inclination of the micropile

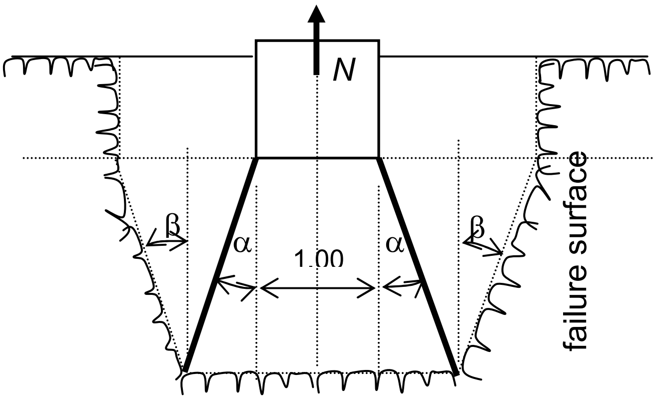


FIGURE 14: Truncated cone and cylinder in case of an anchor group

Experiences show that a generalized shear failure in a soil mass is normally associated with relatively shallow foundations and usually occurs when the micropile/anchor depth is less than 3 – 4 m. With deeper foundations the failure surface does not normally propagate to the ground surface.

4 Site requirements and construction workmanship

4.1 General

The construction of a micropile is a sequence of several separate interconnected operations: site investigations, design procedures, hole drilling, tendon manufacture, anchor installation, grouting and testing. Non-uniform ground conditions can go undetected in routine site investigation. To ensure and confirm the workmanship of a micropile installation, it is normal practice to introduce quality controls as a mandatory procedure during construction, including the use of routine/proof tests, specifying a defined acceptance criterion for the long term behavior and if appropriate, commence a monitoring programme for the service life of the micropile foundation. Construction of a micropile should always be carried out by skilled and experienced contractors specializing in this type of installation.

4.2 Hole drilling

Selecting the most suitable drilling method depends on several factors, including type of ground, site accessibility and topography, hole geometry and size, scale of drilling operations, type and capacity of the micropile, flushing medium, local labor costs and construction restrictions at the site.

In the majority of cases, the holes are drilled with a near the vertical inclination. The drilling method should be chosen to satisfy the following basic conditions:

- There should be minimum disturbance to the surrounding ground; correspondingly, hole flushing should be avoided in weak, finely grained soil.
- Hole stability must be permanently maintained.
- Loosening of the borehole walls in cohesionless soil should be avoided, whereas in cohesive soil and sensitive rock appreciable changes in water content and smoothing of the borehole surface can be detrimental to the transfer of load.
- Drilling should be completed in a manner that allows direct detection of the ground, and registers any major changes in ground characteristics compared to the design values.

The most important factor influencing the choice of drilling method is the type of ground - rock or soil. For each type of ground condition, various drilling methods are available. A hole can be drilled with the use of rotary, percussive, or rotary – percussive equipment. Occasionally, vibratory driving techniques are suitable. Core drilling is a high cost method with the risk of reducing the bond on account of the smooth hole surface. Any drilling machine and drilling procedure can be used that produces a hole that has the specified dimensions and tolerance.

Percussive drills. These accomplish penetration by the action of an impulsive blow. Repeated applications of high-intensity, short-duration force shatters hard material provided the blow is sufficiently heavy.

Rotary drills. Rotary drills impart their action through a combined axial thrust (static action) and rotational torque (dynamic action).

Rotary – percussive drills. These are combinations of the previous two types. Their action is primarily derived from an axial thrust of lower magnitude than that of rotary drill, with a torque lower than that of a rotary drill but much higher than that of a percussive drill and an impact of a lower magnitude than that of a percussive tool.

For the preliminary selection of a suitable drilling system, the following guidelines may be useful:

1. Ground strata to be drilled, micropile type, and capacity of the micropile will initially determine the length and diameter of the hole. With these parameters a range of suitable drilling methods and equipment can be established.

2. The percussive tools are preferable for most rock strata. Rotary methods are suggested for deeper holes or poor rock conditions.

3. In soil liable to collapse, drilling a hole will regularly require the use of a casing, drilled or driven into the ground to a specified depth.

4. In urban areas there has been a tendency to abandon the use of percussive tools in favor of rotary drills due to restrictions on acceptable noise levels and occasional vibratory effects.

4.2.1 Hole deviation and tolerance

Deviations in hole alignment should be monitored. Misalignment usually originates from two sources: (a) initial incorrect setting of the drill and (b) deviations of the hole from the correct initial line and angle during drilling. The condition in (a) is avoided and checked by the use of special mats. Deviations during drilling usually do not begin from a single cause; it may be caused by the use of rods that are too small, from excessive thrust, presence of fissures and rock discontinuities. Various authors and codes reveal considerable differences of opinion. An angular tolerances range up to 2° is generally taken as acceptable.

4.3 Flushing

All particles and byproduct materials from the bit should be removed quickly and completely. The most common flushing media are water and air or a bentonite slurry. Air is most efficient and is best used in dry ground. Water flushing generally improves the ground conditions, and is best used in sticky - clay soil; its sweeping action cleans the hole sides producing an increased bond at the grout – ground interface. Water is also a common flushing medium below the natural water table. Bentonite slurry flushing is not very common, but is used successfully for open hole drilling through silts and sand overlying rock; its suspending power keeps individual earth particles in its volume and facilitates their removal, while its sealing action keeps the hole from collapsing.

After the hole is drilled and flushed out, it should be sounded to detect the presence of any foreign materials. If the probe is satisfactory, the top of the hole is plugged in order to protect it and ensure it remains free of falling debris. The flushing medium may be introduced through the drill rods and the drill bit, and return to the surface between the rods and the walls of the hole. Alternatively, the flushing medium may follow the opposite way.

Local variations in the ground conditions, sometimes occurring within metres, can have considerable effect on anchor performance. It is recommended to keep records regarding the groundwater and flush medium. Additional qualitative data on ground conditions can be obtained by recording drilling rates and the extent of bit blocking, and by observing changes in the amount and composition of flush return.

4.4 Tendon preparation and installation

Micropiles are fabricated in a workshop or in the field by trained personnel and under competent supervision. During manufacture, handling, and installation, micropiles and their components should remain clean and free of any mechanical or structural defect, and also be continuously protected against corrosion.

National codes and standards on anchors provide guidelines and recommendations on the subject of storage and handling. In general, steel for micropile tendons should be stored indoors in clean and dry conditions. If tendons remain outdoors, they should be stacked off the ground and completely covered by a waterproof cover. Tendons should not be dragged across abrasive surfaces or through deleterious materials.

Fabrication and assembly. After cleaning, bar tendons should be checked to ensure that the bars are properly screwed into couplers and that the full thread engagement is obtained in all nuts and tapped plates.

The ‘centralizer’ (Figure 15) keeps the single bar tendon centrally located in the borehole and thus ensures a uniform grout cover in the fixed bearing zone. With flexible reinforcement, properly spaced ‘centralizers’ help minimize the sagging effect of the steel between support points.

Spacers, usually made of steel or plastic, are used in both the free and fixed sections of a multi-component tendon. Like centralizers, spacers help to maintain anchor components parallel and in their correct alignment (Figure 15). In the fixed anchor zone spacers serve three primary purposes:

< To centralize the tendon system in the borehole for an adequate and uniform grout cover, this both enhances the corrosion protection and provides a good grout bond at the borehole interface.

< To provide a positive grip for the tendon and the grout without restricting the flow of the grout in the borehole; thereby, ensuring complete grout penetration of the space around the tendon unit. This is a basic condition for the efficient transmission of bond stress.

< To help prevent contamination of the tendon parts such as clay smear. Spacers in this zone can also be used in conjunction with intermediate fastenings to form nodes, intended to provide a positive mechanical interlock between the tendon and surrounding grout.

The characteristics of spacers and centralizers can be combined into one unit. This trend reflects the relatively large number of design and construction details available for the fixed anchor length available in industry. These details, however, are not standardized, and they may depend both on the method of grout placement and whether the strands are arranged in a parallel configuration or connected at suitable intervals.

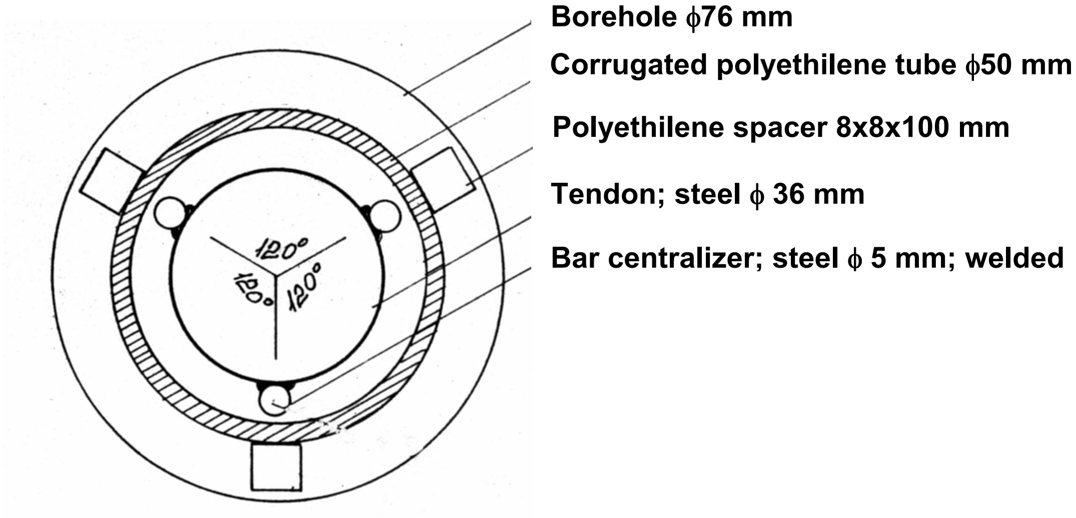


FIGURE 15: Typical bar centralizer – spacer details

Installation of micropile (homing). Homing should always be undertaken as soon as possible, since it is advantageous to complete drilling, micropile installation, and grouting on the same day. A delay from drilling to grouting with the hole open can be cause for ground deterioration, particularly in overconsolidated fissured clays and rock soils.

Immediately prior to its installation the micropile should be inspected and checked for possible damage to its components and protective system. The homing will largely depend on micropile length and mass. In practice, any method can be used provided the tendon is lowered at a steady controlled rate. Mechanical handling equipment is recommended as manual handling tends to be difficult and hazardous. For cased holes, it is recommended to use a tube or a circular entry pipe at the top of the hole to guide the tendon as it passes the sharp edge at the top of the casing, and avoid possible damage.

For micropiles, the normal practice is to drill, grout, install tendon – rebar or cage, and then remove any temporary casing

4.5 Grouting

4.5.1 General

To minimize construction effects, it is essential that the drilling and tendon installation is coordinated with the subsequent grouting operation. As already mentioned, certain ground types are prone to time dependent changes in their properties. This can be avoided by minimizing the time between drilling, tendon homing and grouting. If a delay is unavoidable, the hole should be plugged to prevent entry of foreign materials, and where the ground is prone to swelling tendon homing and grouting should follow drilling as quickly as possible.

The choice and design of a suitable grout system depends on the ground conditions in which it is to be placed, the setting time, strength, and intended functions of the grout. Low cost materials chosen for this application include a wide range of conventional cements. The general requirements are defined in appropriate specifications or in national codes and standards. Pre-designed and ready-mixed grout materials delivered bagged to the site are becoming increasingly popular since they help avoid delays and produce a consistent mix.

Grouts in general will perform any or all of the following functions:

(a) Holding the tendon to the ground by forming a load-transfer zone;
(b) Filling the void space within and around the tendon to augment protection against corrosion, which can be done simultaneously with (a) or as a second stage (secondary grout);
(c) Filling voids or fissures in the ground prior to tendon installation where pre-grouting is necessary.

The choice of grout should consider the aggressivity of ground toward the grout and aggressivity of the cement toward the tendon steel. Under normal conditions most cement grouts are durable. However, in the long and short term severe and quick deterioration can occur in adverse environmental conditions, such as chemical attack in the presence of dissolved sulphates or acids contained in groundwater. These effects are magnified, if there are deficiencies in grout quality, for example, low density and high permeability. Minimum cement content is essential to ensure reasonable durability under the expected conditions of exposure. Grout defense against chemical attack is further improved by use of rapid hardening, sulphate-resisting cements, and especially low-heat varieties or the use of alternative cementious materials, e.g. pulverized fly ash. Admixtures are occasionally recommended to ensure fluidity and to control shrinkage and setting time. Inert fillers such as ground quartz, limestone dust, fine sand and sawdust have been added to mixes used primarily to waterproof or consolidate the boreholes prior to re-drilling.

Chemical admixtures may offer certain advantages, but their compatibility with the cement type must be checked prior to use. Different types of admixtures should not be included in the same grout.

Mixing influences the quality of the grout, particularly its strength. For good mixing, the following guidelines should be followed:

- The cement and the admixtures should be measured accurately by weight.
- Water and admixtures should be added to the mixer before the cement.
- Mixing time for each batch should be long enough to produce a mix of uniform composition.
- Mixing by hand should not be attempted.
- All mixing equipment and pumps should be clean and well maintained.

Sufficient grout strength must be attained for bond at both the grout – tendon and grout – ground interfaces. A usual measure is the unconfined compressive strength at 7 days and at 28 days. Variables affecting grout strength are the W/C (water/cement) ratio, the pore ratio, the type of cement, and the presence of admixtures.

4.5.2 Grouting methods

Grouting can be accomplished by two distinct modes: two – stage and single – stage injection.

The two – stage grouting involves first the injection of a primary grout to create the bond zone in the anchor length, and after tendon homing a secondary grout is introduced, mainly for corrosion protection of the tendon. The tendon should be homed no later than 30 min after first injection. In the single – stage grouting process the borehole is filled in a single continuous operation.

Grouting always begins at the lower end of the section to be grouted. For proper filling, air and water should be allowed to escape. Before commencing grout placement, all pipes and their joints should be cleaned and checked for air tightness. Each stage of injection should be performed in one continuous operation, and at no stage should the end of the grout pipe be lifted above surface of the grout. If grouting is interrupted or delayed beyond the setting period, the tendon should be withdrawn, the grout removed by flushing or re-drilling, and the grout stage repeated. Where casing is used, further coordination is necessary between grout injection and withdrawal of the casing.

In general, grouting pressures are recommended by the specialist contractors, and may be followed with the stipulation that, where necessary, test trials will be undertaken before a construction procedure is accepted. Figure 16 illustrates the relationship between grouting pressure and the ultimate anchor resistance.

Grouting pressures, however, considered appropriate for a given project are largely a matter of conjecture. Excessive pressure must be avoided to prevent distress in the ground.

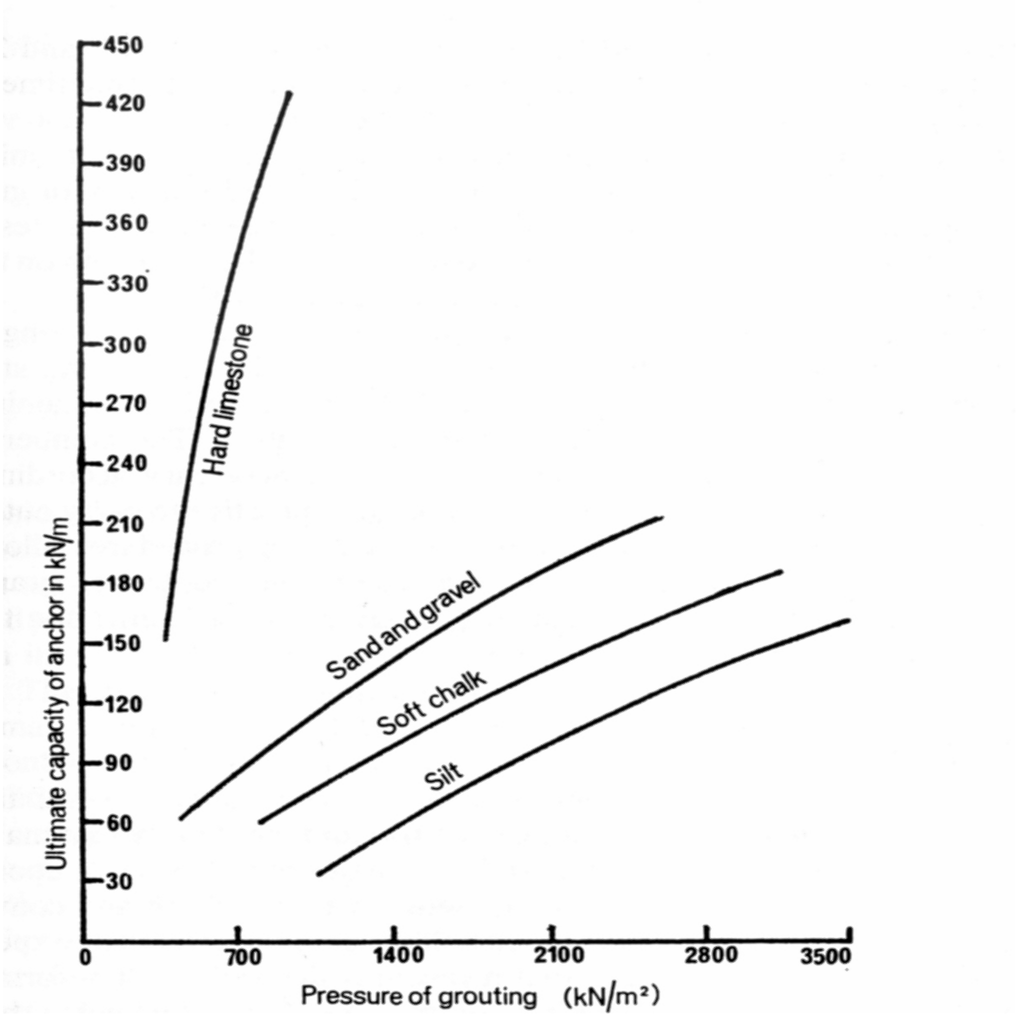


FIGURE 16: Proposed relationship between ultimate anchor resistance and grouting pressure (Soletanche, 1970)

4.5.3 Quality control

In general, when a new mix is introduced, its adequacy for the intended purpose is established through various tests carried out to determine and record its properties. The following information should be obtained:

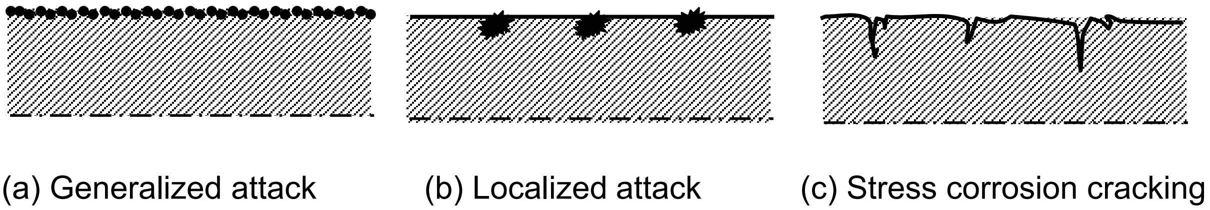
(a) W/C ratio and type of cement;
(b) admixture type and concentration;
(c) viscosity;
(d) crushing strength (at 3, 7, 14 and 28 days);
(e) data on expansion, shrinkage and final setting time.

If chemical contamination of the grout is detected, by the measurement of the pH value. The number and frequency of these tests is not standardized and may vary according to the site condition and job requirements: however, it is good practice to carry out the tests on a daily basis.

4.6 Corrosion protection

4.6.1 Types of corrosion

With respect to the causes of corrosion and the resulting effects, the types of corrosion can be grouped into three main categories: (a) generalized attack; (b) localized attack (shallow or deep pitting); (c) cracking (due to either hydrogen embrittlement or stress corrosion). These three categories are illustrated in Figure 17.



(a) Generalized attack (b) Localized attack (c) Stress corrosion cracking

FIGURE 17: Main types of corrosion

Generalized attack is approximately uniform and covers the surface of metal as shown in Figure 17 – (a). It is possible for the corrosion product to form a continuous film, which thereafter may act as protective layer and inhibit further attack. The accompanying reduction of cross section is basically uniform, whereas the center of the metal remains intact and sound.

Localized attack may be termed electrochemical corrosion, and is manifested as deep or shallow pitting (Figure 17 - (b)). Separate corrosion cells are distributed over the metal surface and are associated with the presence of a protective oxide film on metal. The formation of holes causes local stress concentration and eventually premature failure. Pitting can have severe consequences, yet the overall metal loss is small.

Stress corrosion cracking (SCC) is a form of corrosion where physical causes predominate. SCC is produced by the combined action of static tensile stress on the steel and localized corrosion. The precise mechanism is not completely understood. However, it is clear that localized action of corrosion produces a narrow pit, which allows the tension forces to concentrate at the tip of the pit, resulting in the formation of fresh metal surfaces where further dissolution can occur. With this combined action, crack propagation occurs causing cracking either along grain boundaries or along slip planes within the crystalline lattice of the material.

4.6.2 Requirements of corrosion protection

The condition of a tendon unit during the manufacturing and storing should be carefully checked for defects. A film of rust on the tendon at the time of its delivery is not necessarily harmful and may improve bond, but tendons showing signs of pitting or transverse defects should be rejected under any circumstances. Adequate protection should be afforded to the tendon and its parts by the manufacturer to avoid corrosion and mechanical damage before delivery. At the site, care should be exercised during storage to ensure protection of the tendon prior to its homing.

Methods and criteria used to determine the protection level reflect the following factors:

(a) the intended service (economic) life of the foundation;
(b) the aggressivity of the environment;
(c) the consequence of failure caused by corrosion.

Correspondingly, it is evident, that the micropile foundations for OHL supports require, because of (c), a very reliable level of the corrosion protection.

As a general rule, permanent anchors, which micropiles in our case are, should be protected preferably with double protection. The design solution may be based on assumption that an aggressive environment will at some time exist, and that environment changes during the service life cannot be predicted. So exposure to aggressive conditions cannot be excluded.

Single and double protection. By definition, single protection constitutes one physical barrier (for example grout) between the steel tendon and the corrosive front. Double protection means that two such barriers are provided, and the main purpose of the outer barrier is to protect the inner barrier against damage during tendon handling and homing.

The protective system should satisfy the following requirements:

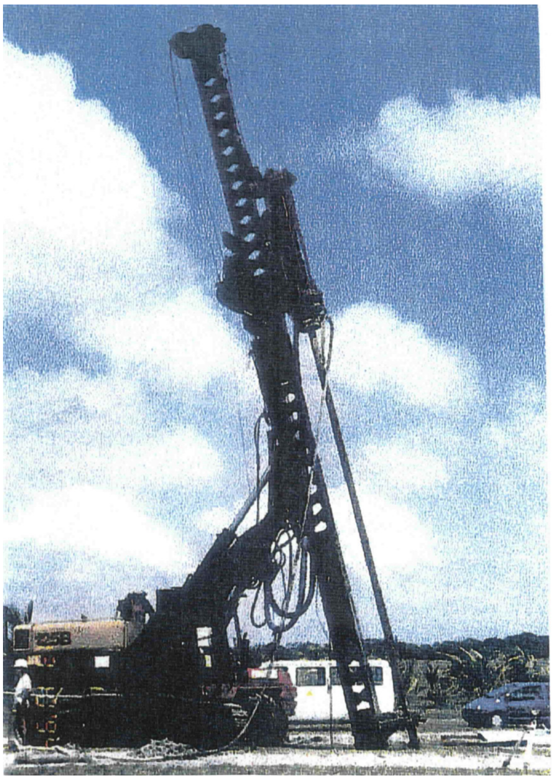
1) The system should have an effective life at least equal to service life of the OHL foundation.
2) Should not interact with the environment, and should not have adverse effects on the efficiency of the protected micropile foundation and its part.
3) Should consist of materials that are mutually compatible with the deformability and performance of the micropile, and will also inhibit potential induction of corrosive conditions.
4) The use of these materials should involve a single treatment, since protective systems can be neither replaced nor maintained.
5) The system should neither fail during stressing to proof load, nor disrupt the tendon-grout interaction, especially at junctions between components.
6) Should be flexible but strong enough to withstand handling stresses and distortions during manufacture, transport, and installation.
7) Should be delivered and packed in a manner allowing easy inspection before installation.

The prior requirements are satisfied if a protective system can exclude moist gaseous atmosphere around the metal by completely enclosing it within an impervious coating. The effectiveness of this treatment depends on maintaining the continuity of the coating; on the external fluid pressure gradients across coatings and joints; on the content and cleanliness of the atmosphere during application of the coating; on junction details; and on the electrochemical potential at the metal surface.

The industry offers a variety of protective coatings or coverings. The principles of protection are essentially the same for all parts of the system. Among the protective materials available are one or more coatings applied during manufacture of the tendon and basically attached to it. Grout injected in situ to form the anchorage zone and bond the tendon to the ground is not considered part of protective system unless its quality and integrity is assured.

Alternative forms of corrosion protection can take the form of using corrosion resistant steel, e.g. stainless steel.

5 Some examples of foundations for OHL towers using micropiles (anchors)

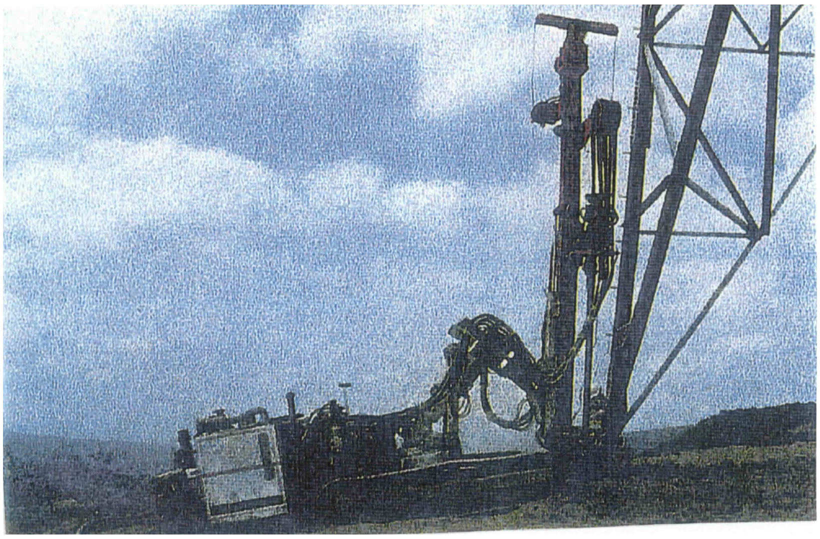


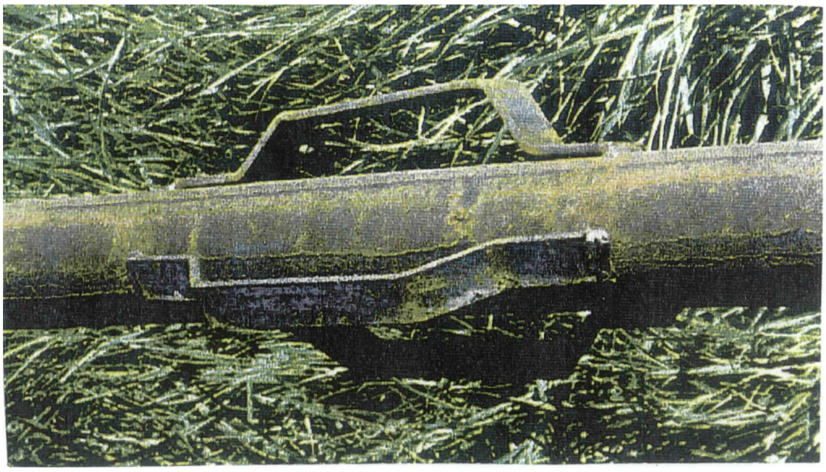
FRANCE, EDF
Grouted driven micropiles and metallic piles

FRANCE (EDF)
Screwed metallic piles

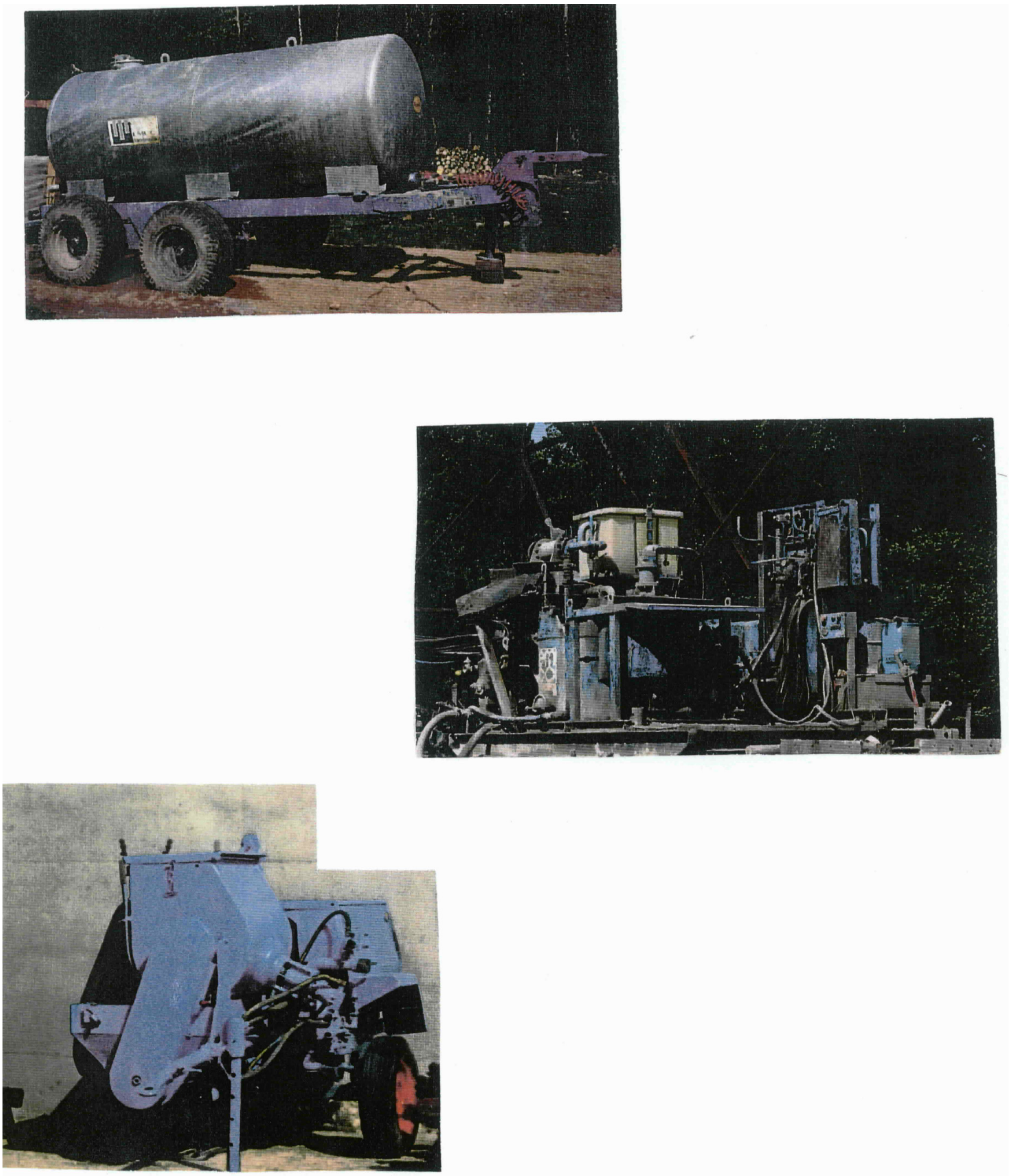
FRANCE (EDF)
Screwed metallic piles

FRANCE (EDF)
Grouted bored micropiles





FRANCE (EDF)
Grouted bored micropiles



FRANCE (EDF)

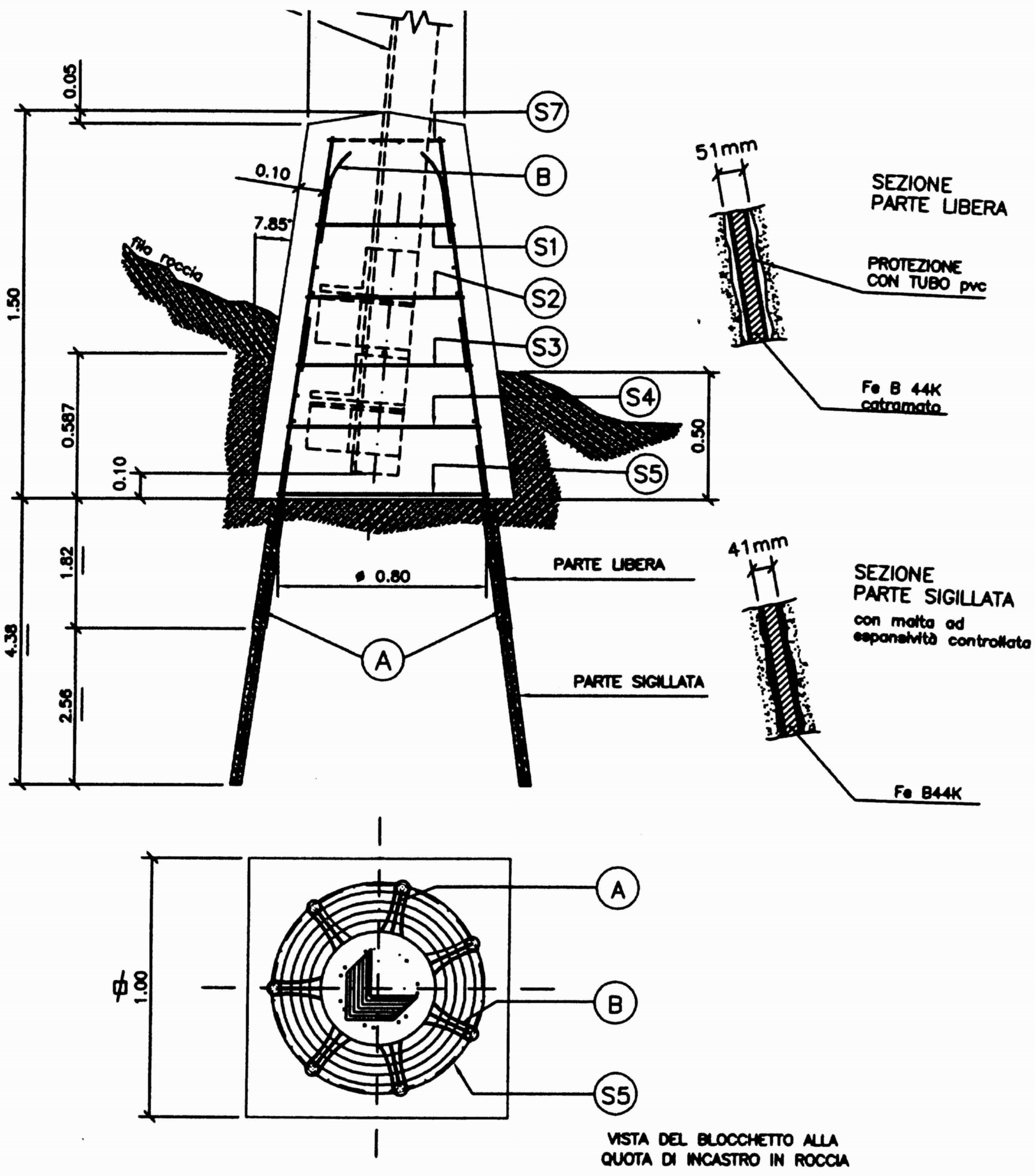
Grouted bored micropiles - Injection

FRANCE (EDF)

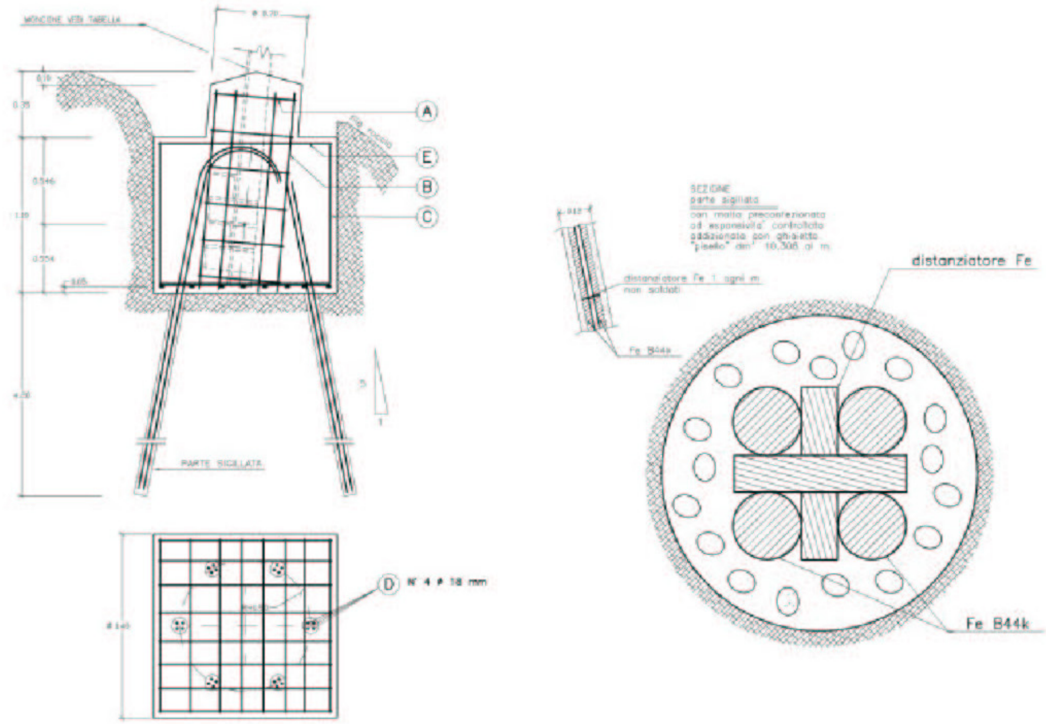
Grouted bored micropiles – Cutting footing

SLOVENIA, Micropile tensile strength test

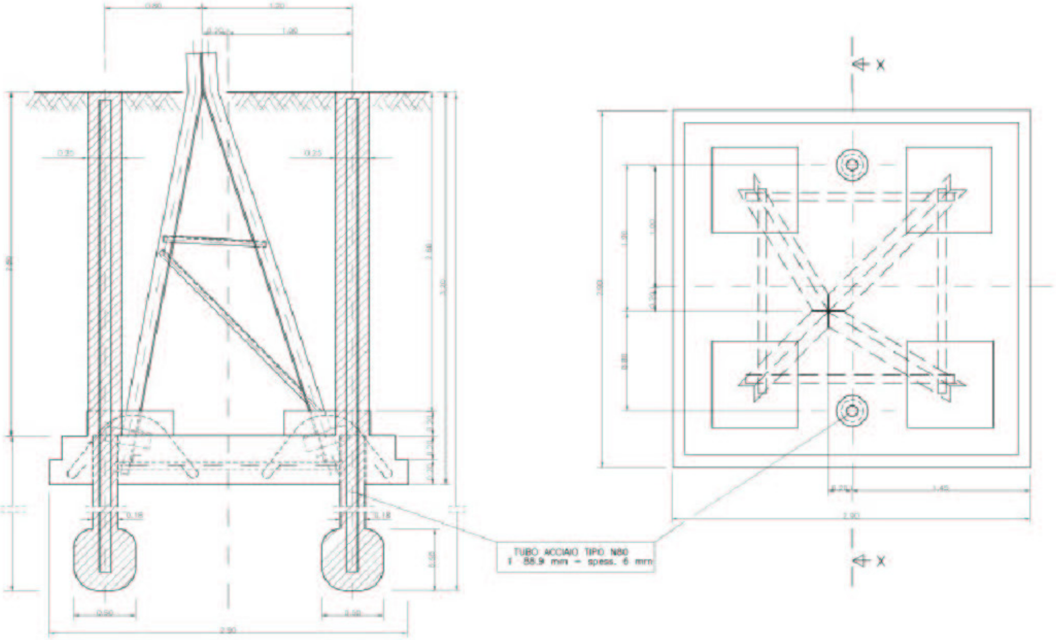
SLOVENIA, Cylindrical extension of the lower part of the micropile; used in hard expanding clay



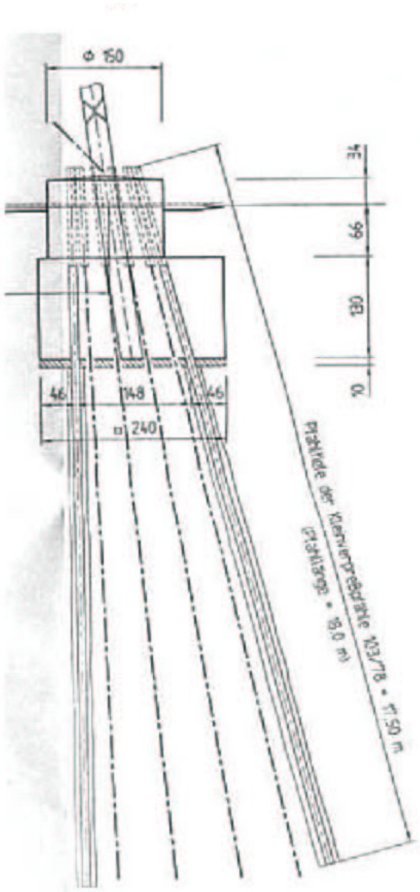
ITALY
Rock anchor foundation for 380-kV-single-circuit-tower
Ultimate loads: uplift/compression = 733/781 kN
horizontal = 23 kN



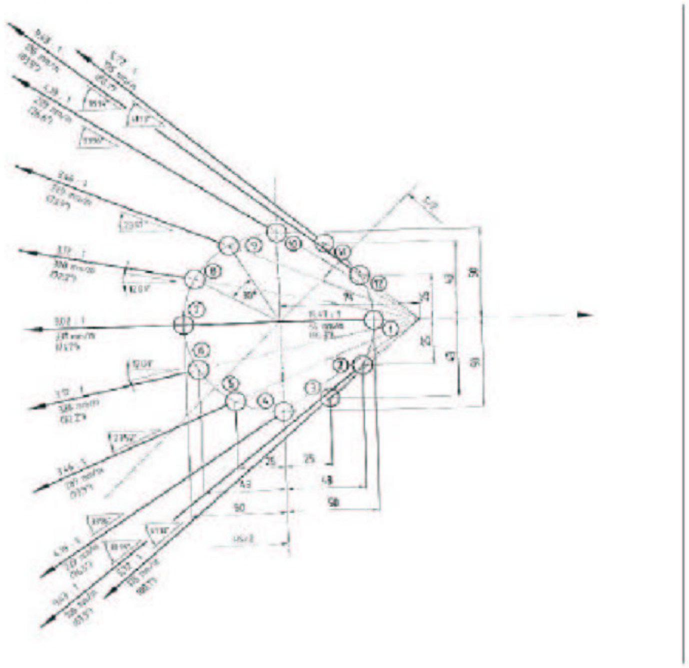
ITALY
Micropile-foundation in rock for 380-kV-double-circuit-tower
Ultimate loads: 189 kN-Horizontal, 1071 kN-Uplift, 1169 kN-Compression



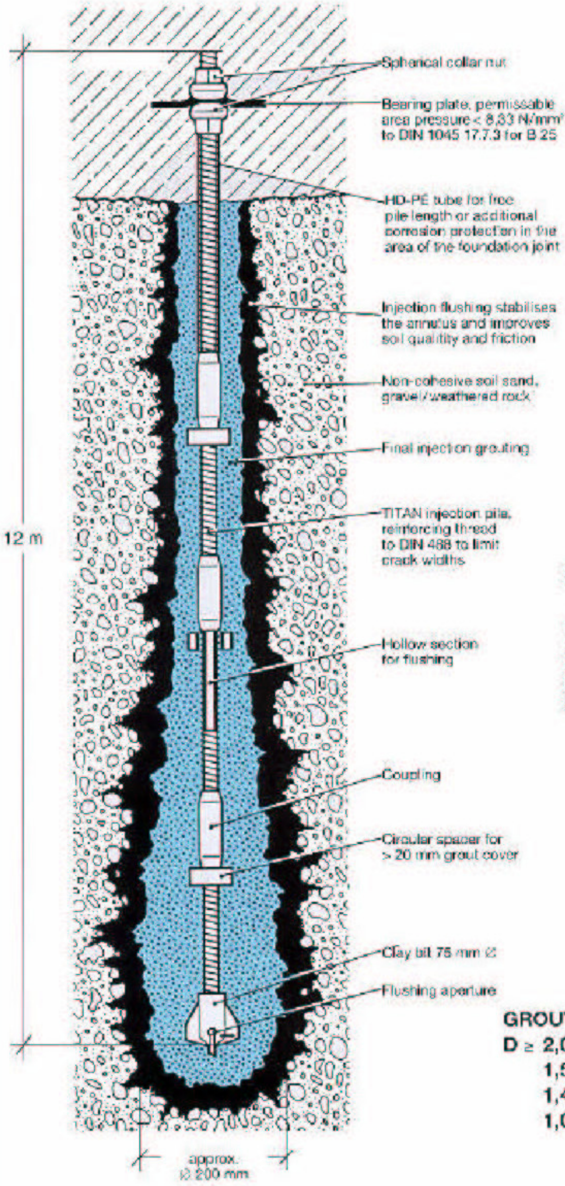
ITALY
Reinforcement of grillage foundation with micropiles (type TUBIX)

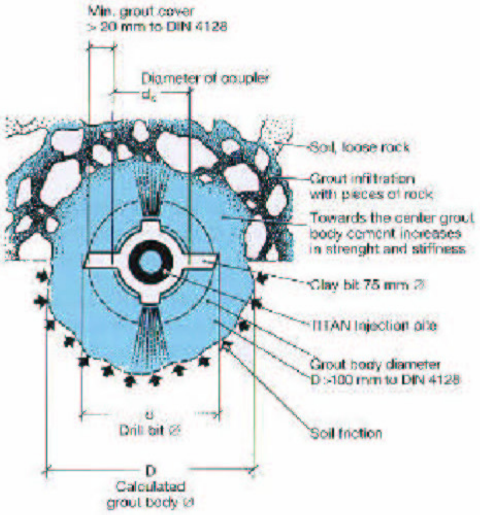


GERMANY
Micropile Foundation with
12 Ischebeck-piles
Working Loads:
Uplift 2260 kN
Compression 2700 kN



Self Drilling, Hollow Injection Grouted Micro Piles
complying with DIN 4128, EAU E 28 and draft CEN/TC 288/WG 8
with ultimate loads from 220 kN to 3460 kN (50 to 778 Kips)

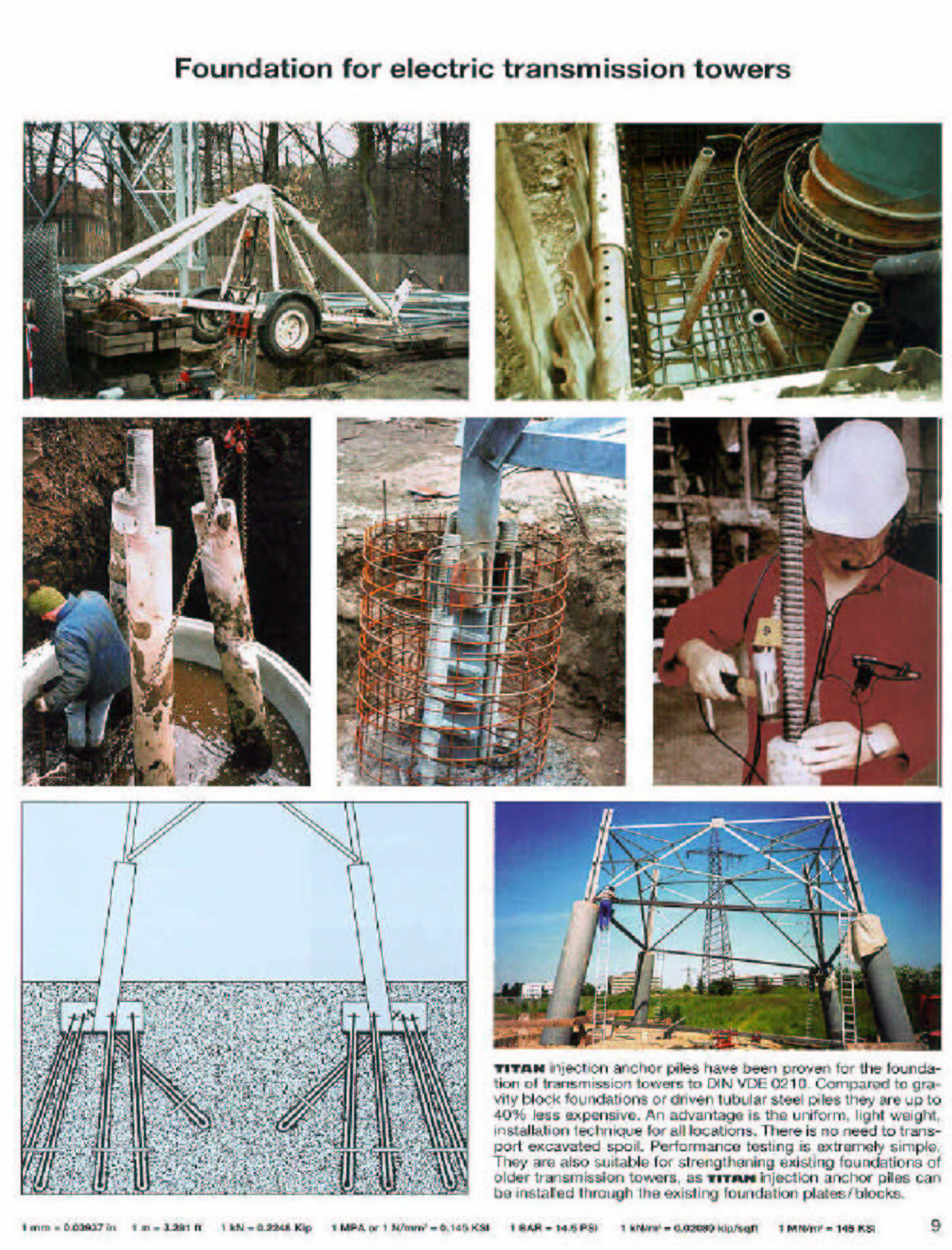




GROUT BODY DIAMETERS:

D ≥ 2,0 x d	**for medium + coarse cobbles**
1,5 x d	**for sand and gravelly sand**
1,4 x d	**for cohesive soil (clay, marl)**
1,0 x d	**for weathered sandstone, phylitte, slate**

GERMANY (Ischebeck)



Foundation for electric transmission towers

TITAN injection anchor piles have been proven for the foundation of transmission towers to DIN VDE 0210. Compared to gravity block foundations or driven tubular steel piles they are up to 40% less expensive. An advantage is the uniform, light weight, installation technique for all locations. There is no need to transport excavated spoil. Performance testing is extremely simple. They are also suitable for strengthening existing foundations of older transmission towers, as **TITAN** injection anchor piles can be installed through the existing foundation plates/blocks.

1 mm = 0.03937 in 1 m = 3.281 ft 1 kN = 0.2248 Kip 1 MPA or 1 N/mm² = 0.145 KSI 1 BAR = 14.5 PSI 1 kN/m² = 0.02089 kip/sqft 1 MN/m² = 145 KSI

Ischebeck micropiles
for OHL foundation
and reinforcing of
existing foundations

6 Site investigation and soil classification

The purpose of this annex is to describe basic site characterization and the soil and rock property evaluation for micropile and ground anchor design.

These activities generally include field reconnaissance, subsurface investigation, in-situ testing, and laboratory testing. The engineering properties and behavior of soil and rock material should be evaluated, since these materials provide the support for a micropile/anchor foundation.

Site investigation and testing programs are necessary to evaluate the technical and economical feasibility of a micropile/anchor foundation for an OHL-project application.

The extent of the site investigation and testing components for an OHL-project should be consistent with the project scope (i.e., location, size, critical nature of the structure, and budget), the project objectives (i.e., temporary or permanent structures), and the project constraints (i.e., geometry, constructability, performance, and environmental impact).

Typical elements of a site investigation and testing program are described herein.

6.1 Field reconnaissance

Field reconnaissance involves visual inspection of the site and examination of available documentation regarding site conditions. Information collected during field reconnaissance should include the following:

< surface topography and adjacent land use;
< surface drainage patterns, and surface geologic patterns including rock outcrops, landforms, existing excavations, and evidence of surface settlement;
< site access conditions and traffic control requirements for both investigation and construction activities;
< areas of potential instability, e.g. deposits of organic or weak soils, steep terrain slide debris, unfavorably jointed or dipping rock, and areas with a high groundwater table;
< extent and condition (e.g., visible damage, corrosion) of existing above and below ground utilities and structures; and
< available right-of-way (ROW) and easements required for the installation of micropile/anchor foundations.

6.2 Subsurface investigations

6.2.1 General

Subsurface investigation activities for micropile/anchor foundations typically involve soil borings and rock coring. As a minimum, it is recommended to perform at least one boring at each support location, or if possible one boring at each support leg. Information on the subsurface soil and rock stratigraphy and ground water conditions are typically obtained from subsurface investigation activities. Subsurface investigation may also involve conducting in-situ soil or rock tests and obtaining disturbed and undisturbed samples for subsequent laboratory testing.

6.2.2 Soil and Rock Stratigraphy

The soil and rock stratigraphy at the project site, including the thickness, elevation, and (if necessary) lateral extent of various layers, should be evaluated through implementation of a project-specific subsurface investigation. The following potentially problematic soils and rock should also be identified during the subsurface investigation, which may significantly affect the design and construction of the micropile/anchor foundations:

- Cohesionless sands and silts which tend to be unstable (i.e., cave-in) when exposed, particularly when water is encountered, and which may also be susceptible to liquefaction or vibration-induced densification;
- Weak soil or rock layers which are susceptible to sliding instability;
- Highly compressible materials such as high plasticity clays and organic soils which are susceptible to long-term (i.e., creep) deformations;
- Obstructions, boulders, and cemented layers which adversely affect anchor hole drilling, grouting, and encasing installation.

Borings should if possible be located at the corner legs of the tower site, or at the support centre-peg, so that stratigraphical information can be interpolated from the boring information.

Boring depths should, as a minimum, penetrate to a depth below the pile depth of 2 m in case of soil or 1 m in case of rock. Borings should be advanced deeper if there is a potential for soft, weak, collapsible, or liquefiable soils at depth. Additional borings may be required to characterize the geometry of a landslide slip surface

.

As a general recommendation, soil samples should be obtained at regular intervals, approximately 1.5 m deep and at all changes in the underlying soil strata for visual identification and laboratory testing. Methods of soil sampling include the Standard Penetration Test (SPT) and for cohesive soils, the use of thin-wall tubes. The cone penetration test (CPT) may be used, if necessary, to develop a continuous subsurface soil profile.

A minimum rock core of 3 m should be recovered for subsurface conditions where bedrock is encountered within the previously recommended investigation depths and for all designs that include rock anchors. A description of the rock type, mineral composition, texture (i.e., stratification, foliation), degree of weathering, and discontinuities is generally required. An estimate of intact rock strength can be evaluated using the percentage of core recovery and rock quality designation (RQD).

The orientations (i.e., strike and dip) of discontinuities and fractures should be included, whenever possible, in the rock description so that the potential for sliding instability can be evaluated. This latter information may be available from rock outcrop exposures at or near the site. For jointed rock, where the joints have been infilled with soil, the joint fill material should be sampled for subsequent laboratory shear strength testing. Soil samples and rock cores collected during the site investigation should be preserved and made available to the designer and the contractor during the design and bidding phase of a project, respectively.

6.2.3 Groundwater

The groundwater table and any perched groundwater zones should be evaluated as part of the subsurface investigation programme. The presence of ground water affects the overall stability of the system, may generate vertical uplift forces on structures, the watertightness requirements at anchor connections, corrosion protection requirements, and construction procedures. At a minimum, the following items need to be considered for micropile/anchor foundations that will be constructed within or near the groundwater table:

- average high and low groundwater levels;
- corrosion potential of micropiles/ground anchors based on the aggressivity of the ground water;
- necessity for excavation dewatering and specialized drilling and grouting procedures;
- liquefaction potential of cohesionless soils.

Groundwater level information is often obtained by the observation of the depth to which water accumulates in an open borehole at the time of, or shortly after, exploration. However, it is important to allow sufficient time to lapse after the borehole excavation, so that the water level can reach equilibrium. Groundwater level may be more accurately measured using piezometers or observation wells.

Water level measurements should be made over sufficient time duration to obtain an indication of potential water level fluctuations.

6.3 Laboratory Soil and Rock Testing

6.3.1 General

Laboratory testing of soil and rock samples recovered during subsurface exploration is often undertaken to evaluate specific properties necessary for the design of a micropile/anchor system. In this section, typical laboratory tests undertaken to evaluate soil and rock properties are presented.

6.3.2 Classification and Index Properties

All soil samples taken from borings and rock core samples should be visually identified in the laboratory and classified according to a classification system (e.g. ASTM D2488 and ASTM D2487, DIN 4022 or the Unified Rock Classification System (URCS)).

Index soil properties used in the analysis and design of micropile/anchor systems include unit weight, moisture content, gradation, and Atterberg limits. Unit weights of the foundation material and retained soil are used in evaluation of earth pressures and in determining the external stability of the micropile/anchor system. Moisture content and Atterberg limits may be used with existing correlations to estimate compressibility and shear strength of in-situ clay soils and to evaluate the suitability of micropiles/ground anchors in cohesive soils. In addition, the presence of organic materials should be determined by either a visual description or according to available standards. The results of soil particle size distribution may be used to develop appropriate drilling and grouting procedures and to identify potentially liquefiable soils.

6.3.3 Shear Strength

Unconfined compression, direct shear, or triaxial compression testing are typically performed to evaluate soil shear strength. Total stress and effective stress strength parameters of cohesive soils are typically evaluated from the results of undrained triaxial tests with pore pressure measurements.

For permanent anchor applications involving cohesive soils, both undrained and drained strength parameters should be obtained, and the design of the anchored system should consider both short-term and long-term conditions. For critical applications involving cohesionless soils, direct shear or triaxial compression testing can be used to evaluate drained shear strength. Typically, however, drained shear strength of cohesionless soil is usually determined based on correlations with in-situ test results (e.g., SPT and CPT). The selection of design soil shear strength for anchored systems is described in section 2.3.

Laboratory strength testing of intact rock samples is not often performed for micropile/anchor system applications. For the actual field conditions, the strength of the rock mass is typically controlled by discontinuities. If, however, no adverse planes of weakness exist, the compressive strength of the intact rock, evaluated using unconfined compression, direct shear, or triaxial compression testing, may be used to estimate the ultimate bond stress.

6.3.4 Consolidation

Settlement analyses are not usually performed for micropile/anchor systems constructed in stiff soils and cohesionless soils, but should be undertaken for foundations located in compressible soils subjected to groundwater drawdown (both during construction and for long-term conditions. Excessive settlement in these conditions may be detrimental to nearby structures and these settlements may result in long-term lateral movements of anchored systems that exceed tolerable limits. The results of index tests including moisture content and Atterberg limits can be used for initial evaluation of settlement parameters. Results of one-dimensional consolidation tests are used to evaluate the parameters necessary for a settlement analysis.

6.3.5 Electrochemical Criteria

For micropile/anchor foundations, the aggressiveness of the ground must be evaluated. Aggressive ground conditions usually do not preclude the use of micropiles/anchors, if proper corrosion protection for the micropile/anchor system is provided. Corrosion potential is of primary concern in aggressive soil applications and is evaluated based on results of tests to measure the following properties: pH-value, electrical resistivity, chloride content, and sulfate content.

Detailed information on micropile/ground anchor corrosion and corrosion protection measures is described in section 4.6.2.

6.4 In-Situ Soil and Rock Testing

In-situ testing techniques are often used to estimate several of the soil properties previously introduced in section 1.2.3. There are in-situ testing techniques which can be used to estimate rock properties, although the use of in-situ testing in rocks is not as widespread as the use in soils.

The SPT is the most common in-situ geotechnical test used, in evaluating the suitability of micropiles/ground anchors in cohesionless soils. The SPT blow count ‘N’ value can be used to estimate the relative density (see Table 4) and shear strength of sandy soils. The advantage of the SPT over other in-situ tests is that its use is widespread throughout the world and a disturbed sample can be obtained for visual identification and laboratory index testing. For cohesionless soils, SPT N<10 may indicate that the ground is not suitable for ground anchors. SPT blow counts may be used to evaluate the consistency of cohesive soil strata (see Table 4), but not as a reliable indication of shear strength.

Other in-situ testing procedures may be used to evaluate the suitability of ground anchors for a particular type of ground. These include: cone penetration test (CPT), vane shear test (FVT), pressure meter test (PMT), and flat plate dilatometer test (DMT).

TABLE 4: Soil density/ consistency description based on SPT blow count values (after AASHTO, 1988).

Cohesionless Soils		Cohesive Soils	
Relative Density	SPT N (blows/300mm)	Consistency	SPT N (blows/300mm)
Very loose	0 – 4	Very soft	0 – 1
Loose	5 – 10	Soft	2 – 4
Medium dense	11 – 24	Medium stiff	5 – 8
Dense	25 – 50	Stiff	9 – 15
Very dense	> 50	Very stiff	16 – 30
		Hard	31 – 60
		Very hard	> 60

Basic information on these tests is summarized in Table 5. Empirical correlations have been developed and may be used to obtain a preliminary estimate of property values. Details of these correlations have been published e.g., Kulhawy and Mayne, 1990 (7).

In many parts of the world, correlations have been developed for these tests in recognition of local soils and local conditions.

TABLE 5: Summary of common in situ tests for soils.

Type of Test	Suitable for	Not suitable for	Properties that can be estimated
SPT	sand	soft to firm clays, gravel	stratigraphy, strength, relative density
CPT	sand, silt, and clay	gravel	continuous evaluation of stratigraphy, strength of sand, undrained shear strength of clay, relative density, in situ stress, pore pressures
FVT	soft to medium clay	sand and gravel	undrained shear strength
PMT	soft rock, dense sand, nonsensitive clay, and gravel	soft, sensitive clays, loose silts and sands	strength, coefficient of earth pressure at rest (K_o), overconsolidation ratio (OCR), in situ stress, compressibility, hydraulic conductivity, elastic shear modulus
DMT	sand and clay	gravel	soil type, K_o, OCR, undrained shear strength, and elastic modulus

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8 STANDARDS

AASHTO (1988)
ASTM D2487
ASTM D2488
DIN 4022
DIN 4094 Soil: exploration by penetration tests. Deutsches Institut fuer Normung, 1990
DIN 4128
EN 1537 Execution of special geotechnical works, Ground Anchors, 2000
IEC 61773 Testing of foundation for structures, IEC, 1996
Unified Rock Classification System (URCS)