Drilled and Grouted Micropiles: State-of-Practice Review

Volume III: Construction, QA/QC, and Testing

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FOREWORD

The four-volume series that constitutes the state-of-practice review is the larger of two deliverables from the contract let in September 1993 on drilled and grouted micropiles. The volumes cover all aspects of the technology, with special reference to practices in the United States, France, Italy, Germany, and Great Britain – those countries that are most active. This final report was originally prepared as one document. However, its length is such that it is now divided into four separate volumes, each containing certain groups of chapters from the original final report.

Volume I (FHWA-RD-96-016) provides a general and historical framework and a new classification of micropile types based on both the concept of design and the mode of construction (chapter 1). Chapter 2 introduces the applications in a structured format, while chapters 3 and 4 deal with feasibility and cost, and contracting practices, respectively. Volume II (FHWA-RD-96-017) reviews design. Chapter 1 covers the design of single micropiles, chapter 2 covers groups of micropiles, and chapter 3 covers networks of micropiles. Volume III (FHWA-RD-96-018) includes a review of construction methods (chapter 1) and provides an introduction to specifying QA/QC and testing procedures (chapter 2). Volume IV (FHWA-RD-96-019) is a summary of 20 major case histories specially chosen to illustrate the various principles and procedures detailed in volumes I, II, and III.

These volumes together are intended as a reference work for owners, designers, and contractors, and as a statement of current practice to complement the companion French national research program, FOREVER.

Charles J. Nemmers, P.E.
Director, Office of Engineering
Research and Development

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16. Abstract

Micropiles are small-diameter, drilled and grouted reinforced piles used for both structural support and in situ earth reinforcement. They were conceived in Italy in 1952, but have become popular in the United States only since the mid-1980's. This report provides a comprehensive state-of-practice review, drawing on data from an international basis. Volume I provides a general and historical framework, and new classifications of type and application. Cost and feasibility are also discussed. Volume II deals with the design of single piles, and groups and networks of piles. Volume III reviews construction, QA/QC, and testing issues, while Volume IV provides summaries of 20 major case histories illustrating the principles and procedures of volumes I, II, and III.

This volume is the third in a series. The other volumes in the series are:

FHWA-RD-96-016 Volume I: Background, Classification, Cost
FHWA-RD-96-017 Volume II: Design
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DEDICATION

This study is dedicated to Dr. Fernando Lizzi, of Napoli, Italy, whose technical acumen in developing the concept of micropiles has been matched only by his imagination in applying them. Since obtaining the first micropile patents in 1952, Dr. Lizzi has overseen the growth in their use on five continents. He has been inspirational to all associated with preparing this study, and doubtless will remain so to all those who read it.

Fernando Lizzi
"The Father of Micropiles"

PREFACE

When designing this study, the Federal Highway Administration recognized the necessity of ensuring input by practicing engineers, in general, and those in Europe, in particular. This was reflective of the origins of micropiles and of the countries of most common use.

This input has been forthcoming to the Principal Investigators through both written submittals and commentaries on drafts, and through the attendance of these specialists at a series of workshops.

At the first workshop held in Washington, DC, March 10-11, 1994, discussions were held about the structure and purpose of the study, and attendees made presentations on local and national practices. By the second workshop, also in Washington, DC, October 27-28, 1994, several chapters had been prepared in draft form, and these were reviewed by the group. At the third workshop in San Francisco, March 10-13, 1995, all chapters were reviewed in anticipation of concluding the Final Draft Report, and considerable verbal and written comments were received. In addition, the International Advisory Board also provided the Principal Investigators with published and unpublished data.

Throughout this report, all such published or unpublished written reports are duly acknowledged. However, there are numerous examples of statements made by individual participants that are not specifically listed. These statements were made during the workshops and have not been separately referenced because: (1) this saves space and improves the flow of the text, and (2) other researchers have no means of retrieving such unwritten references. This report also contains information obtained by the Principal Investigators on study trips to specialists in Europe.

ACKNOWLEDGMENTS

The Principal Investigators extend their sincerest thanks to their colleagues in the Nicholson Construction Company and at the Polytechnic University of Brooklyn, New York, for their assistance. In addition, they wish to thank the members of the International Advisory Board for their vital contributions, both written and verbal. Federal regulations prevent the names of the participants from being listed.

Data were also provided by specialists in the following commercial concerns:

Terrasol (France)
Turner Geotechnical Associates (United Kingdom)
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Franki (South Africa, via Dr. Ross Parry-Davies)
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California Department of Transportation (U.S.A.)
Ground Engineering (U.S.A.)

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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CHAPTER 1. CONSTRUCTION PRACTICES

INTRODUCTION

The load-carrying capacity of a micropile as related to grout/soil bond capacity is highly sensitive to the major processes used during construction, principally the techniques used for drilling, flushing, and grouting. Figure 1 illustrates a typical sequence of construction for a simple Type A or B micropile, although as is described below, there is a wide range of details, depending on national practice and constructor preference and experience. In general, the drilling and grouting equipment and systems used for micropile construction are similar to those used for the installation of soil nails, ground anchors, and grout holes (figures 2, 3, and 4).

GENERAL OVERVIEW OF CONSTRUCTION TECHNIQUES

Drilling

The drilling method is chosen to impart minimal disturbance or upheaval to the structure or soil, while being the most efficient, economic, and reliable means of penetration. Micropile holes must often be drilled through an overlying weak material to reach a more competent bearing stratum. They therefore typically require the use of overburden drilling techniques to penetrate and support weak and unconsolidated soils and fills. In addition, unless the bearing stratum is rock or a self-supporting material such as a stiff clay or marl, the drill hole may need temporary support for its full length. An exception to this, for example, would be where piles are required to strengthen masonry, weak, or broken rock. In such a case, the drill hole can possibly be formed by open-hole techniques, i.e., without the need for temporary hole support by drill casing or hollow stem auger.

In addition, it is common for a different drilling method to be used first to penetrate through an existing structure. This operation often requires concrete coring techniques to provide an oversize hole to allow the subsequent drill casing to pass through. In some cases, conventional rock drilling methods involving rotary percussive techniques can be used to penetrate existing footings or structures with only light reinforcement and where a certain level of vibrational energy can be tolerated.

Water is the most common cooling, cleansing, and flushing medium, followed by foam. Air flush (except for down-the-hole hammer drilling) is used only infrequently and with great caution, especially in urban or industrial environments. The use of bentonite slurries to stabilize holes instead of using casing is generally believed to impair grout/ground bond capacity (by creating a skin of clay at the interface), although this is not an uncommon choice in Italian and French practice with Type D piles.

Drilling techniques as introduced in table 1 are summarized below (Bruce, 1989) and are illustrated in figure 5.

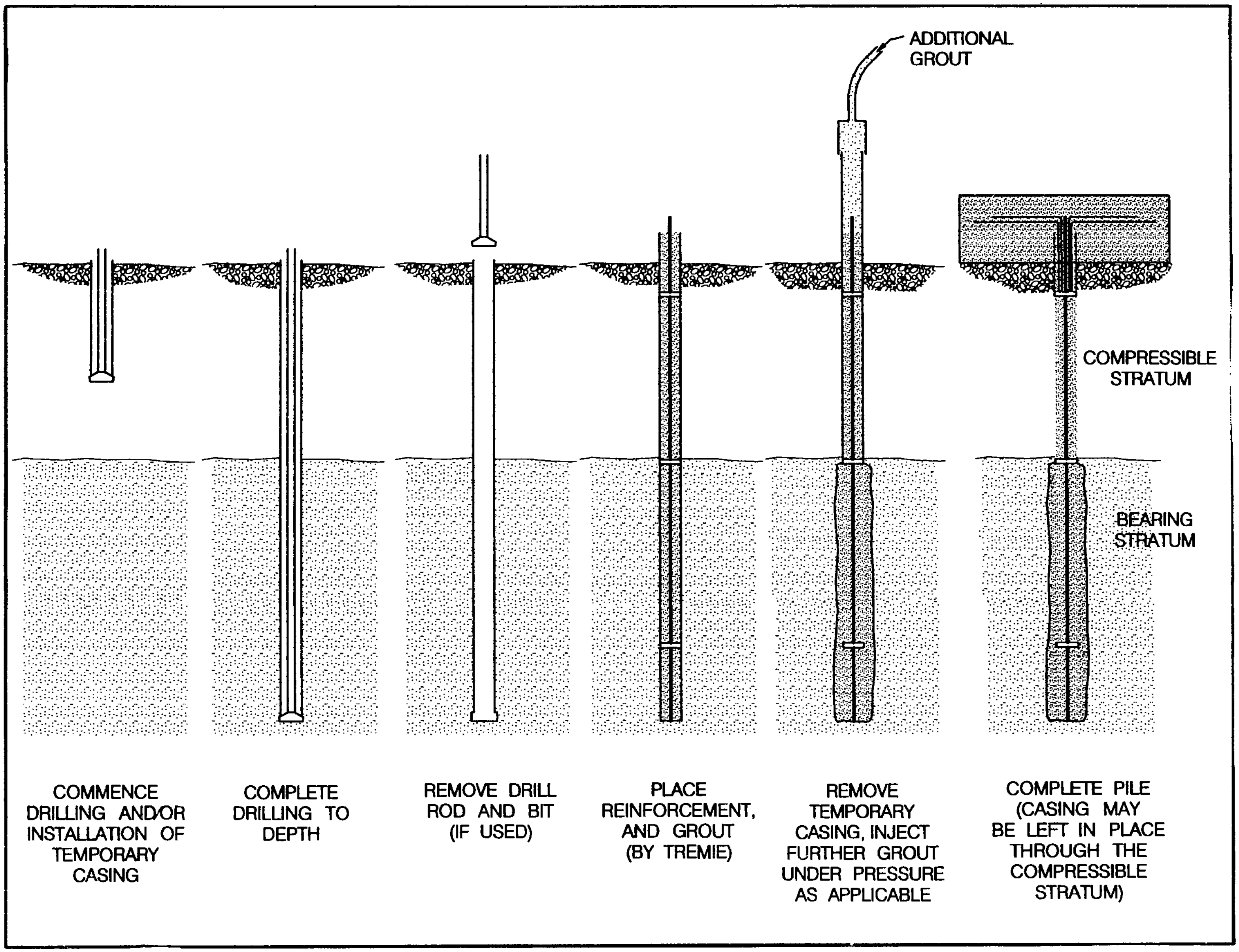


Figure 1. Typical construction sequence for a Type A or B micropile.

Figure 2. Long mast diesel-hydraulic track rig installing test micropiles. Reinforcement consists of both drill casing (bottom center) and bars (with centralizers, bottom right). Typical grout mixer/pump unit shown at bottom left.

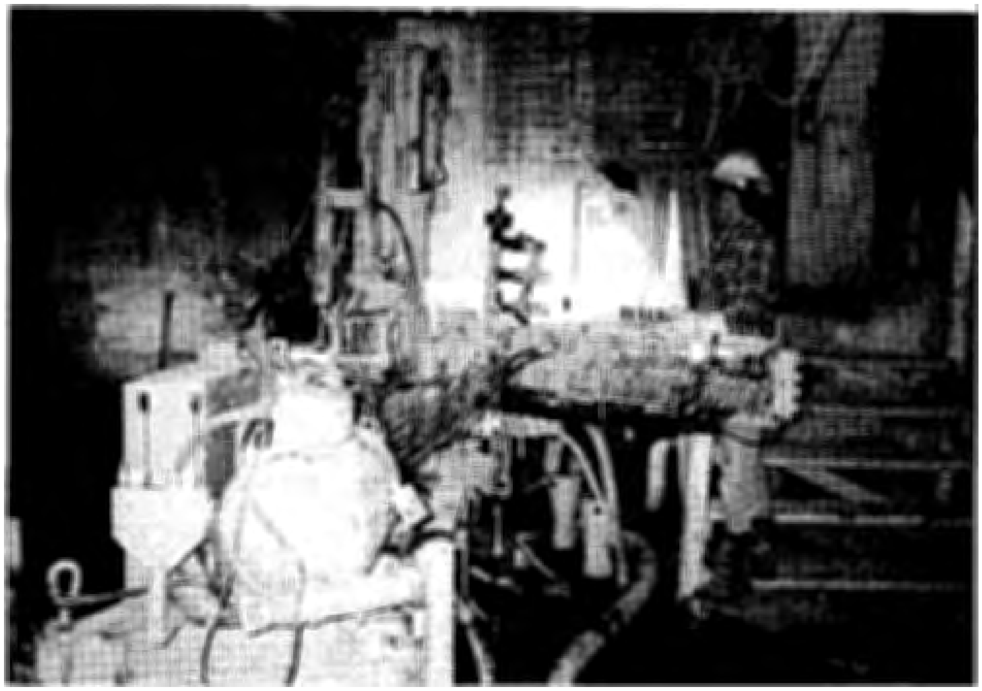


Figure 3. Short mast electro-hydraulic drilling rig operating in very restricted access and overhead conditions.

Figure 4. Similar micropile rig drilling in close proximity to wall. (Photograph courtesy of Hutte and Co.)

Table 1. Overburden drilling methods (Bruce, 1989).

DRILLING METHOD	PRINCIPLE	COMMON DIAMETERS AND DEPTHS	NOTES
1. Single tube advancement			
a) Drill drilling	Casing, with "lost point" percussed without flush.	50 - 100 mm to 30 m	Obstructions or very dense soils problematical.
b) External flush	Casing, with shoe, rotated with strong water flush.	100 - 200 mm to 60 m	Very common for anchor installation. Needs high torque and powerful flush pump.
2. Rotary duplex	Simultaneous rotation and advancement of casing plus internal rod, carrying flush.	100 - 200 mm to 70 m	Used only in very sensitive soil/ site conditions. Needs positive flush return. Needs high torque.
3. Rotary percussive concentric duplex	As 2, above, except casing and rods percussed as well as rotated.	89 - 175 mm to 40 m	Useful in obstructed/bouldery conditions. Needs powerful top rotary percussive hammer.
4. Rotary percussive eccentric duplex	As 2, except eccentric bit on rod cuts oversized hole to ease casing advance.	89 - 200 mm to 60 m	Somewhat obsolescent and technically difficult system for variable overburden.
5. "Double head" duplex	As 2 or 3, except casing and rods rotate in opposite senses.	100 - 150 mm to 60 m	Powerful, new system for fast, straight drilling in very difficult soils.
6. Hollow stem auger	Auger rotated to depth to permit subsequent introduction of reinforcement through stem.	150 - 400 mm to 30 m	Obstructions problematical; care must be exercised in cohesionless soils to avoid cavitation and/or loosening. Prevents application of higher grout pressures.

Note: Drive drilling, being purely a percussive method, is not described in the text as it has no application in micropile construction.

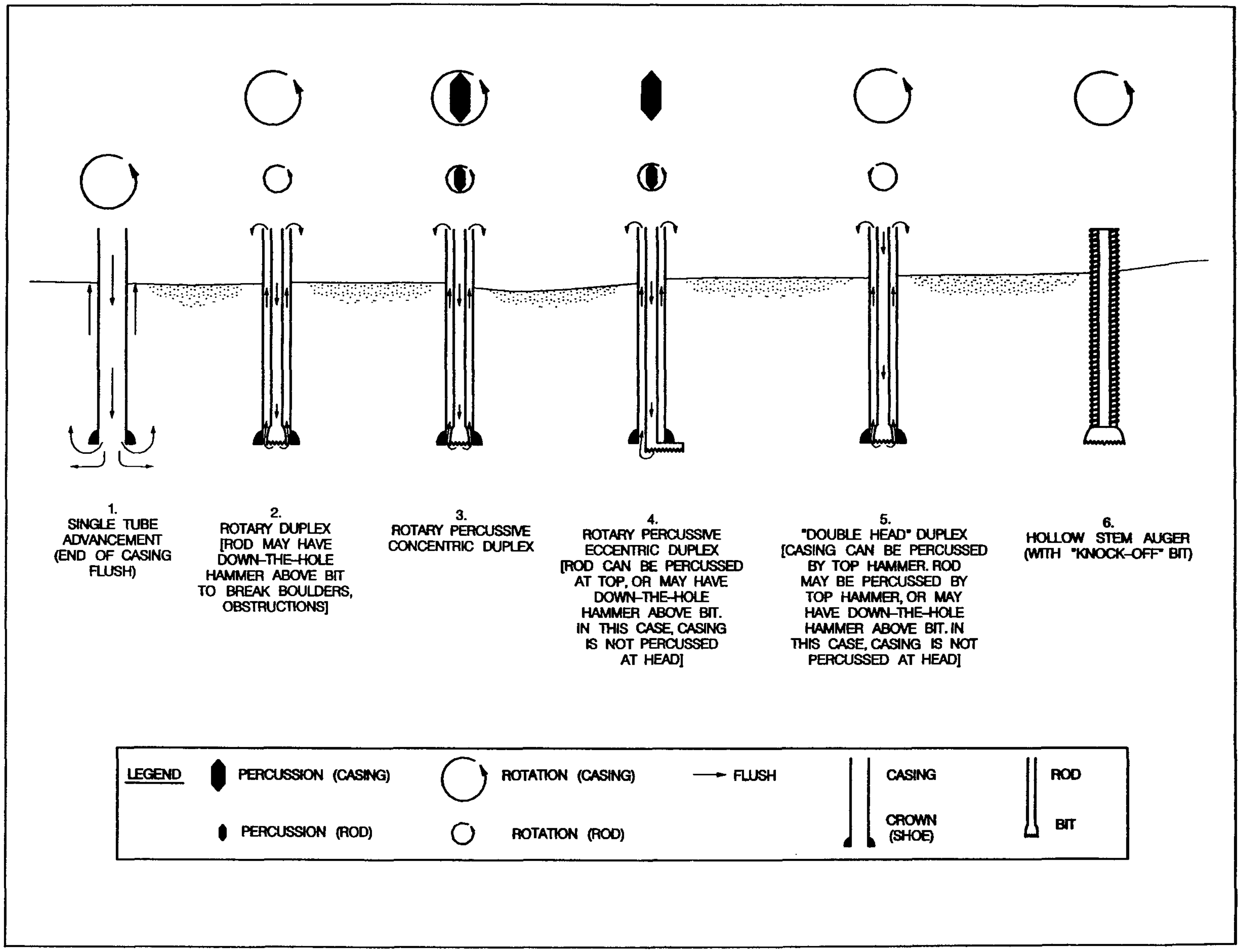


Figure 5. Schematic representation of the six generic overburden drilling methods (Bruce, 1989).

Overburden Drilling Techniques

There is a large number of proprietary overburden drilling systems sold by drilling equipment suppliers worldwide. In addition, specialty contractors have often developed their own variations in response to local conditions and demands. The result is a potentially bewildering array of systems and methods, which does, however, contain many that are of limited application, and many that are either obsolete or virtually experimental. Closer examination of this array further confirms that there are essentially only six generic methods in use internationally in the field of specialty geotechnical construction (i.e., hole diameters less than 300 mm, and depths less than 60 m).

Single tube advancement - external flush (wash boring)

The toe of the drill casing is fitted with an open crown or bit, and the casing is advanced into the ground by rotation by the drill head. Water flush is pumped continuously through the casing and washes debris out and away from the crown. The waterborne debris typically escapes to the surface around the outside of the casing, but may be lost into especially loose and permeable upper horizons. Care has to be exercised below sensitive structures that uncontrolled washing does not damage the structure by causing cavitation.

Air flush is not normally used with this system, due to the dangers of accidentally over-pressurizing the ground in an uncontrolled manner and causing ground disturbance. Conversely, experience (Bruce et al., 1992) 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/soil bond development as may be the case with bentonite slurries.

Rotary duplex

A drill rod with a suitable drill bit is placed inside the drill casing. It is attached to the same drive head as the casing. This allows the 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 through the annulus between the drill rods and the casing. At the surface, the flush exits through ports in the drill head. 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, although this danger with duplex is less than when using the single tube method.

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.

Rotary percussive duplex (eccentric)

Originally sold as Overburden Drilling Eccentric (ODEX), this system involves the use of rotary percussive drilling combined with an eccentric under-reaming bit. The eccentric bit undercuts the drill casing, which therefore can be pushed into the oversized drill hole with much less rotational energy or thrust than is the case with the concentric method. In addition, the drill casing does not require an expensive cutting shoe and suffers less wear and abrasion.

The larger diameter options – more than 127 mm in diameter – 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. This is synonymous with air flush drilling which may not be acceptable for certain micropile applications, although foam can also be used with a down-the-hole hammer. This method has proved very useful in projects involving karstic limestone bedrock conditions.

Most recently, similar systems to ODEX, now sold as TUBEX, for proprietary reasons, have appeared. Some are merely mechanically simpler versions of TUBEX, whereas others, such as the new Japanese system "Supermaxbit," are more distant cousins. 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

As 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 thus enhanced soil and obstruction cutting potential), but with the penalty of relatively 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 counterrotation also discourages blockage of the casing/rod annulus by debris carried in the exiting drill flush. In addition, the inner rods can use either purely rotary techniques or rotary percussion using top-drive or down-the-hole hammers. The counterrotation feature promotes exceptional hole straightness, and provides a guarantee of 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.

Various forms of cutting shoes or drill bits can be attached to the lead auger, but heavy obstructions such as old foundations are difficult to penetrate economically with the system. In addition, great care must be exercised when using augers: uncontrolled penetration rates or excessive "hole cleaning" may lead to excessive spoil removal and the risk of soil loosening or cavitation in certain circumstances. As a guide, the *DFI Augercast Pile Manual* (1990) allows a minimum spoil volume of 115 percent of nominal hole volume, although for micropiles, the 110 percent figure allowed by BS8081 (1989) for anchor holes is a more realistic maximum value.

Open-Hole Techniques

When the pile can be formed in stable and free-standing conditions, the advancement of casing may be suspended and the hole continued to final depth by open-hole drilling techniques. There is a balance in cost between the time lost in changing to a less expensive open-hole system and continuing with a more expensive overburden drilling system for the full hole depth. Open-hole drilling techniques may be classified as follows:

Rotary drilling

Rotary drilling involves using only rotary energy provided by the drilling head. This method typically employs drag or tri-cone drilling bits and a suitable flushing medium (usually water or foam flush). In almost all cases, a full-face bit is used (i.e., one that cuts across the full diameter of the borehole). Rigs used for rotary drilling must be capable of applying substantial thrust to the bit to obtain high productivities.

Rotary percussive drilling

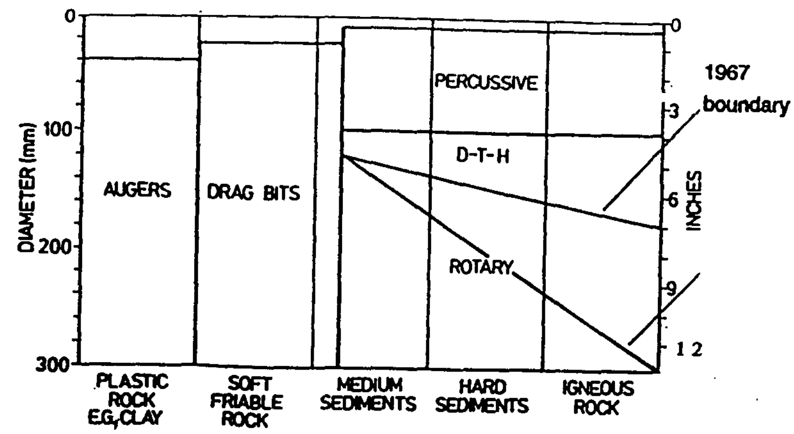
Particularly for rocks of high compressive strength, rotary percussive techniques using either top-drive or down-the-hole hammers are utilized. With the hole diameters typically used for micropiles (i.e., 100 to 300 mm), down-the-hole techniques are the most economical and common, and use air, air/water mist, or foam as the flush.

Top-drive systems can permit the use of air, water, or other flushing systems, but are of limited diameter and depth capacities, and are relatively noisy.

Continuous solid-core flight auger

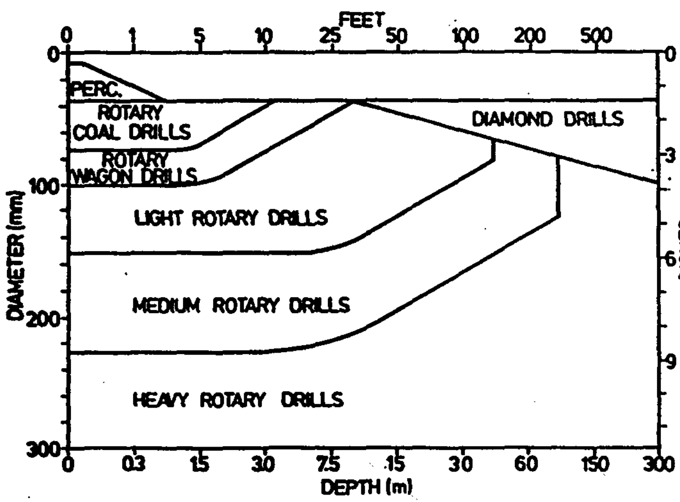
In stiff to hard clays without boulders and in some weak rocks, drilling may be conducted with a continuous flight auger. Such drilling techniques are rapid, quiet, and do not require the introduction of a flushing medium to remove the spoil. They thus avoid the problems of soil softening and interface smear associated with the use of rotary techniques with water flush, although there may be the risk of lateral decompression or wall remolding, both of which may adversely affect grout/soil bond. Such augers are used in conditions where the careful collection and disposal of drill spoils are particularly important environmentally.

A guide to selection of the appropriate rock-drilling method is provided in figure 6. These charts were first developed in the middle 1960's; since then, many technological developments have been made, especially in the use of down-the-hole (D-T-H) drilling, as noted in figure 6a.

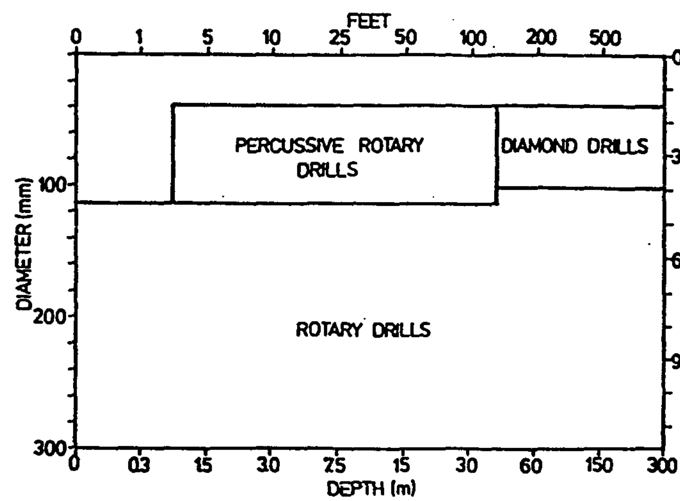


(McGregor, 1967)

a) Preferred methods of drilling different classes of rock and at different hole diameters. Depth of hole generalized.



b) Preferred methods in soft friable rocks. (McGregor, 1967)



c) Preferred methods in variable strata. (McGregor, 1967)

Figure 6. Guide to selection of rock-drilling methods (Littlejohn and Bruce, 1977).

Underreaming

Various devices have been developed to enlarge or underream open holes in cohesive soils or soft sediments, especially when the piles are to act in tension, for example, under transmission towers. These tools can be mechanically or hydraulically activated and will cut or abrade single or multiple underreams or "bells." However, this is a time-consuming process, and it is rarely possible or convenient to verify its effectiveness. In addition, the cleaning of the underreams is often difficult; water is the best medium, but it may cause softening of the ground. For all these reasons, it is rare to find underreaming conducted in contemporary micropile practice, and increases in load-holding capacity favor pressure-grouting techniques.

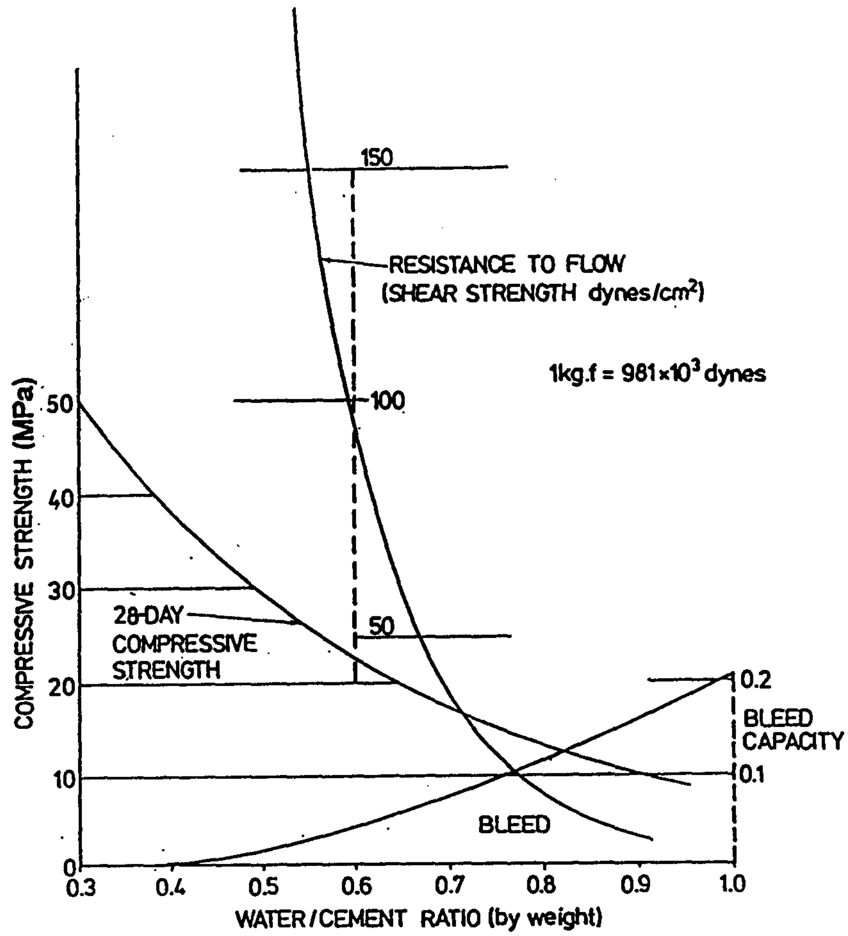


Figure 7. **Effect of water content on grout properties (Littlejohn and Bruce, 1977).**

Grouting

As described in volume I, the processes of the grouting operation have major control over subsequent micropile capacity, and indeed form the most fundamental construction basis for micropile classification. Details of each type of grouting vary somewhat throughout the world, depending on the origins of the practice and the quality of the local resources. However, as general observations, it may be noted that:

- Grouts are designed to provide high strength and stability, but must also be pumpable. As shown in figure 7, this implies water/cement (w/c) ratios in the range of 0.45 to 0.50 by weight for micropile grout.
- Grouts are produced with fresh water, to reduce the danger of reinforcement corrosion.

- The best quality grouts, in terms of both fluid and set properties, are produced by high-speed, high-shear mixers (figure 8) as opposed to low-speed, low-energy mixers, such as those that depend on paddles (figure 9).

- Cement is supplied either in bagged or bulk form, depending on site condition, job size, local availability, and cost.

- Equipment can be driven by air, diesel, or electricity and is available in a wide range of capacities and sizes from many manufacturers.

- Lower pressure injection (to 1 MPa) is usually effected via constant pressure, rotary-screw-type pumps (figure 10); while higher pressure grouting, such as for Type C or D micropiles, usually requires a fluctuating pressure piston or ram pump (figure 11).

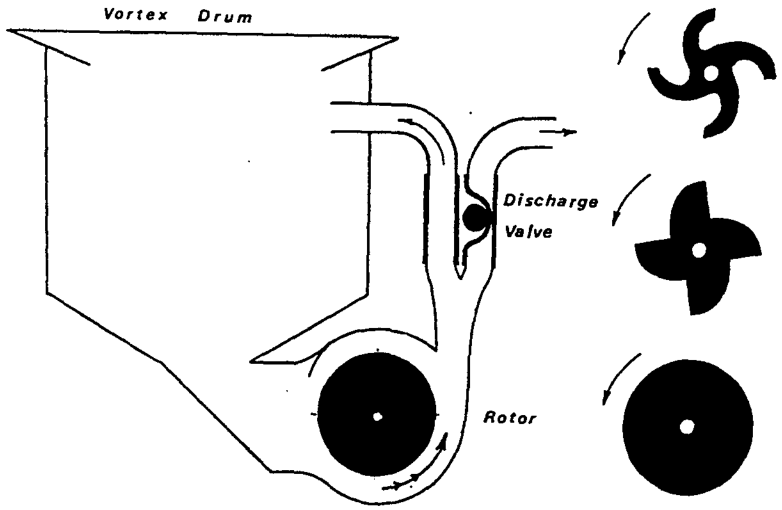
- Most grouting is conducted with neat cement-water mixes, although sand is a common additive in certain countries (e.g., Italy and Great Britain). Bentonite (which reduces grout strength) is used in primary mixes only with extreme caution, while additives are restricted only to those that improve pumpability over long distances and/or in hot conditions (e.g., high-range water reducers).

Comprehensive guides to cement grout mix design, performance, and equipment in general are provided by Littlejohn (1982); Gourlay and Carson (1982); and Houlsby (1990). Similar issues relating solely to the similar demands of prestressed ground anchors are summarized by Littlejohn and Bruce (1977).

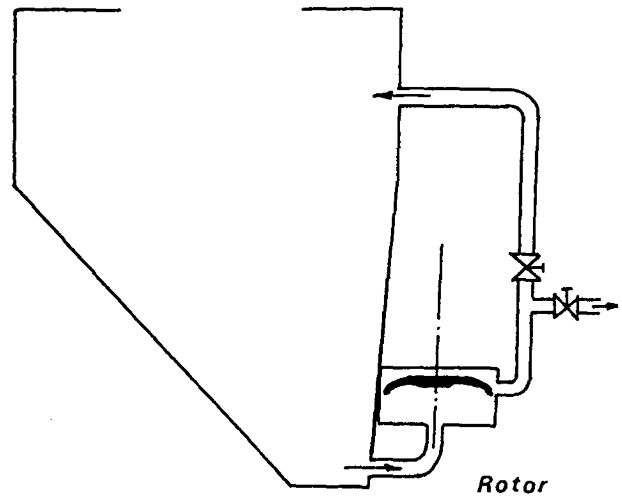
Gravity Fill Techniques (Type A Micropiles)

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 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.

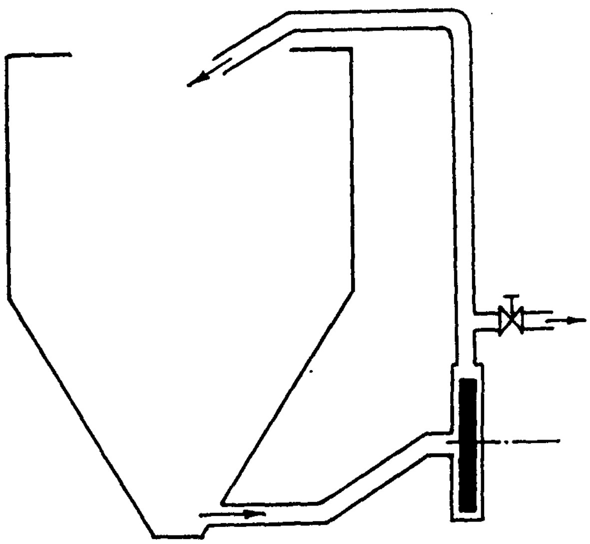
The grout usually comprises a neat cement mix with w/c ratio between 0.45 and 0.50 by weight. In addition, sanded mixes of up to 1:1 or 2:1 sand/cement ratio have been used in European practice, although they are becoming less common as there is a growing trend towards types of micropiles using higher pressures, which therefore require unsanded grouts. Gravity fill techniques tend to be used only when the pile is founded in rock, or when low-capacity piles are being installed in stiff or hard cohesives, and pressure grouting is unnecessary to generate adequate bond (e.g., Bruce and Gemme, 1992). For sanded mixes, the w/c ratio is often extended to 0.60 (e.g., Barley and Woodward, 1992).



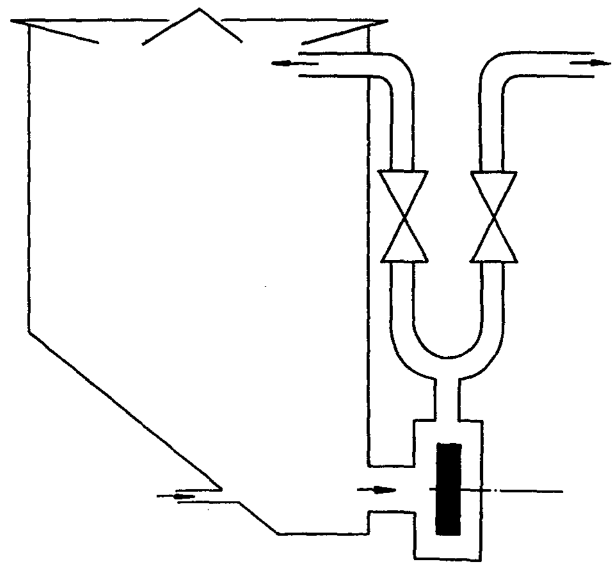
Colcrete mixer. Rotor shapes: (Upper) Solid disc for cement mixing only. (Middle) Discar for cement and sand. (Lower) For sand (in double-drum model).



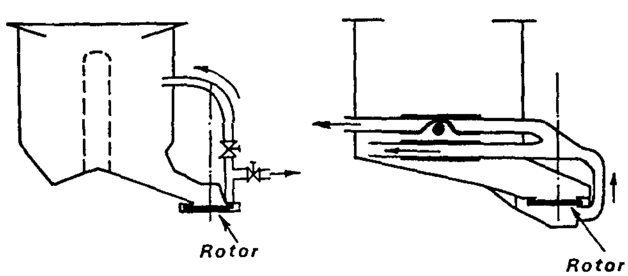
Häny mixer.



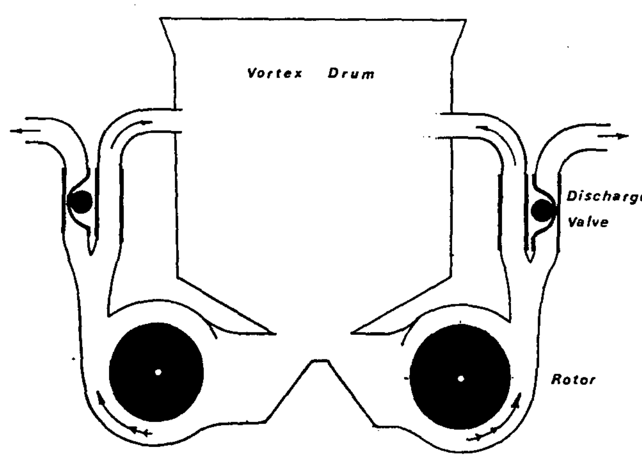
The original Chemgrout "colloidal" mixer.



Sketch of latest Chemgrout mixer.

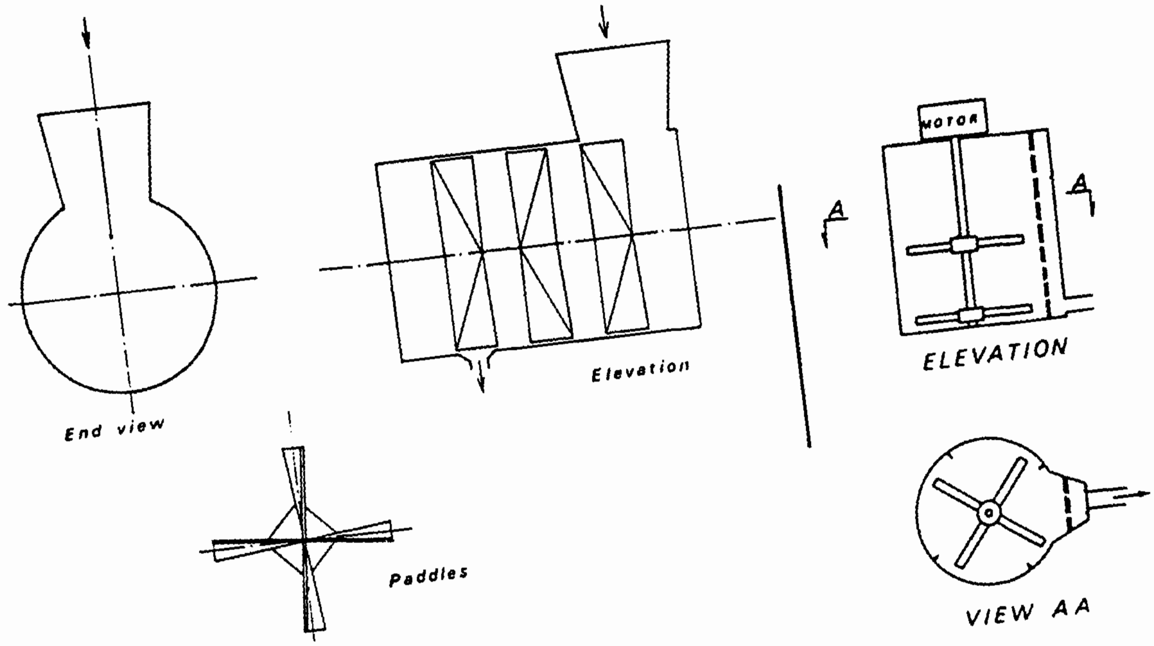


Cemix mixers. Left: Model 175. Right: Model 200.

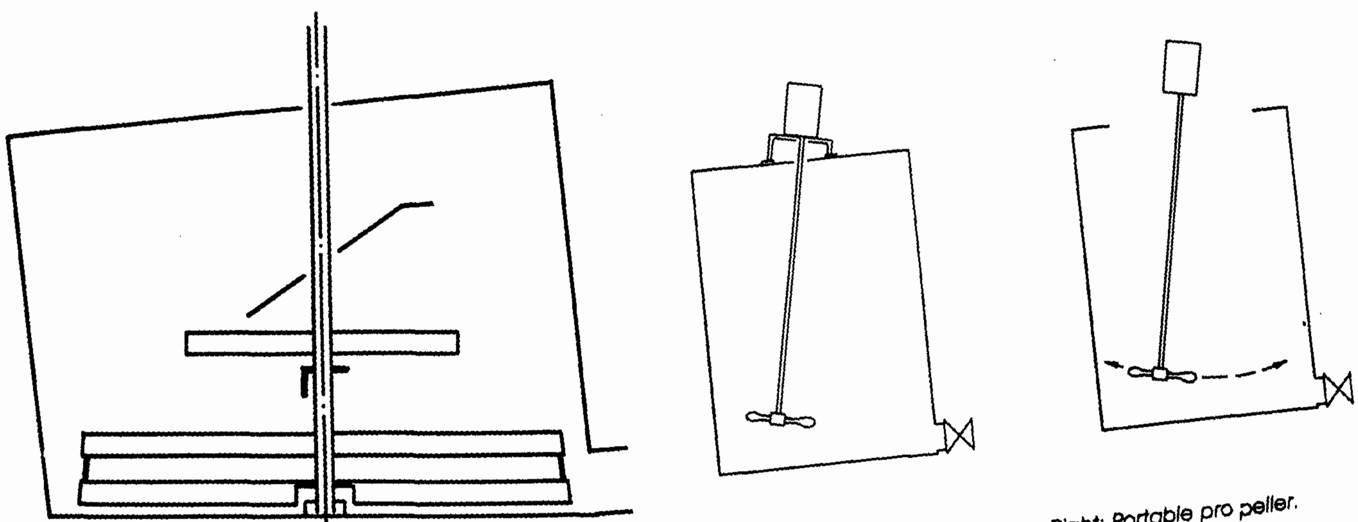


Double-rotor Colcrete mixer.

Figure 8. Various types of colloidal mixers (Houlsby, 1990).



Left: Horizontal paddle mixer. Right: Vertical paddle mixer.



Left: Propeller fixed in drum. Right: Portable pro peller.

Chemgrout paddle mixer.

Figure 9. Various types of paddle mixers (Houlsby, 1990).

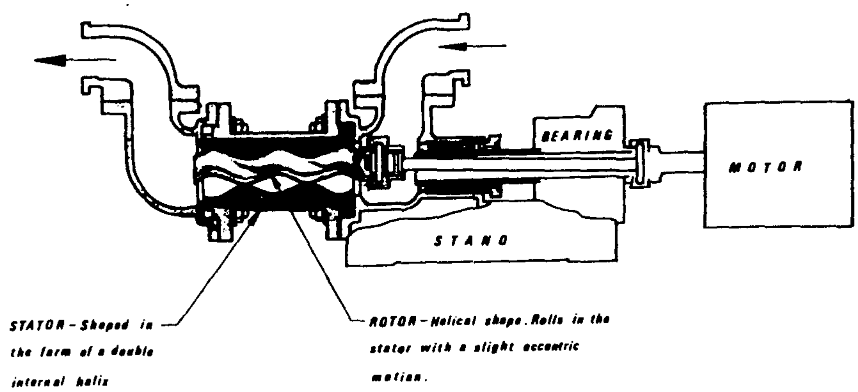


Figure 10. Helical rotor pump (Houlsby, 1990).

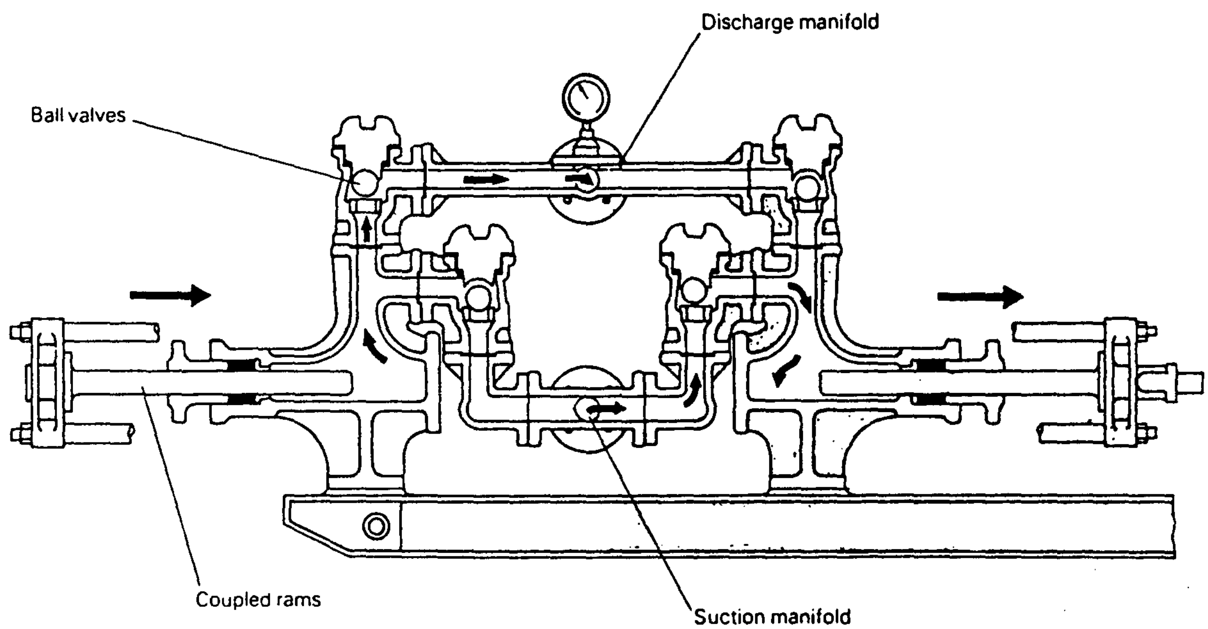


Figure 11. "Evans" ram pump (Gourlay and Carson, 1982; Houlsby, 1990).

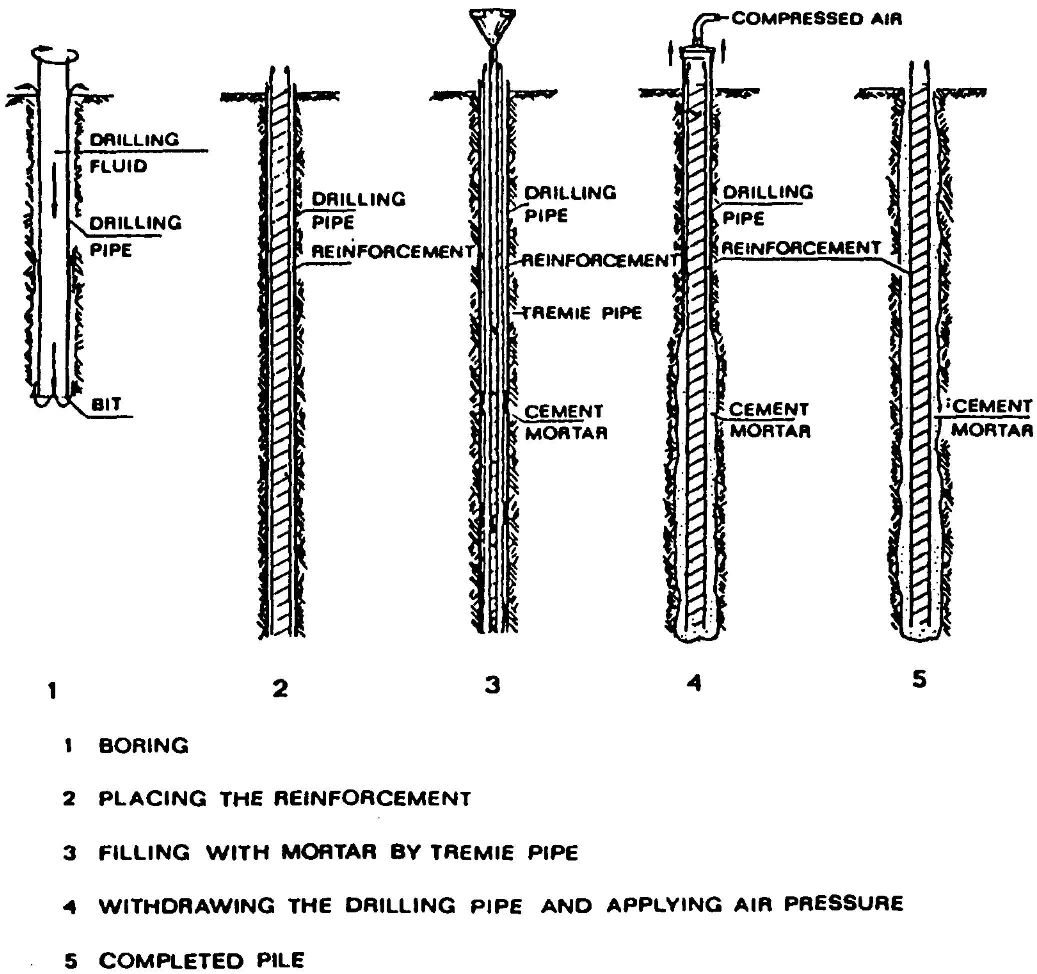


Figure 12. Construction phases of an early root pile (Mascardi, 1982).

<u>Pressure Grouting Through the Casing</u> (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 subsequent grout/soil 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. In the early days, this pressurization of the grout was achieved by applying compressed air through the grout line (figure 12), since contemporary drill head details and grout pump technology could not accommodate these relatively viscous, sand-cement mortars. This method has now been rendered obsolete by the developments in grout pump capabilities, combined with the trend to use stable, neat cement grouts without sand.

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), then it is common to find a pressure gauge mounted on the cap itself. Contractors acknowledge that there will be line losses in the system, but typically record the pressure as indicated on the pressure gauge without correction, reasoning that such losses are compensated by the extra pressure exerted by the grout column due to its self weight in the borehole.

American practice is to inject additional grout at a maximum pressure of between 0.5 and 1 MPa, with the aim of reinstating lateral soil pressures that may have been reduced by the drilling process, and achieving some degree of permeation into coarser grained granular soils or fractured rocks. The maximum injection pressures are dictated by the following factors:

- The need to avoid soil hydrofracture, 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 soil to form a "seal" around the casing during its extraction and pressure grouting.
- The "groutability" of the soil.
- The required grout/soil bond capacity.

Similar practices are found in the United Kingdom, although in Continental Europe, and France in particular, higher pressures tend to be used.

The injection of grout under pressure is aimed at improving grout/soil bond, and 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. This is discussed in detail in volume II.

When pressure grouting in granular soils, a certain amount of permeation and replacement of loosened soils takes place. In addition, 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. It is thus quicker setting and has higher strength. It also causes the formation of a cake-like cement paste along the grout/soil interface that improves bond. In cohesive soils, some lateral displacement, compaction, or localized improvement of the soil can occur around the bond zone, although the effect is generally less well marked than for cohesionless soils.

Pressure grouting also appears to cause a recompaction or redensification 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/movement performance. Such pressure grouting may also improve mechanically the soil between piles. This is an interesting concept within the CASE 2 pile application, but is as yet untested.

Post-Grouting (Types C and D Micropiles)

It may not be possible to exert excess grout pressures during the casing removal stage. For example, there may be leakage around the casing, or uncontrolled hydrofracture or "lensing" into horizontal soil structures. 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 ease of pumpability) and may have higher water content than the primary grout for the same reason, being in the range of 0.50 to 0.75 by weight. 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).

As described in the following paragraphs, high post-grouting pressures are typically applied locally for very restricted periods. For example, it may only take a few minutes to inject grout at a particular horizon. Herbst (1994) noted that the required aim of providing higher grout/soil bond capacity may, in fact, be more efficiently achieved in Type B micropiles, where grouting pressures are lower, but are exerted over a larger area and a much longer period. This has yet to be evaluated.

The construction-based classification of volume I identified two types of post-grouted piles, namely Type C and Type D.

- Type C - Neat cement grout is placed in the hole as for Type A. Between 15 and 25 minutes later, 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 suitably perforated reinforcement) at a pressure of at least 1 MPa. This type of pile is referred to in France as a Type III or Injection Globale et Unitaire (IGU) pile.

- Type D - Neat cement grout is placed in the hole as for Type A. Some hours later, 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 first time. There is usually an interval of at least 24 h between successive phases. This facility is reflected in the French term "Injection Répétitive et Sélective (IRS)" for these piles, which are also referred to as Type IV. Three or four phases of injection are not uncommon, contributing additional grout volumes as much as 50 percent of the primary volume.

Variations on the technique are found. The post-grout tube can be a separate 25-or 38-mm-diameter sleeved plastic pipe (tube à manchettes) placed together with the steel reinforcement (figure 13), or it can be the reinforcement tube itself, suitably sleeved (figure 14). In each of these cases, a double packer is used to grout through the tubes from the bottom sleeve upwards.

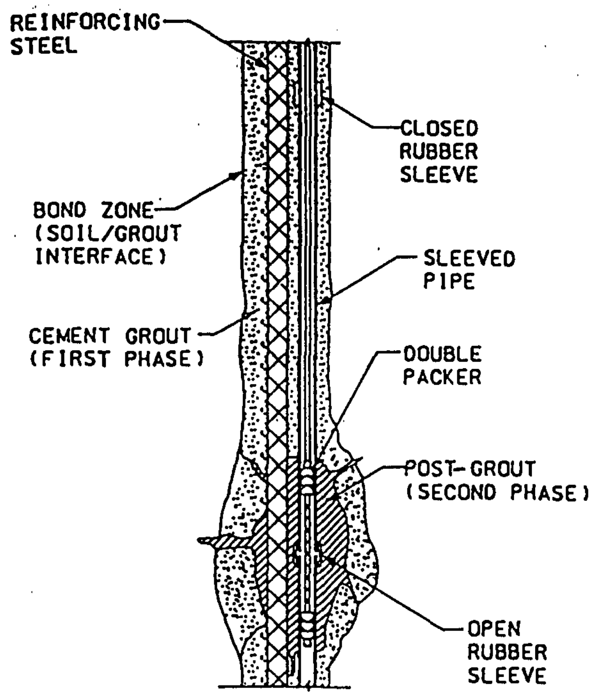


Figure 13. Principle of the tube à manchette method of post-grouting injection (schematic).

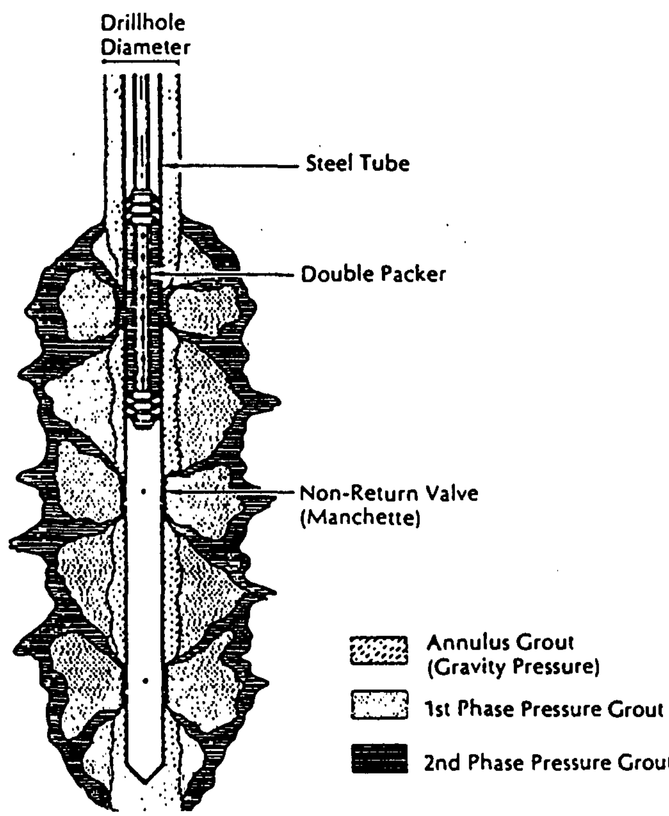


Figure 14. Use of reinforcement tube as a tube à manchette post-grouting system (schematic) [Mascardi, 1982].

Alternatively, this pressure grouting can be conducted from the surface via a circulating loop arrangement (figure 15). Grout is pumped around the system and the pressure is increased steadily by closing the pressurization valve on the outlet side. At the critical "break-out" pressure, dictated by the lateral resistance provided by the adjacent ground, the grout begins to flow out of the tube through one or more sleeves and enters the ground at that horizon. When using the loop method, it is assumed that with each successive phase of injection, different sleeves open, ultimately ensuring treatment over the entire sleeved length (a feature guaranteed by the tube à manchette method using double packers).

A slight cautionary note against the use of post-grouting was raised by Lizzi (1994) regarding leaving "foreign materials" (i.e., the post-grouting tubes) in the pile and contractors being tempted to use low-strength primary grouts to reduce "break-out" pressures. However, this appears not to be an issue for practical concern in good practice, bearing in mind always that most (and occasionally all) of the applied load is designed to be accepted by the steel reinforcement, and not the surrounding grout.

Top-Off (Secondary) Grouting

Due to slow grout seepage, bleed, or shrinkage, it is common to find that the grout level drops a little during the stiffening and hardening phases. In ground anchorage practice, this is simply rectified by topping off the hole with the lowest water-content grout practical, at some later time. However, in micropile practice, such a cold joint should be avoided since the grout column should be continuous for load transfer and corrosion protection reasons. Topping off is therefore best conducted during the stiffening phase to ensure integrity. Where particularly high interfacial bond stresses must be resisted between the pile and an existing structure, proprietary high-strength, non-shrink grouts may be considered.

Reinforcement

Types of Reinforcement

The reinforcement may consist of a cage of reinforcing bars (either standard or high strength), a single high-strength bar (or group of bars), or a steel pipe. When using a steel pipe reinforcement, this may be formed from the drill casing itself or from a tube of smaller diameter placed within the temporary casing.

For low-headroom applications, it is often not possible or practical to place the reinforcement in a single length or in lapped lengths. In such cases, the use of threaded and coupled reinforcing bars is a simple and effective measure. A common choice throughout the world is the GEWI bar (figure 16 and table 2). These bars have a characteristic yield stress of approximately 420 or 525 MPa, conforming to ASTM A615 and A706, and BS4449 Deformed High-Yield Steel, as examples. The bar has a coarse pitch, continuous ribbed thread rolled on during production. This ensures good grout/steel bond, but in addition, the bar can be cut at any point for joining with couplers to restore full reinforcement strength in both compression and tension. Coupled bars permit the continuous full-length reinforcement of long micropiles in even low-headroom conditions. The manufacturer also claims that the bar contributes a "large ductility to the pile, which is required to compensate shear deformations and seismic loadings" (figure 17).

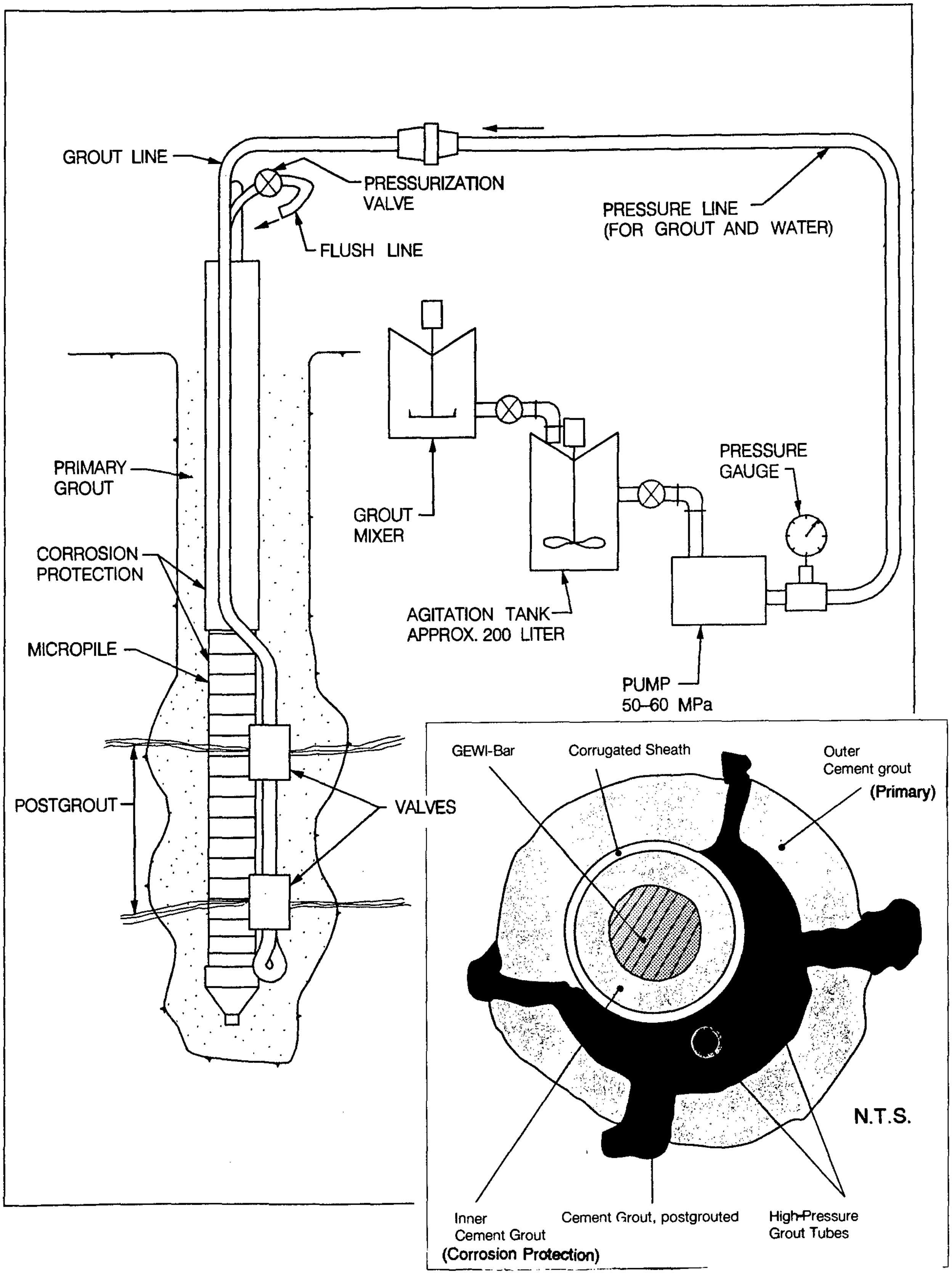


Figure 15. Elevation (schematic) and cross section of "loop-type" post-grouting system (DSI, 1992).

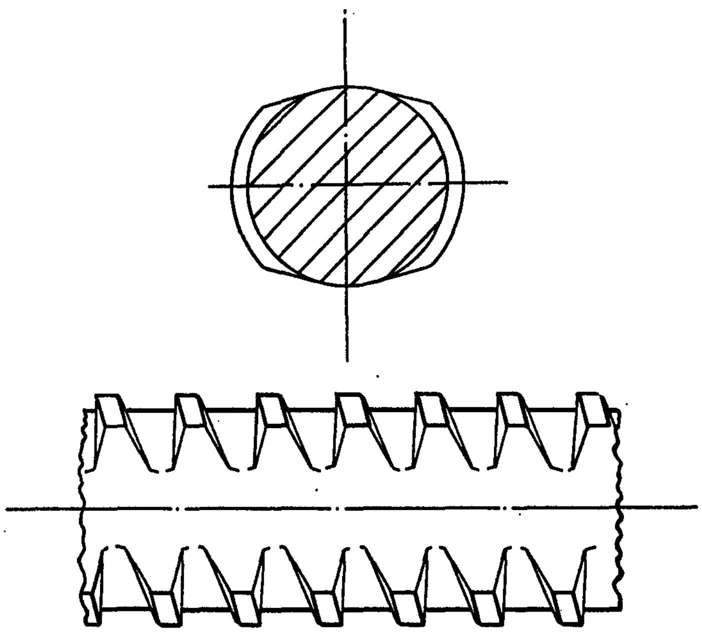
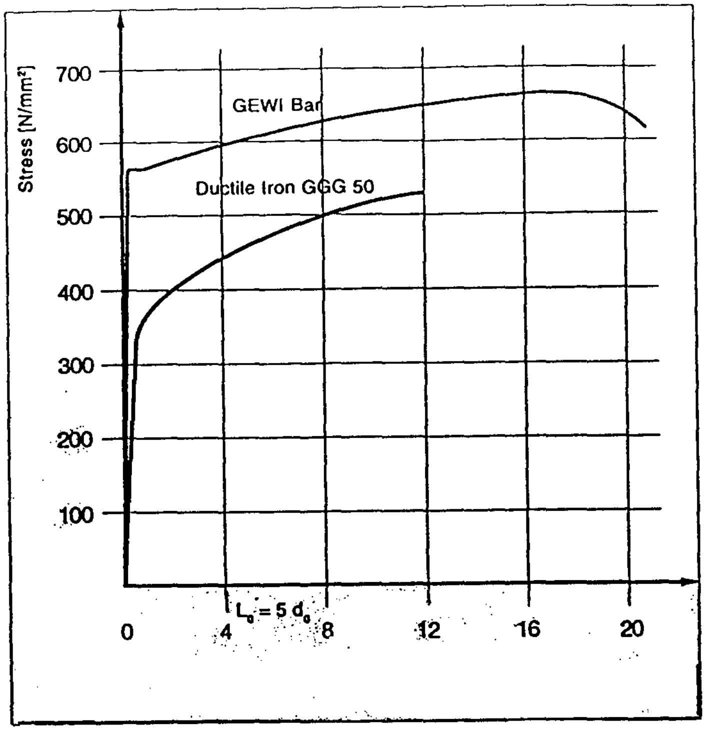


Figure 16. Details of continuously threaded GEWI bar (DSI, 1992).

Table 2. Details of GEWI bars (DSI, 1992).

GEWI Pile, steel properties													
Steel grade	Bar dia.		Cross-sectional area A_{PS}		Ultimate load $A_{PS} \cdot f_u$		Yield load $A_{PS} \cdot f_Y$		Nominal weight		Maximum threadbar dia.	
	No.	mm	in²	mm²	kips	kN	kips	kN	lbs / ft	kg / m	in	mm
60	# 14	43	2,25	1452	202,5	901	135	601	7,65	11,38	1,86	47,2
60	# 18	57	4,00	2581	360	1601	240	1068	13,60	20,24	2,50	63,5
75	# 20	63,5	4,91	3167	491	2184	368	1637	16,71	24,86	2,72	69,0
60	3 x # 14	3 x 43	6,75	4356	607,5	2702	405	1802	22,95	34,14	-	-
60	3 x # 18	3 x 57	12,00	7743	1080	4804	720	3203	40,80	60,72	-	-
75	3 x # 20	3 x 63,5	14,73	9501	1473	6552	1104	4911	50,13	74,58	-	-

Note: Grade 75 #14 and #18 are also available;
all combinations of up to three GEWI bars are possible.



Notes: (1) Grade 75 #14 and #18 are also available.
(2) All combinations of up to three GEWI bars are possible.

Figure 17. Stress-strain performance of GEWI bar (DSI, 1992).

In certain high-capacity pile types, such as the "composite" Type A3 or B3, more than one type of reinforcement may be needed (figure 18) to satisfy design requirements or construction restraints.

Corrosion Protection

Traditionally, and except for permanent tension piles in aggressive ground conditions, little protection other than the surrounding cement grout has been provided to the reinforcement in most countries. The design of corrosion protection is detailed in volume II, although the principles can be reviewed at this point. Attention to this important detail is growing, in line with the use of higher capacities in aggressive ground conditions.

Protective barriers can be applied to the reinforcement (e.g., epoxy coating) or around bar reinforcement (e.g., corrugated sheathing, (figure 19) prior to placing in the pile hole). Minimum thicknesses of grout cover (e.g., 20 mm, increasing to 30 mm in "strongly aggressive" conditions, in Germany) are then specified to prevent groundwater from reaching the steel, the assumption being that microfissures are unlikely to develop through grout in service, compressive conditions.

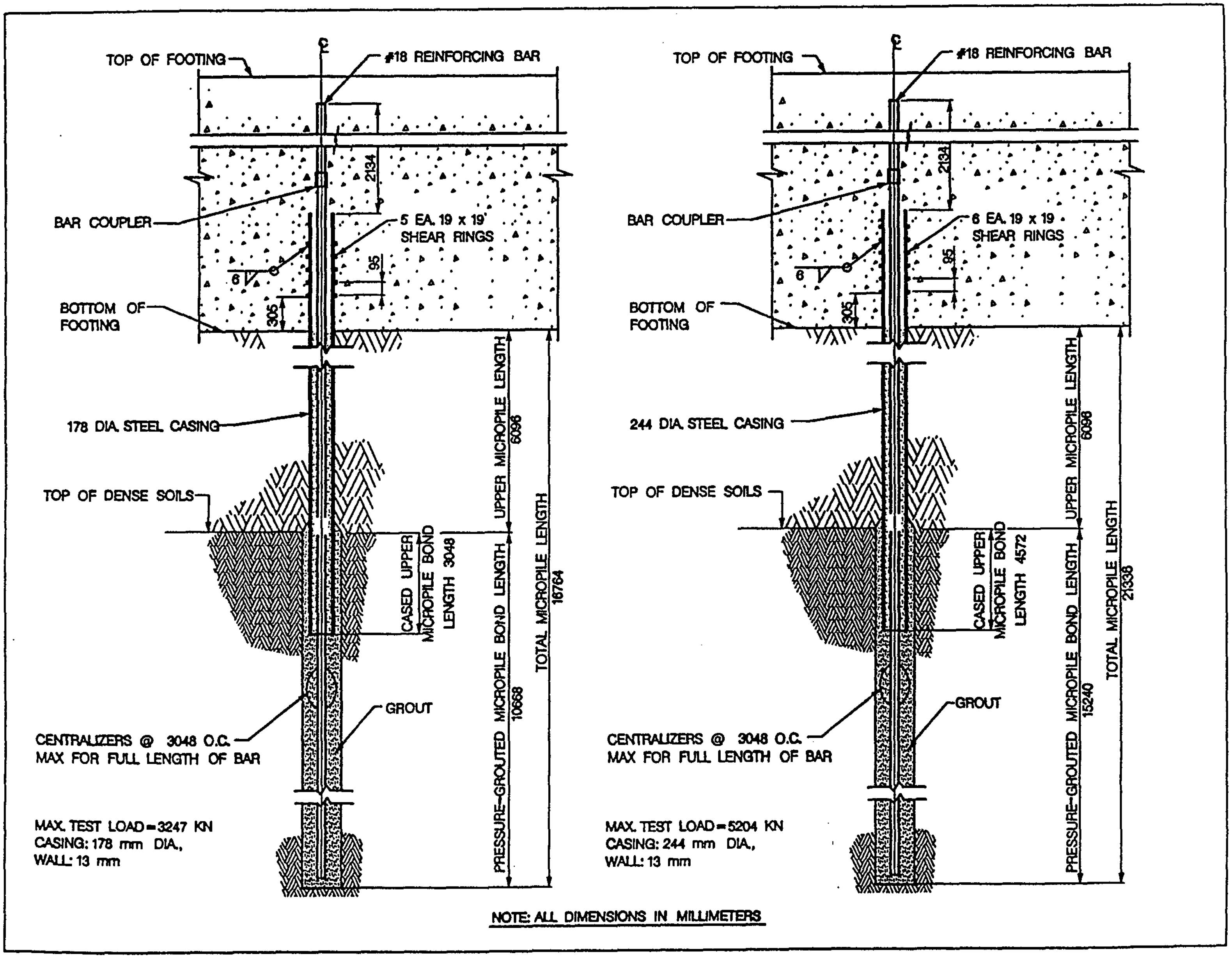


Figure 18. Details of composite high-capacity Type 1B micropiles, Vandenberg Air Force Base, California.

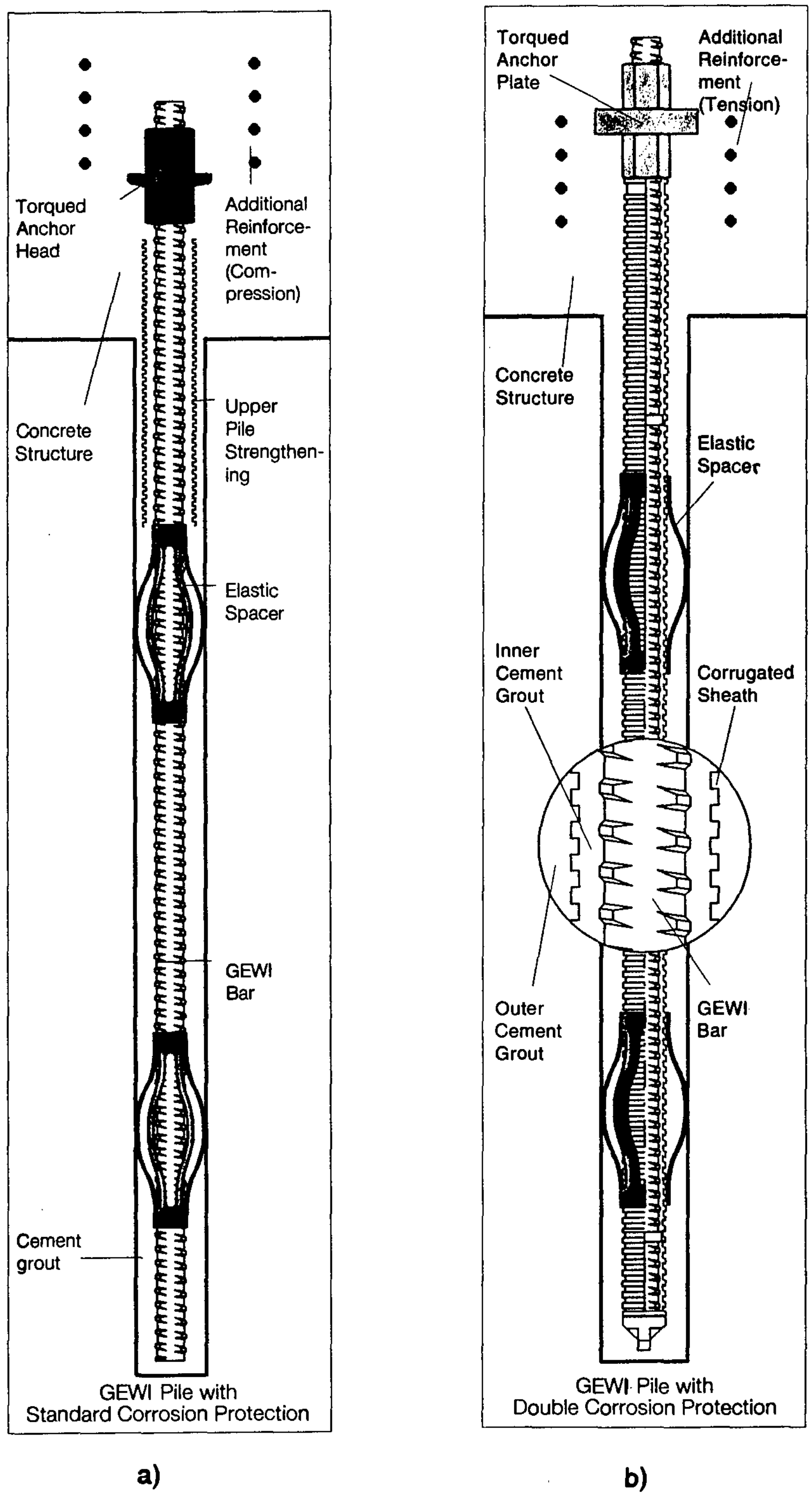


Figure 19. GEWI piles with (a) standard corrosion protection and (b) double corrosion protection (DSI, 1992).

Other philosophies prevail where additional protection is not applied, where a continuous grout cover of adequate thickness cannot be guaranteed, or where the reinforcement acts in tension. Large-diameter reinforcing bars have a surface area that is small in comparison with the cross-sectional area. Suppliers refer to this as a geometric corrosion protection, which relates to the concept that a progressive loss of section with time is allowable, and typical rates are widely quoted (e.g., Fleming et al., 1985).

Spacing/Centralizing of Reinforcement

The individual elements of a group reinforcement are separated by the use of spacers, or ties to the helical reinforcement in order to ensure that grout flows between the bars so that uniform contact is ensured with the hardened cement grout. In addition, centralizers are commonly attached to the reinforcement to ensure adequate cover of grout between the bars and the sides of the borehole for corrosion protection purposes.

Typical centralizers used with mono- and multi-bar systems are illustrated in figures 19 and 20.

Where the drill casing is used as an external tube reinforcement, allowance may be made in the design thickness of the steel wall for loss of section due to corrosion.

Removal of Drill Casing

Drill casings are removed by either pulling with the drilling head while slowly being rotated, or by independent jacking systems that straddle the casing and bear on the ground surface or piling platform. The choice reflects the capabilities of the equipment available, the type of grouting to be conducted, and the usual general considerations of cost, effectiveness, and site logistics.

Connection to the Structure

The connection of CASE 1 micropiles to the structure that is to be directly supported is an important detail. This is particularly significant in seismic applications where the connection may have to be designed to take movement during seismic events.

Where piles have been installed through existing structures, the normally smooth borehole wall, typically formed by coring, can be grooved and roughened to provide additional mechanical interlock and to ensure adequate bond. One such proprietary system, known as Ankerbonder, was used successfully on a major underpinning project for cooling towers in England (Anonymous, 1987), while other non-proprietary options also exist, based on the same principle of "roughening up" the interface to provide enhanced mechanical bond.

In another approach, horizontal post-tensioning of the foundation beam cast around the pile heads has also been used to achieve a clamping effect to guarantee structure/pile continuity. As noted above, it is also common to find the upper pile section grouted within the structure using high-strength, non-shrink grout. Such grouts, with strengths often two times greater than

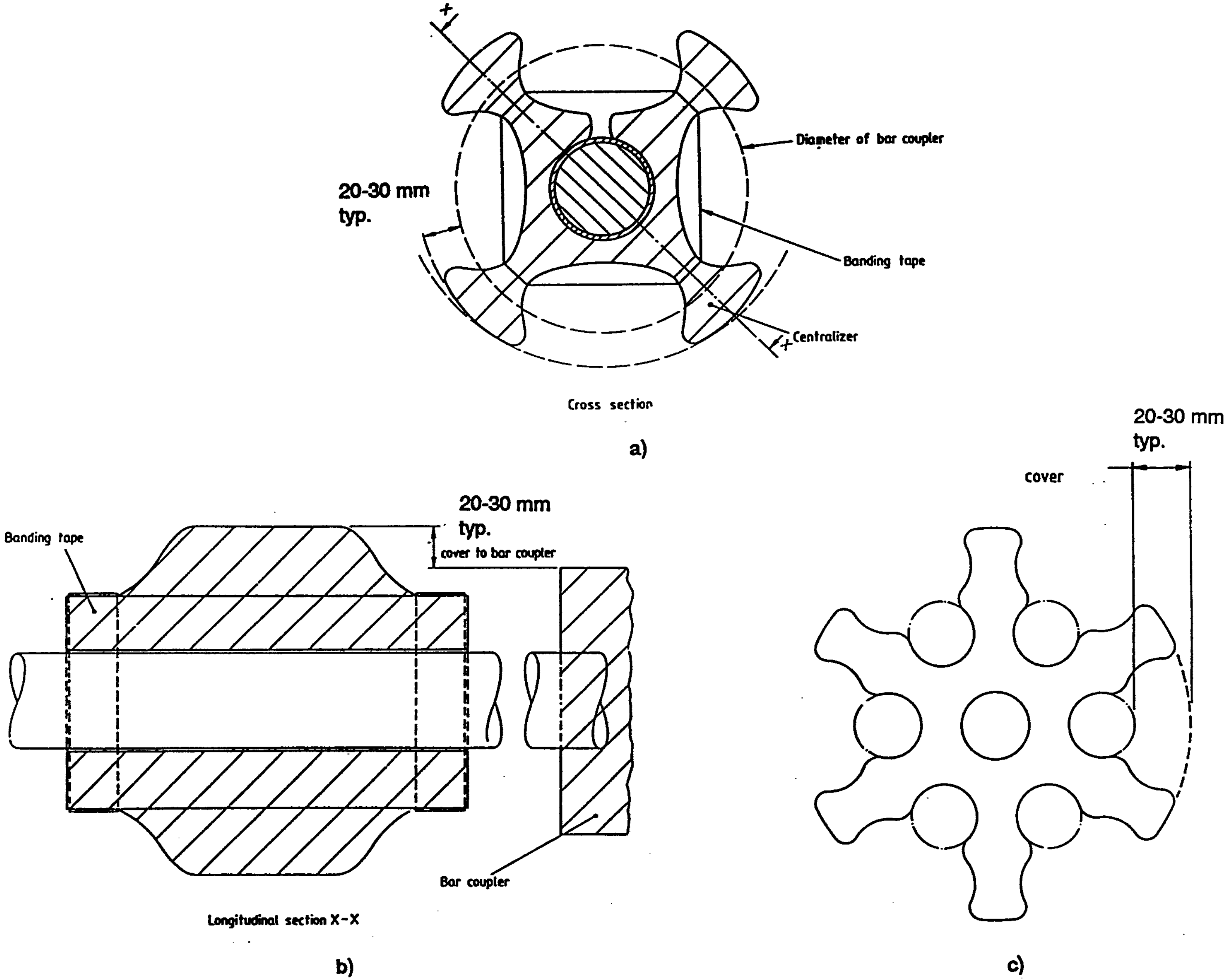


Figure 20. Typical spacer/centralizer units used for bar-reinforced micropiles.

normal, are able to resist the high stresses that may be associated with this region, where loads may have to be transferred over the short distances that reflect the thickness of the original footing.

Where permanent steel tubes are used as the reinforcing member, bond between the foundation grout or concrete and the tubes can be provided by the use of shear rings or keys or studs welded to the steel tubes (figures 21 and 22). For bar reinforcements in new footings, these may be equipped with an anchor nut, or may be expected to transfer load solely by bond in footings of higher strength (figure 23).

For new works, similar considerations apply to the design of pile caps for micropiles as for the design of conventional piles. The small cross-sectional area of micropiles in conjunction with their higher axial load may mean that design against punching, shear, and bursting failure has to be addressed in particular detail in the pile cap design.

Preloading of CASE 1 Piles

Micropiles, constructed as described above, provide excellent load/movement performance, often with less than 10 mm of total movement at service load (Bruce, 1989a). In certain cases, however, movements of even this magnitude are unacceptable structurally. Preloading techniques can then be used with certain types of CASE 1 piles. The service load is preapplied to compress the pile and induce compression and movement *prior* to its connection to the structure. The frictional resistance of the pile is thus "activated," without needing further, future settlement of the structure. Preloading also has potential in seismic retrofit applications, since it makes the pile more resistant to "rocking" motions induced on the foundations of tall structures such as bridges with tall columns.

Preloading can be accomplished in several ways. In one system (**ROPRESS**), the pile is preloaded by a hydraulic jack reacting on an anchor pipe and locked-off against the pipe when the desired load/movement criterion is achieved. The pile head is then grouted to the structure, and the preload is released (figure 24). A similar system can be used for thread-bar reinforcement (figure 25). Conversely, the Nicholson Compressed Anchor (NCA) pile provides a similar stiff prestressed pile/anchor system that acts equally efficiently in tension and compression (Bruce and Gemme, 1992). Here preloading is achieved through a strand tendon founded below the tip of the pile (figure 26), as in the case of the Pocomoke River Bridge (Bruce et al., 1990). After the grout has reached a certain target strength, the tendon is stressed against the steel casing up to the service load. The annulus between the casing and structure is then grouted with special high-strength grout. Several days later, the prestress is released, thereby allowing full structural load transfer to the pile, but without causing further pile head movement.

An additional benefit of preloading is that this operation routinely tests each pile to at least its working load. For preloading to be practical, the pile must not be fully bonded over its entire length; rather, it must have a "free length" (capable of being compressed) and a lower "bond length." Of course, since fully bonded piles exhibit less movement (volume I), then preloading is not needed anyway.

Figure 21. Welding shear studs onto micropile casing prior to incorporation into new reinforced concrete ground beam.

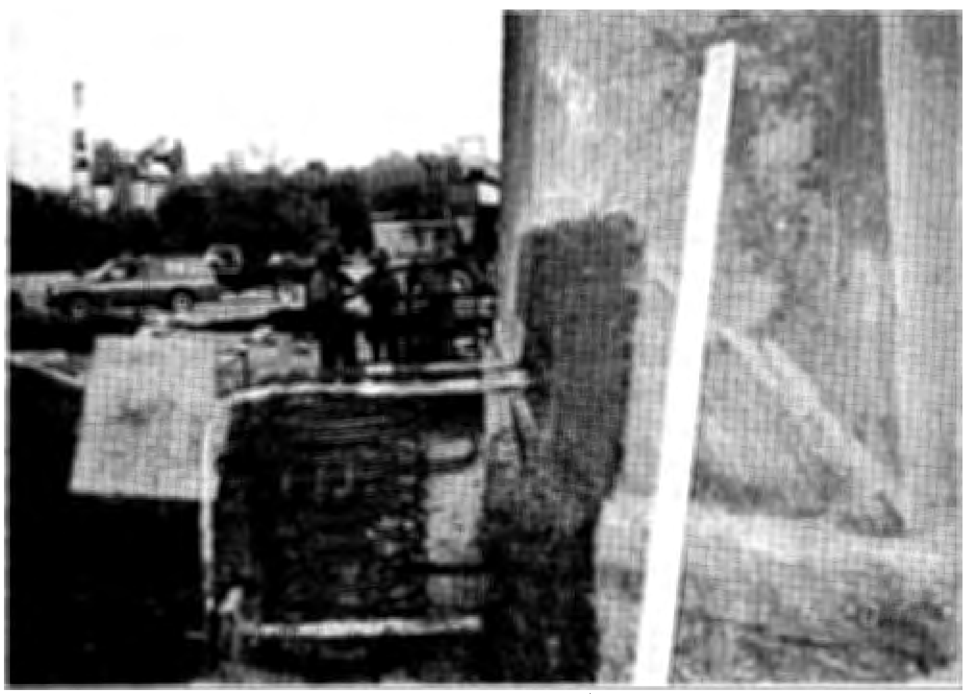
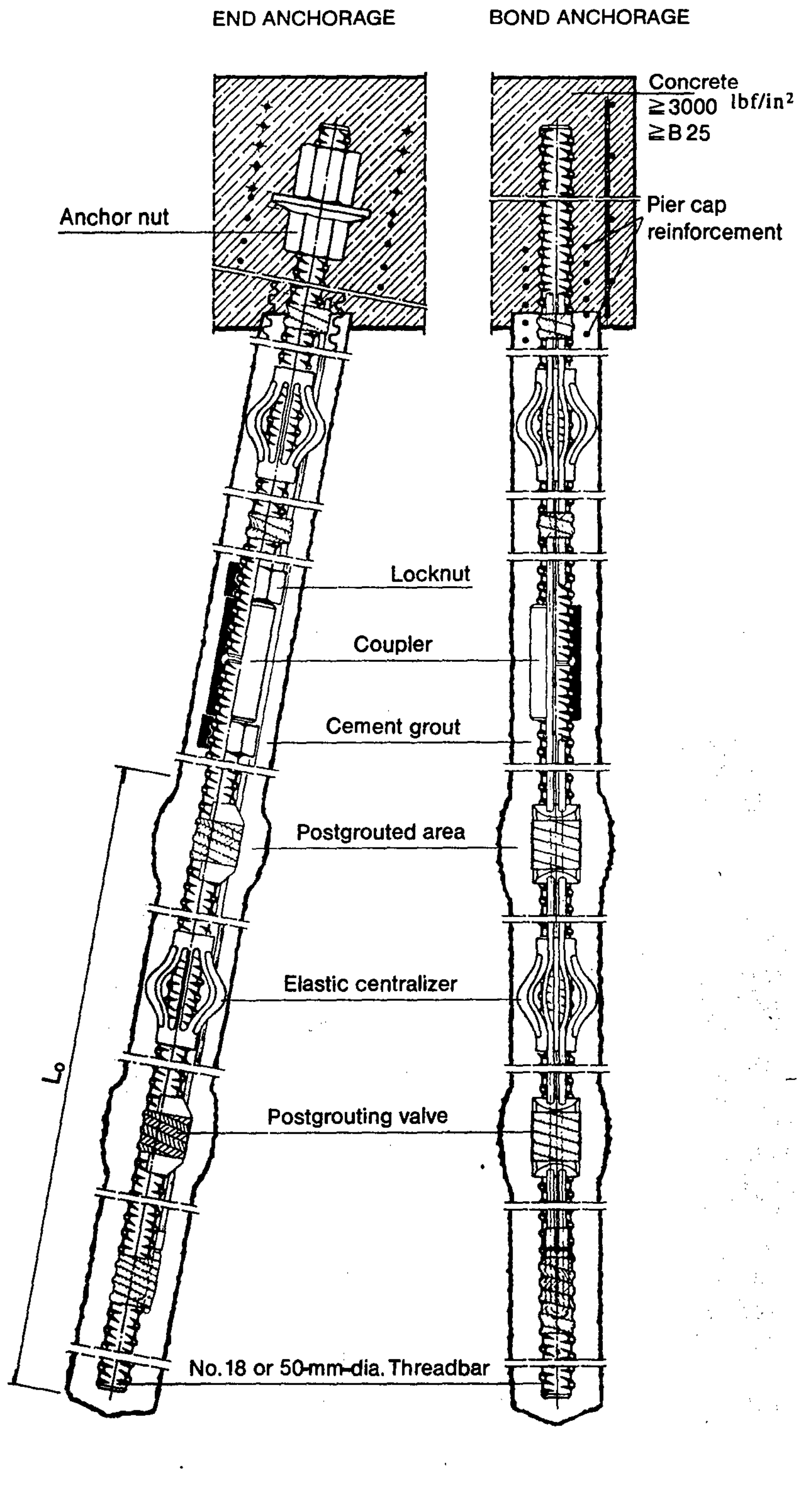


Figure 22. Reinforcement placed around micropile head prior to pouring of concrete ground beam.



1 lbf/in^2 = 6.9 kPa

Figure 23. End and bond anchorage bar micropiles (DSI, 1992).

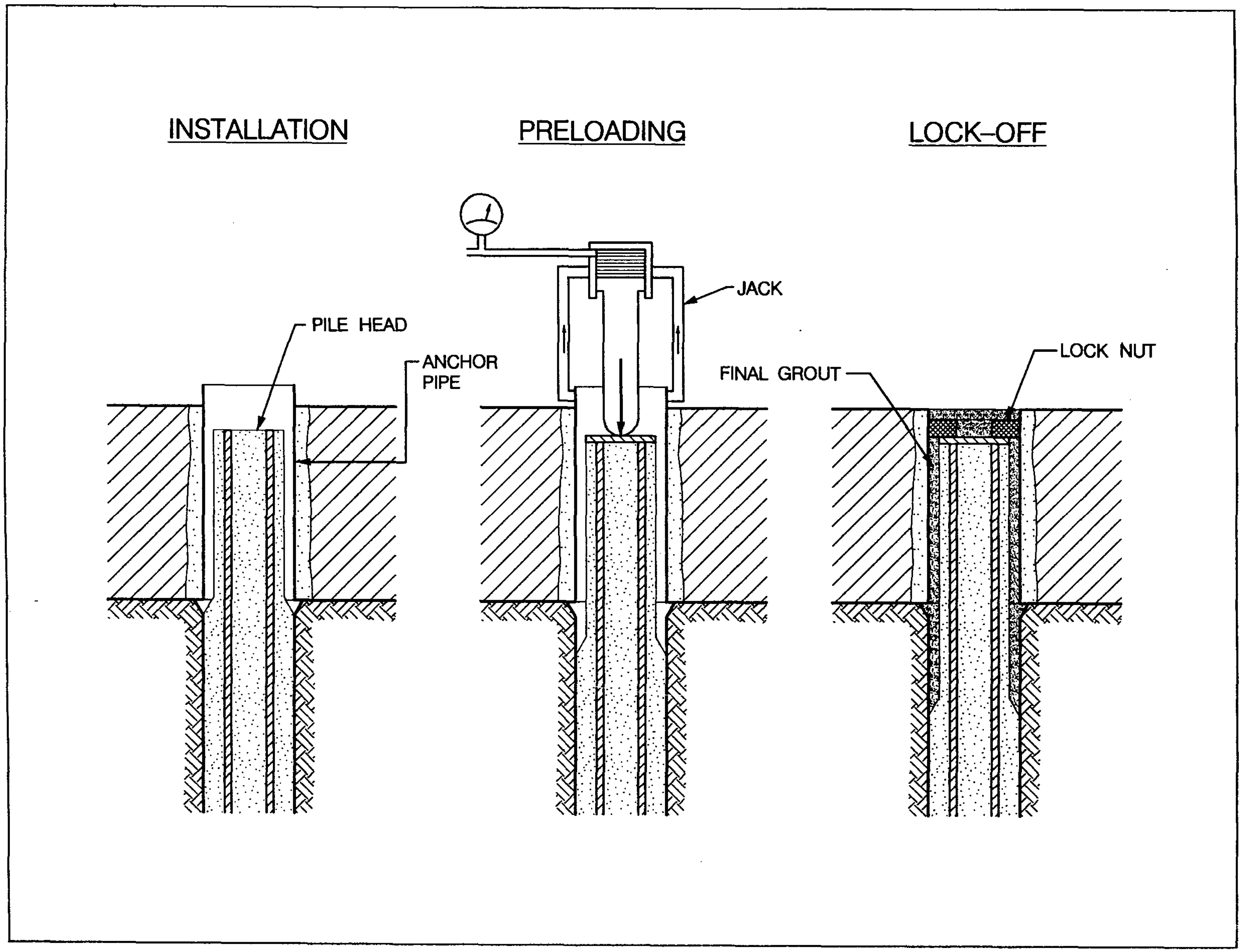


Figure 24. Details of preloading sequence, ROPRESS micropile (Rodio, 1994).

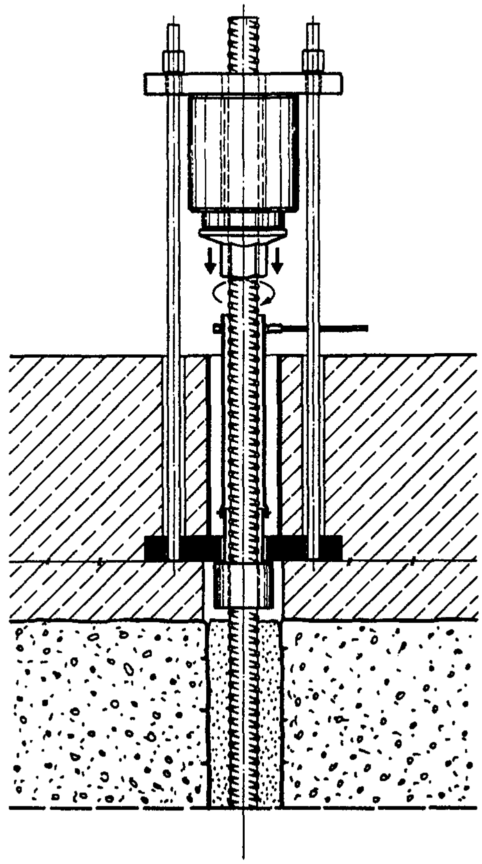
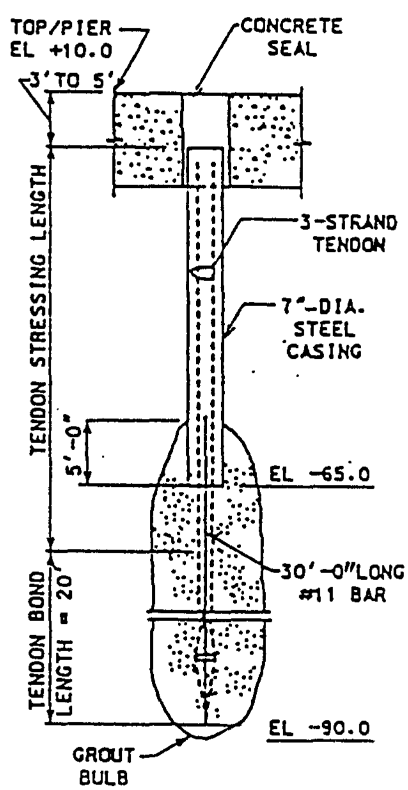


Figure 25. Preloading a GEWI pile against a foundation (DSI, 1992).



1 in = 25.4 mm
1 ft = 0.305 m

Figure 26. Concept of preloading via strand anchorage founded below the micropile toe (Bruce et al., 1990).

As a cautionary note, Lizzi (1982) recommended that preloading, when the preload is induced via reaction against the structure itself, should be "decisively rejected...for the following reasons:

1) The preload introduces, into the soil and the building, stresses that constitute a striking disturbance to the existing stability, the consequences of which could also be very grave.

2) The safety factor of an underpinning such as that, is no longer the safety factor of the existing foundation plus the safety factor of the piling, but on the contrary, only that of the piling alone; that is, it has to rely only on the capacity of the piles, without the essential contribution that the existing subsoil could supply.

3) The building is transferred onto the piling, losing its contact with the soil; this [soil], freed from the building load, loses, sooner or later, its high degree of consolidation reached, sometimes, after centuries. Should the building settle again, pressing anew on the soil, it would be found to have characteristics much worse than those it had before being imprudently detached.

4) The connection between the piles and the structure has to be postponed until the time when the piling is complete, at least in some parts, and in a condition of being subjected to the preload. Instead of a progressive improvement of the piling as in the case of a normal pali radice, a long state of crisis would occur that could terminate only with the end of the piling and its connection to the structures above ground level.

What could happen in the meantime to the building? Very probably, shoring or other supporting structures would be necessary, with technical and economic inconveniences."

Clearly, preloading is not an option in fully bonded CASE 1 piles or in any type of CASE 2 piles. In addition, its practicality must also consider the ability of the structure to accept the short-term stresses applied. Nevertheless, preloading is an extremely useful facility and a powerful option, and has wide use in appropriate applications.

OUTLINE OF TYPICAL PROPRIETARY SYSTEMS AND NATIONAL TRENDS

The wide range of techniques available for drilling, placing reinforcement, and grouting means that many proprietary systems have been developed. These reflect practices within a company, a region, or even a country, where national codes and building regulations may favor developments along a particular path. The following outlines cover a range of micropiling techniques that illustrate the breadth of typical common practices.

United States Practice

From the early applications of low- to medium-capacity Type A and Type B piles, the national trend has been towards providing higher unit pile capacities, especially in compression. This has led to the development of the Nicholson Pin pile and its different variations. Numerous other contractors now provide basically similar options, and practices across the country are fairly similar. This is a result of a strong technical literature base, and the growing confidence and ability of consultants to design micropile systems, rather than rely on contractors to provide proprietary solutions. As described in volume I, however, even those specifications tend to be performance-based rather than prescriptive, and so there still remains scope and initiative for innovation and development. This is particularly evident in the approach to seismic applications.

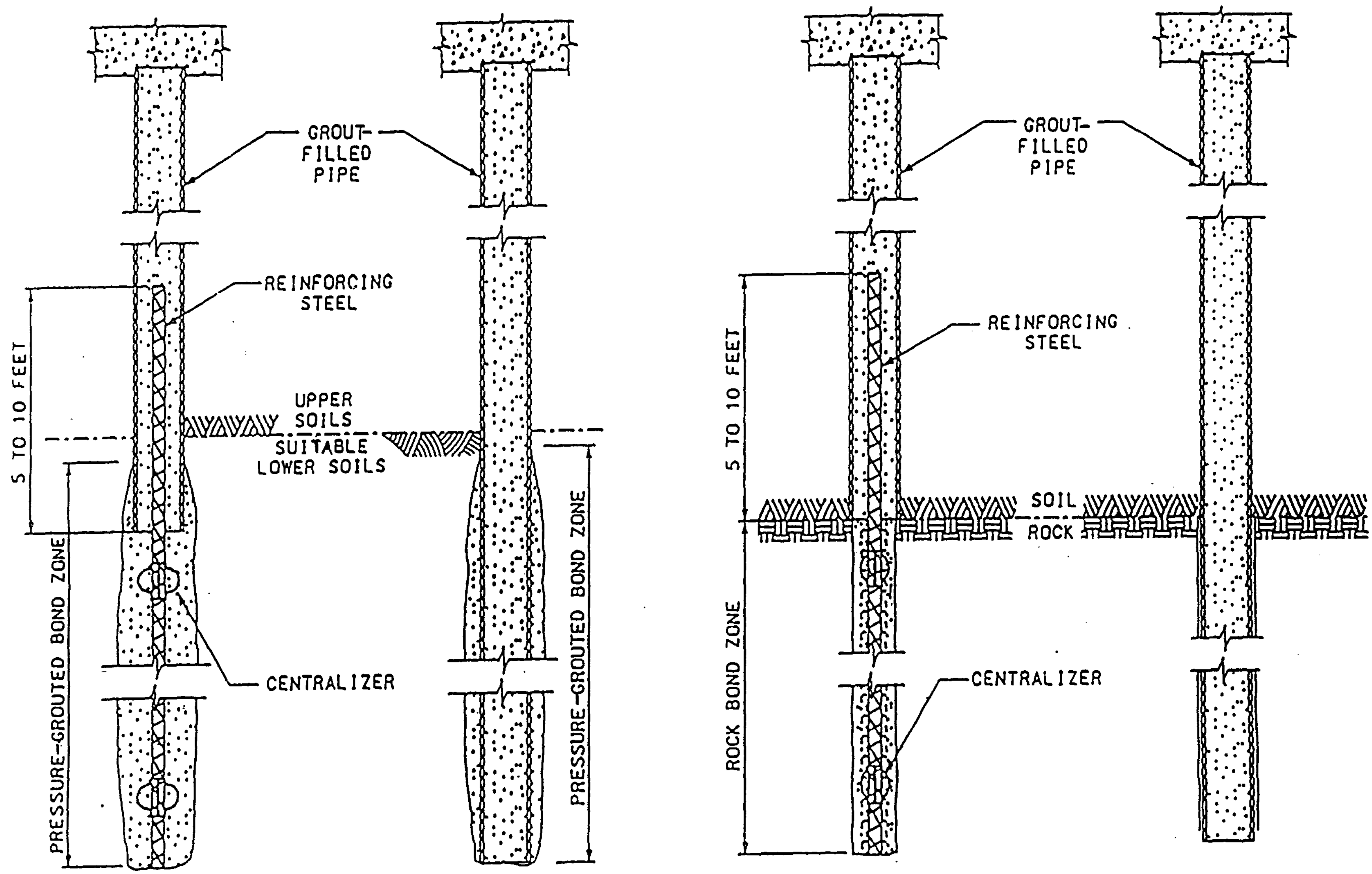
All current American applications are of the CASE 1 type, although a limited number of CASE 2 structures, for slope stabilization, were installed in the 1970's. Types A and B are most common, although Type D piles are becoming more popular, especially on the East Coast. Preloading, via the central tendon method, is required infrequently, although it has proved to be very successful when actually adopted (Bruce, 1988, 1989).

Pearlman and Wolosick (1992) proposed four micropile types, based on the configuration of the reinforcement and the geology of the founding stratum. These are illustrated in figure 27 and, as shown in table 1 of volume I, are readily assimilated into the construction classification used in this report.

Type S1 - This type of configuration is used when the pile is founded in soil and it is the most common type of micropile installed by contractors in the United States.

- Drilling. A steel casing is rotated into the soil, most commonly using duplex drilling methods with water or foam flush. Single-tube advancement is preferred whenever soil and environmental conditions permit. Air flushing is typically avoided due to the potential for soil and structural disturbance. Contemporary applications utilize track-mounted, diesel- hydraulic or electro-hydraulic drill rigs with high-torque rotary heads. In low-headroom conditions, modular, wheel- or skid-mounted rigs are used. Pile diameters are typically between 120 and 300 mm, with the majority less than 200 mm.

- Grouting/Reinforcement. Once the casing has reached the design depth, neat cement grout is tremied from the bottom of the hole to displace the drilling fluid. This and other configurations typically utilize a neat cement grout with a water/cement (w/c) ratio of 0.45 to 0.50. The reinforcing element, appropriately centralized, is then placed to the bottom of the hole. Such micropiles are constructed most often with a single, high-strength bar of yield strength f_y = 415 or 517 MPa or low-steel alloy pipe (f_y = 380 MPa) as the primary reinforcing element. Many of the high-capacity piles also utilize N80 casing (f_y = 551 MPa) in conjunction with a centralized bar or pipe. Cages or groups of bars may also be used. Details of bars and casings used in American practice are shown in tables 3 and 4, respectively. Additional grout is then pumped under significant excess pressure, typically between 0.4 and 1 MPa (Type B), while the casing is simultaneously withdrawn. This process establishes the pile bond zone. Certain very-high-capacity piles employ a composite reinforcement system, consisting of a centralized reinforcing bar in the bond zone and a casing in the upper part. Once the full length of the centrally reinforced bond zone has been formed, the casing is then plunged back down 1.5 to 3 m into this pressurized grout for permanent seating. For lower capacity elements, the temporary drill casing is usually fully extracted.



PIN PILE TYPE S1
(Types B or D)

PIN PILE TYPE S2
(Type B)

PIN PILE TYPE R1
(Type A)

PIN PILE TYPE R2
(Type A)

Figure 27. Micropile types in soil and rock
(Pearlman and Wolosick, 1992).
1 ft = 0.305 m

Table 3. Axial tension and compression loads for ASTM A615 and ASTM A706 reinforcing bars.

Grade	Grade 60						Grade 75		
Rebar No.	8	9	10	11	14	18	11	14	18
Length in	1.00	1.13	1.27	1.41	1.69	2.26	1.41	1.69	2.26
mm	25	29	32	35	43	57	35	43	57
Area in^2	0.79	1.00	1.27	1.56	2.25	4.00	1.56	2.25	4.00
mm^2	510	645	819	1006	1452	2580	1006	1452	2580
Yield Load kips	47	60	76	94	135	240	117	169	300
kN	211	267	339	416	600	1068	520	751	1334
Ult. Load kips	71	90	114	140	203	360	164	236	420
kN	316	400	508	624	901	1601	729	1051	1868

BAR NO.	APPROXIMATE DIAMETER (mm)	BAR PROPERTIES DIAMETER (inches)	AREA ($inches^2$)	CONVERTED AREA (mm^2)
2	6.4	0.250	0.05	32.255
3	9.5	0.375	0.11	70.961
4	12.8	0.500	0.2	129.02
5	16.0	0.625	0.31	199.981
6	19.0	0.750	0.44	283.844
7	22.2	0.875	0.6	387.06
8	25.5	1.000	0.79	509.629
9	28.7	1.128	1	645.1
10	32.3	1.270	1.27	819.277
11	35.8	1.410	1.56	1006.356
14	43.0	1.693	2.25	1451.475
18	57.3	2.257	4	2580.4

Notes:

(1) Certain dimensions are shown rounded off in the table. Specifically, bars #9, #14, and #18 have diameters of 1.128, 1.693, and 2.257 inches, respectively.

(2) Grade 60 reinforcing steel has a yield stress of f_y = 60 kips/in^2 (415 MPa) and a tensile strength of f_u = 90 kips/in^2 (620 MPa).

(3) Grade 75 reinforcing steel has a yield stress of f_y = 75 kips/in^2 (517 MPa) and a tensile strength of f_u = 105 kips/in^2 (723 MPa).

(4) Conversion data are: 1 inch = 25.4 mm; 1 in^2 = 64.5 mm^2; 1 kip/in^2 = 6.89 MPa; 1 kip = 4,448 kN

Grouts are typically neat water cement mixes. Type I/II cement is most common, although Type III is used for special occasions such as precontract test programs. At this point, post-grouting techniques may be used if enhanced grout/soil bond characteristics are needed. The tube-à-manchette system with double packers gives the best performance (Type D), although in many cases, an acceptable level of performance can be obtained by merely grouting from the top of the tube (without packers) and permitting the grout to exit at the sleeves of its choice.

Type S2 - This pile type also is designed for use in soils. Its installation is similar to that for Type S1, but with the following differences:

- The centralized reinforcing element is not needed, since the steel casing is reinserted to the full depth of the bond zone after pressure grouting.
- Post-grouting cannot be used. (Hence, these piles are Type B in the general classification.) Such piles are less common than Type S1, and are used only under special conditions, usually reflecting cost and logistical factors.

Table 4. Axial tension and compression loads for API N-80 steel casing.

Casing OD	in	5-1/2	7	9-5/8
	mm	139.7	177.8	244.5
Wall Thickness	in	0.361	0.498	0.472
	mm	9.17	12.65	11.99
Steel Area	in^2	5.83	10.17	13.57
	mm^2	3760	6563	8756
Yield Load	kips	466	814	1086
	kN	2075	3619	4829

Notes:

(1) Casing outside diameter (OD) and wall thickness (t) are nominal dimensions.

(2) Steel area is calculated as $A_s = \pi t (OD - t)$.

(3) Nominal yield stress for API N-80 steel is F_y = 80 kips/in^2 (551 MPa).

(4) Conversion data are: 1 inch = 25.4 mm; 1 in^2 = 64.5 mm^2; 1 kip/in^2 = 6.89 MPa; 1 kip = 4.448 kN.

Type R1 - This configuration is used when the pile is to be founded in rock and is the more common of the two rock pile options. It is also (rarely) used when founding low-capacity piles in hard or stiff cohesive soils.

- Drilling. The drill casing is advanced in the same manner as for the Type S1 pile, except that it terminates at the top of rock. After the casing is seated into rock, a drill string is then advanced through its center to drill the rock bond zone. For cohesives, augering is typically used, so as not to cause water flush softening of the borehole wall.

- Grouting/Reinforcement. Once drilling is complete, neat cement grout is then tremied from the bottom, and a reinforcing element is placed in the bond zone to complete installation. This makes it a Type A pile.

Type R2 - Also intended for rock formations, this type of micropile differs from Type R1 in that the steel casing extends the full length, as in Type S2.

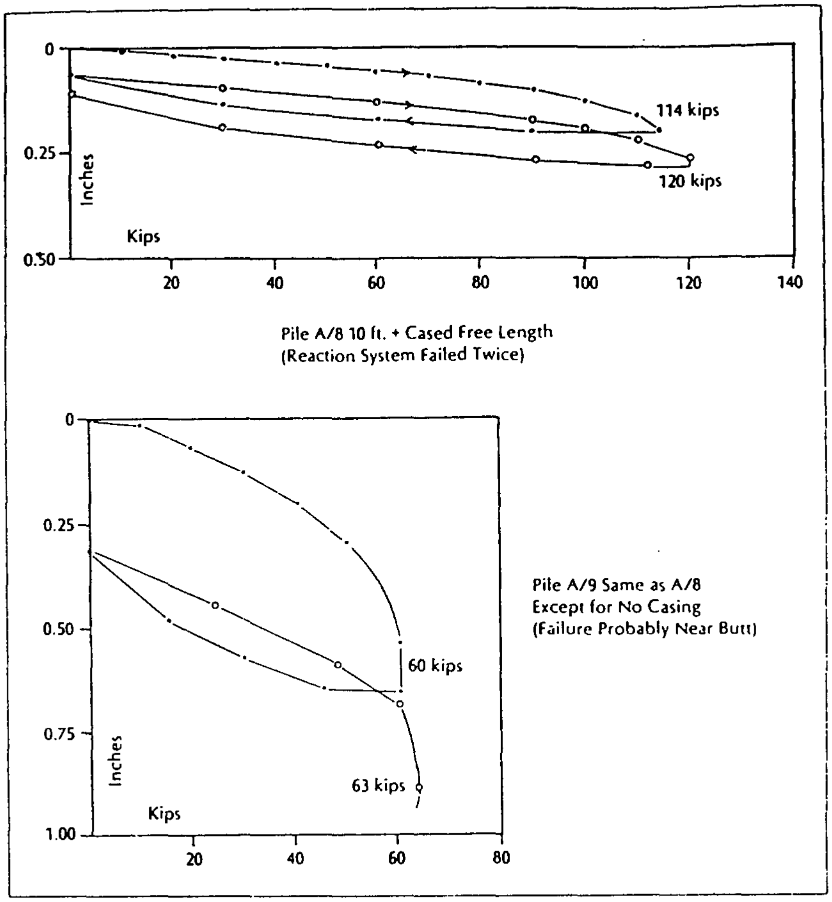
- Drilling. In order to penetrate both the overburden and the rock, a permanent drill bit that is attached to the end of the steel casing and that is greater in diameter is employed. This is also a Type A pile. Underreaming is not conducted in American practice.

- Removal of Drill Casing. As indicated above, a distinguishing characteristic of high-capacity micropiles is that the drill casing is usually left in place through the upper zones of the pile, whether or not it is used as the main reinforcing element. This contrasts with the practice of other countries, where the drill casing is often fully extracted. Testing has proved that both vertical and lateral micropile performance are enhanced when the casing is left in place above the pressure-grouted zone (figure 28). This practice also prevents grout loss into the often permeable upper strata and adds an extra measure of corrosion protection to any full-length internal reinforcement that has been placed. Casing is most typically extracted by the drilling rig, although the use of separate hydraulic jacks is not uncommon.

- Load Ranges. It is most common to find CASE 1 micropile service loads in the range of 500 to 1500 kN, although test loads of more than 1780 kN in silty sand and more than 3340 kN in dense alluvial gravel have been reported (Bruce et al., 1993c). Most recently, a maximum test load of 5200 kN was obtained for a 244-mm-diameter Type B pile, and 3300 kN for a 178-mm-diameter Type B pile, both founded in silty sand (figure 18).

- Corrosion Protection. When drill casing is used as the load-bearing element, it is difficult to provide pre-applied corrosion protection in the form of a coating or sheath. Instead, grout is regarded as protection, and a nominal reduction in steel thickness may be considered as sacrificial. It is possible to paint exposed pile sections with protective coatings.

Any elements introduced into pre-drilled holes can be protected, either by separate plastic sheath (grouted annulus around the pipe or bar) or more commonly by epoxy coating (e.g., Groneck et al., 1993).



1 in = 25.4 mm; 1 ft = 0.305 m; 1 kip = 4.448 kN

Figure 28. Test performance comparison of two 11-m-long Type 1B micropiles, with and without permanent casing, Coney Island, New York (Bruce, 1988).

In general, due to the compressive nature of the stresses in most applications, microfissuring of the annular grout around the steel pipe is not considered to occur, and so concerns have not been typically high over-corrosion susceptibility. However, recent applications of high-capacity permanent piles subject to tensile stresses in aggressive environments are drawing attention to this issue.

British Practice

As with the United States, there are many contractors, each with his/her own special techniques, as illustrated in table 5. However, in general, the technology is regarded as mature, and national practice is fairly uniform. There is less emphasis on achieving high capabilities than on minimizing structural movements. Applications tend to be largely designed by consultants, and specifications are therefore mainly prescriptive. All applications have been CASE 1. No preloading case histories have been reported.

Table 5. Main features of various systems of "micropiling" (Fleming et al., 1985).

Pile type	Company	Nominal pile dia. (mm)	Nominal design load (kN)	Special features
Bored or drilled grouted or cast in place	Cementation Piling & Foundations Ltd. (Grouted mini piles)	Range 76 to 290	Range 3 to 550	Versatile with high load transfer characteristics.
	Colcrete Ltd.	Range 125 to 225	Range 200 to 450	
	Fondedile Foundations Ltd. (Pali Radice)	Range 120 to 280	Range 100 to 500	
	Ground Anchors Ltd. (Dywidag GEWI pile)	150 approx.	400 approx.	High load transfer characteristics, especially suited to tensile load as well as in compression.
	The House Piling & Foundation Co.	180	Range 500 to 1500	
	Hydro-Technique (micro-pieux)	Range 60 to 300	Range 20 to 600	
	Quickpile (Ground Engineering Ltd.)	76		Versatile with high load transfer characteristics.
	Terresearch Ltd.	120	150	
Displacement piles	GKN Keller Foundations Ltd.	Range 160 to 300	Range 165 to 350	Top-driven steel tube, by drop hammer. Various rigs available.
	(GKN mini piles)	Tapered 160 to 100	160	Vibro-driven cast in place.
	Menard Techniques Ltd. (Menard mini pile)	Tapered 250 to 150	200	Rapid installation.
	Harison and Co. Ltd.	Range 150 to 300	Range 150 to 300	
	The House Piling & Foundation Co.	112	400	Driven.
	West's Piling & Construction Ltd. (West's mini-shell)	280	300	Conventional shell pile, drop-hammer driven.
	PM Minipile Ltd.	80+	40+	Hydraulically jacked vibration-free system.
	Roger Bullivant	65	20	Driven.

- Drilling. The drill hole, typically 120 to 250 mm in diameter, is formed to the design depth using one of the techniques described previously in this chapter. Rotary or rotary percussive drilling techniques are employed most often in the overburden (duplex) and founding strata (down-the-hole hammer). In the majority of cases, a temporary casing is required to support the borehole, at least through the upper, usually weaker horizons. Water is the most common flushing medium, but air flush is also used if ground conditions can support their use. Augering techniques are common in stiff to hard clays and marls.

  Drilling rigs vary in size, depending on the physical constraints associated with a particular application, and vary from small, low-headroom skid- or crawler-mounted electro-hydraulic rigs to large crawler-mounted diesel-hydraulic drilling machines. Barley and Woodward (1992) reported the use of underreaming techniques for test micropiles, but the method is not commercially used.

- Grouting. Grout typically comprises a neat cement, or 1:1 to 1.5:1 sand/cement mix. Appropriate fluidity is achieved with w/c ratios ranging from 0.45 to 0.60, which, in turn, yield characteristic strengths between 25 and 40 MPa. Many micropiles are constructed with sulfate-resisting portland cement to help protect against the risk of Alkali Silica Reaction (ASR) (Barley and Woodward, 1992). Most systems appear to correspond to Type A or Type B. Grouting, at pressures of up to 0.2 to 0.3 MPa, is usually conducted through the drill casing as it is extracted. Post-grouting techniques (Type D) usually are not employed, although Jones and Turner (1980) reported favorably on earlier developments in cohesive soils.

- Reinforcement. Micropile reinforcement usually comprises a single centralized deformed reinforcing bar such as GEWI. Steel tubes or small-diameter, centralized reinforcing cages (of up to six bars with full helix) are used where moment resistance is high, or very small movements or very high loads are required. The reinforcement is placed either before or after tremie grouting and usually extends the full length of the pile. Composite piles similar to the American Type S1 or Type R1 Pin piles described above have also been installed.

- Removal of Drill Casing. After the reinforcement has been placed and the hole tremied full of grout, the casing is usually completely extracted. Turner (1994) reports on systems drilled by rotary percussive (eccentric) techniques (i.e., TUBEX) where the casing is left in place as the main load-carrying element. Similar applications in Hong Kong by other British- owned companies have also been reported (Bruce and Yeung, 1983), but mainly for rows of micropiles used as retaining walls. During casing removal, to maintain the integrity of the grout column within the micropile hole, a permanent liner, usually in the form of a thin-walled plastic or galvanized steel sleeve, is often incorporated over the upper section of the pile, particularly through fill materials.

- Load Ranges. Working loads typically range between 100 and 500 kN. However, test loads of 1000 kN have been reported for micropiles in stiff sandy, silty clay, and 1800 kN for micropiles in rock (Barley and Woodward, 1992). Turner (1994) reports designs for micropiles with working loads of up to 1400 kN that have been proposed for some projects, utilizing composite reinforcement and bond zones in competent rock.

- Corrosion Protection. It is not usual that specific measures are taken to provide additional protection to the reinforcing steel, other than by the provision of standard design cover of grout to the steel. Turner and Wilson (1990) describe the use of a permanent corrugated plastic liner to a central bar reinforcement through overburden and fill, above the load-bearing stratum (weak sandstone), as illustrated in figure 29. This reflects the type of systems provided by thread-bar manufacturers, as shown also in figure 19.

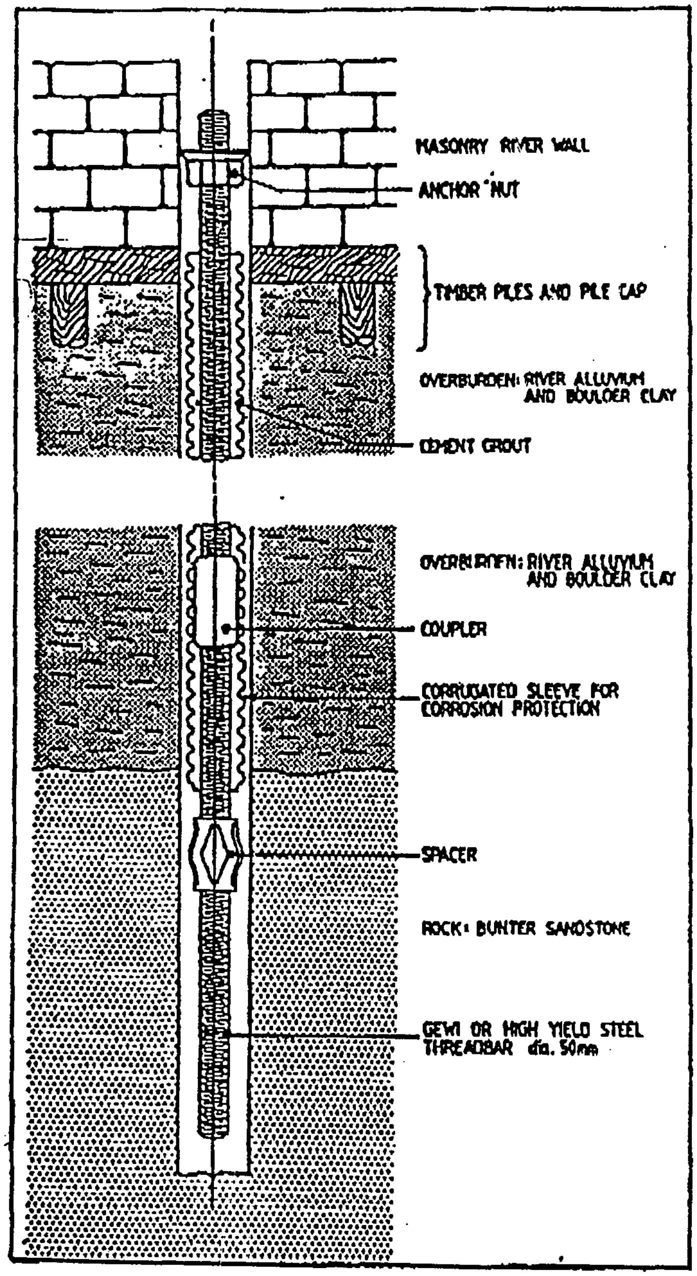


Figure 29. Type 1A micropile, installed at Liverpool, England (Turner and Wilson, 1990). [Note: It was not typical to have this level of corrosion protection in routine practice.]

Italian Practice

Although micropiles were initially developed in Italy, it would seem that current practice is heavily influenced by French and German codes and practices. This reflects the absence of comparable Italian regulations. Specifications in micropiling, as is typical throughout the specialty geotechnical construction industry, are usually of the performance type, and thus considerable scope is afforded the contractor in his/her design and construction procedures. Recent developments appear to be focused toward enhancing the unit pile capacities of CASE 1 piles, although the traditional CASE 1 and CASE 2 root pile concepts of Lizzi still remain popular, especially for the underpinning of old, historic structures. Micropiles of Types A, B, and D are common, and preloading is often used (e.g., ROPRESS system in CASE 1 piles), having been patented in the late 1960's.

- Drilling. Drill holes for lower capacity piles, as with CASE 1 or CASE 2 elements, tend to be in the range of 100 to 150 mm. Larger diameters, up to 270 mm, are used for more highly loaded CASE 1 piles. Water flush is most common, although bentonite slurry is also used during the rotary drilling of difficult rock conditions. The usual range of drilling methods is used, but with a focus on purely rotary techniques.

- Grouting. The same practices common in other countries are used in Italy. Sand-cement grouts are often employed for Type A piles, and cement-bentonite grouts are economically used in highly reinforced Type D piles. Otherwise, high-strength neat cement grouts are standard practice. Water/cement ratios are between 0.45 and 0.50 for primary grouts and up to 1.0 for post-grouting mixes. Type B grouting pressures are rarely more than 0.6 MPa, and Type B piles are relatively less common than in the United States, for example, since more attention is paid to Type D piles. In recent years, the use of air to pressurize the (sand/cement) grout column has declined with the advent of better pumps.

- Reinforcement. No single "local" steel is used and, for example, much use is made of the GEWI bar type reinforcement for medium- and high-capacity piles. Low-capacity piles typically have a single steel bar or a cage of light bars – typical of a CASE 1 or CASE 2 root pile. For Type D piles in which the post-grouting is to be conducted through the reinforcing tube, as in the TUBFIX method, the tube is thick-walled, high-strength steel with perforations and sleeves at 0.5-m centers in the bond zone. Further bar reinforcement may be placed and grouted inside these tubes after post-grouting to provide additional structural capacity. An early classification of Italian micropiles by Mascardi (1970), which focused mainly on the TUBFIX method, is provided in table 6. This appears to have been used routinely by the Rodio Company, who in 1970 published a list of 34 Italian case histories based on Mascardi's classification.

- Removal of Drill Casing. In all published examples, the temporary drill casing, if used, is wholly extracted. A short steel or plastic stand pipe may be placed around the upper pile shaft for corrosion protection or grout flow restriction.

- Load Ranges. Root piles are typically designed for service loads of up to 500 kN. Mascardi (1982) reports TUBFIX capacities of up to 1000 kN, while Rodio (1994) reports a test load of 2465 kN on a ROPRESS pile in calcarenite. Rarely, however, are such high loads needed, as the national emphasis is usually on movement minimization (for CASE 1 piles), or networks of lightly reinforced CASE 2 elements.

- Corrosion Protection. Attitudes appear to be similar to British practice, being less formalized than in Germany.

Table 6. Classification of TUBFIX micropiles (Mascardi, 1970).

Type of Micropile		Borehole diameter	Diameter of reinforcement	$+P_{amm}$ t	$-P_{amm}$ t	$\overline{P}_t$ t	$\overline{w}$ kg/cm²	$\overline{E}_t$ kg/cm²	$\overline{\lambda}$ m	$\overline{P}_t'$ t	$\overline{w}'$ kg/cm²	$\overline{E}_t'$ kg/cm²	λ' m
With tubular reinforcement	Tubfix A/1	85	51,0/35,0	23,8	−13,6	59,5	0,91	10,5	1,95	43,2	1,63	12,3	1,60
	B/1	100	60,3/44,3	30,7	−17,0	76,7	0,69	9,5	2,34	54,2	1,30	11,7	1,92
	B/2	100	60,3/40,3	34,0	−17,0	85,1	0,81	11,0	2,29	62,6	1,44	12,8	1,88
	B/3	100	60,3/35,3	37,8	−21,6	94,4	0,95	12,8	2,21	71,9	1,75	15,5	1,83
	C/1	120	76,1/60,1	41,9	−23,8	104,7	0,51	8,5	2,91	74,3	0,80	9,1	2,46
	C/2	120	76,1/56,1	46,5	−23,8	116,2	0,59	9,7	2,85	85,8	0,94	10,6	2,43
	C/3	120	76,1/51,1	51,8	−28,4	129,5	0,69	11,3	2,78	99,1	1,13	12,7	2,38
	D/1	145	82,5/66,5	53,3	−26,1	133,2	0,38	7,6	3,46	83,0	0,69	8,5	2,67
	D/2	145	82,5/62,5	58,4	−31,3	146,0	0,43	8,5	3,39	95,7	0,81	9,9	2,64
	D/3	145	82,5/57,5	64,3	−33,9	160,8	0,50	9,9	3,30	110,6	0,98	12,0	2,60
	E/1	175	88,9/72,9	68,9	−28,7	172,3	0,28	6,8	4,17	92,0	0,61	8,1	2,89
	E/2	175	88,9/68,9	74,5	−36,8	186,3	0,32	7,7	4,08	106,0	0,72	9,5	2,86
	E/3	175	88,9/63,9	81,1	−36,8	202,7	0,36	8,6	3,97	122,4	0,85	11,2	2,82
	F/1	200	101,6/85,6	86,2	−32,7	215,5	0,29	6,4	4,82	110,6	0,49	7,4	3,31
	F/2	200	101,6/81,6	92,8	−43,2	232,0	0,26	7,1	4,72	127,1	0,57	9,6	3,28
	F/3	200	101,6/76,6	100,6	−47,1	251,6	0,29	7,9	4,61	146,7	0,67	10,1	3,26
Conven monobarra	a	80	24	14,7	—	36,8	0,76	8,1	1,73	16,3	8,07	27,5	0,64
	b	100	30	23,0	—	57,6	0,61	8,2	2,16	25,4	6,46	27,5	0,80
	c	120	36	33,1	—	82,9	0,51	8,2	2,59	36,6	5,38	27,5	0,96

$+p_{amm}$ = permissible compressive load $-p_{amm}$ = permissible tensile load

German Practice

All applications in Germany are CASE 1, and most micropile projects are built around the use of the GEWI bar. Most common micropile types are A, B, and D. Since 1974, the Federal Government has granted certifications of approval, or licenses, which confirm that the particular pile components in question meet certain specified criteria. Contractors are granted national approval to use these components. Such certifications ultimately lead to the development of codes of practice. Preloading, facilitated by the GEWI hardware, is common (figure 25).

- Drilling. German industry is particularly well served by a large and powerful group of drilling manufacturers, including Krupp, Klemm, Wirth, and Hutte. Many of the overburden drilling techniques listed in table 1 were originally developed by these companies, and German specialty contractors have high levels of knowledge and experience, backed by rigorous educational and training schemes. Reflecting the medium-load capacity requirement of the industry in general, micropiles are typically in the range of 100 to 200 mm in diameter with 150 mm being the most common GEWI pile size.

- Grouting. Standard practices are used to provide micropiles of Types A, B, and D. In Type D piles, the "loop" type of post-grouting method (figure 15) is particularly common, since it reduces time (and labor-related cost), and gives adequate performance in medium-capacity piles in the prevailing soil conditions. Often such tubes are incorporated as "an insurance policy" in case service or test loads cannot be achieved with Type A or Type B methods. Post- grouting pressures with the loop system are reported not to exceed 3.5 MPa (Harvey, 1980). Primary grouts are usually neat cement mixes with w/c ratios of 0.36 to 0.44, depending on the type and quality of cement. Admixtures are used only for special grouts for shop fabrication of corrugated sheathed bars. Grout strengths typically exceed 35 MPa at 28 days.

- Reinforcement. Most installations use a GEWI bar or similar bar, and the most common diameters are 40 mm, 50 mm, and 63.5 mm. Details are provided in tables 2 and 7. Since the trend is toward very small borehole diameters, the reinforcement is usually only a single bar, given the additional space needed by couplers or multi-bar units. Increasing attention is being paid to lower capacity "self-drilling" thick-walled piles, such as those produced by MAI and Ischebeck (tables 8 and 9, respectively).

- Removal of Drill Casing. No cases have been reported of drill casing being left in place, although plastic or steel stand pipe may be placed in the upper shaft through very loose or aggressive materials during primary grouting.

- Load Ranges. Pile service loads are typically tied to bar capacities and so, by applying codified safety factors on the yield strength of the steel, service loads of 300 to 800 kN (typically around 500 kN) are found.

- Corrosion Protection. German practice focuses heavily on this aspect (e.g., DIN 4014). Minimum protection of 20 mm of grout around the steel bar is mandated in normal conditions, increasing to 30 mm in very aggressive conditions. If the micropile is to be subjected to tensile loading or is to be installed in particularly aggressive conditions, then a "double corrosion" protection system is used. The bar is encased in a corrugated plastic sheath and the annulus between the bar and sheath is filled (usually in the factory or shop) with a special high-strength, non-shrink grout.

Table 7. Further details on GEWI bars (DSI, 1992).

Service Life	Load Case 1 - 3 (acc. German Standard) Compression		Tension	
< 2 years with Standard Corrosion Protection	1	2 and 3	1 to 3	-
≥ 2 years with Standard Corrosion Protection	1	2 and 3	2 and 3	1
≤/> 2 years with Double Corrosion Protection (only for Monobar Pile)	1	2 and 3	1 to 3	-

	Bar Ø mm	Steel Characteristics* Nom. cross section (mm^2)	yield strength (kN)	Working Load $F_s/1{,}71$ [kN]	$F_s/1{,}50$ [kN]	$F_s/1{,}75$ [kN]	δ_s = 165 kN/mm² [kN]
GEWI-Monobar Pile	32	804	402	235	268	230	133
	40	1257	628	367	419	359	207
	50	1963	982	574	654	561	324
	63,5	3167	1758	1028	1172	1004	523
GEWI-Multibar Pile	3 x 32	2412	1206	705	804	690	399
	1 x 40 1 x 50	3220	1610	942	1073	920	531
	3 x 40	3770	1885	1102	1257	1077	622
	2 x 50	3927	1963	1148	1309	1122	648
	2 x 40 1 x 50	4477	2238	1309	1492	1279	739
	1 x 40 2 x 50	5184	2592	1516	1728	1481	855
	3 x 50	5890	2945	1722	1963	1683	972

* Ø 32, 40, 50 steel grade 500/550

Ø 63,5 steel grade 555/700

Load cases acc. to the German Standard DIN 1054

Load Case 1
permanent loads and regular traffic loads

Load Case 2
Load case 1 plus occasional high-traffic loads

Load Case 3
Load case 2 plus extraordinary loads

Example for using the table

Service Life: ≥ 2 years with standard corrosion protection

Load Case: Load case 1, compression

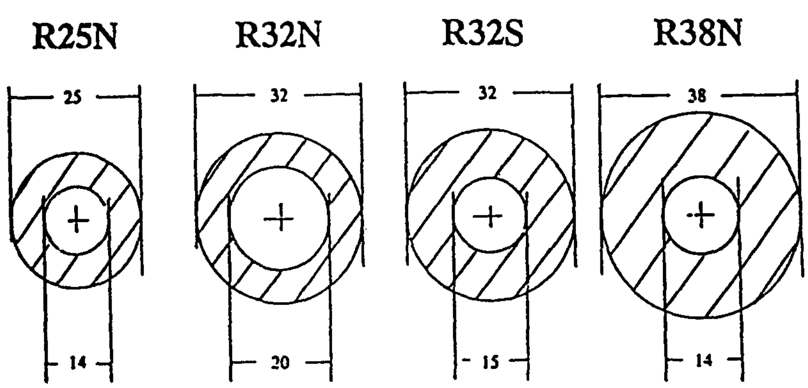
GEWI Pile: 50 mm dia.

Working Load: 574 kN

Table 8. Details of MAI systems reinforcements (MAI, 1992).

	R25N	R32N	R32S	R38N
Tensile strength	200 kN	280 kN	360 kN	500 kN
Yield strength	130 kN	220 kN	280 kN	430 kN
Weight	ca. 2,5 kg/m	ca. 3,6 kg/m	ca. 3,9 kg/m	ca. 6,4 kg/m
Thread	1 in/ISO 1719	1¼ in/ISO 1719	1¼ in/ISO 1719	1½ in/ISO 1720
Standard lengths	2 m, 3 m, 4 m, 6 m			
Custom lengths	Lengths up to 12 m are available on request.			
Special qualities	Special steel qualities and hot dip galvanized finish are available on request.			
Threads	Standard is left-hand thread. Right-hand thread is available on request.			

Technical data is relevant only on correct installation.



Measurements are approximate data before rolling (in millimeters)

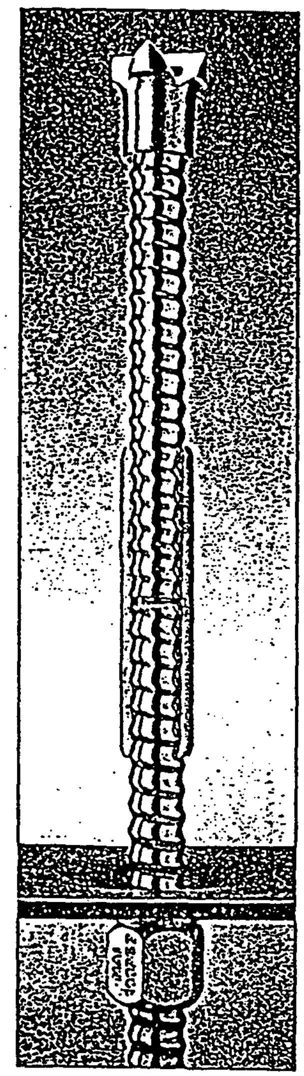


Table 9. Details of TITAN reinforcements (Ischebeck, 1993).

Ankertyp / Pfahltyp	Einheit	TITAN 30/16	TITAN 30/11	TITAN 40/16	TITAN 73/53	TITAN 103/78
Außendurchmesser	mm	30	30	40	73	103
Außendurchmesser für stat. Berechnung	mm	27,2	26,2	37,1	69,9	100,4
Innendurchmesser	mm	16	11	16	53	78
zul. Belastung auf Zug und Druck	kN	100	150	300	554	900
zul. Querkraft	kN	58	88	164	329	535
Bruchlast	kN	220	320	660	1160	1950
Gewicht	kg/m	3,0	3,5	6,9	12,8	24,7
Kleinster Querschnitt	mm^2	382	446	879	1631	3146
Kraft an der Fließgrenze	kN	180	260	490	970	1570
Fließspannung	N/mm^2	470	580	560	590	500
Trägheitsmoment	cm^4	2,37	2,24	8,98	78,5	317
Widerstandsmoment	cm^3	1,79	1,71	4,84	22,4	63,2
Plast. Widerstandsmoment	cm^3	2,67	2,78	7,83	32,1	89,6

French Practice

There is a long and rich history of micropile practice in France, and French methods and regulations have international influence. Much work is done to underpin historic structures and so CASE 1 elements designed to perform with minimum movement are common. The demand for very high capacities is not so strong. Considerable attention is being paid to the potential of CASE 2 structures, as evidenced by the establishment of the FOREVER project. All four pile types are commonly used, with perhaps Type B being less frequent. Preloading is well known but not widely used. Despite the presence of national regulations, there is still considerable scope for innovative technical and contractual practices within these frameworks, and strong technical input is still supplied by -- and solicited from -- a number of very strong specialty contractors of world renown. Although micropiles are discussed in the French Code C.C.T.G. Facsicule 62, titre V, practice is more thoroughly described with reference to the classification adopted in the French Code DTU 13-2 (1992). Micropiles are considered to have diameters less than 250 mm. The classification is illustrated in figure 30.

Type I. A cased borehole is usually formed using water flush, although drilling with bentonite slurry or "self-hardening drilling mud" (i.e., a cement-bentonite grout) is also commonly undertaken (for all types). Reinforcement, if used, is light. A sand-cement mortar with a minimum cement content of 500 kg/m^3 is placed either by gravity or under moderate pressure. The drill casing is wholly extracted. Due to the very low axial capacity ("some tens of tonnes"), Type I piles are very rarely used in practice and have very low lateral resistance. It is equivalent to a low-capacity Type A or B micropile.

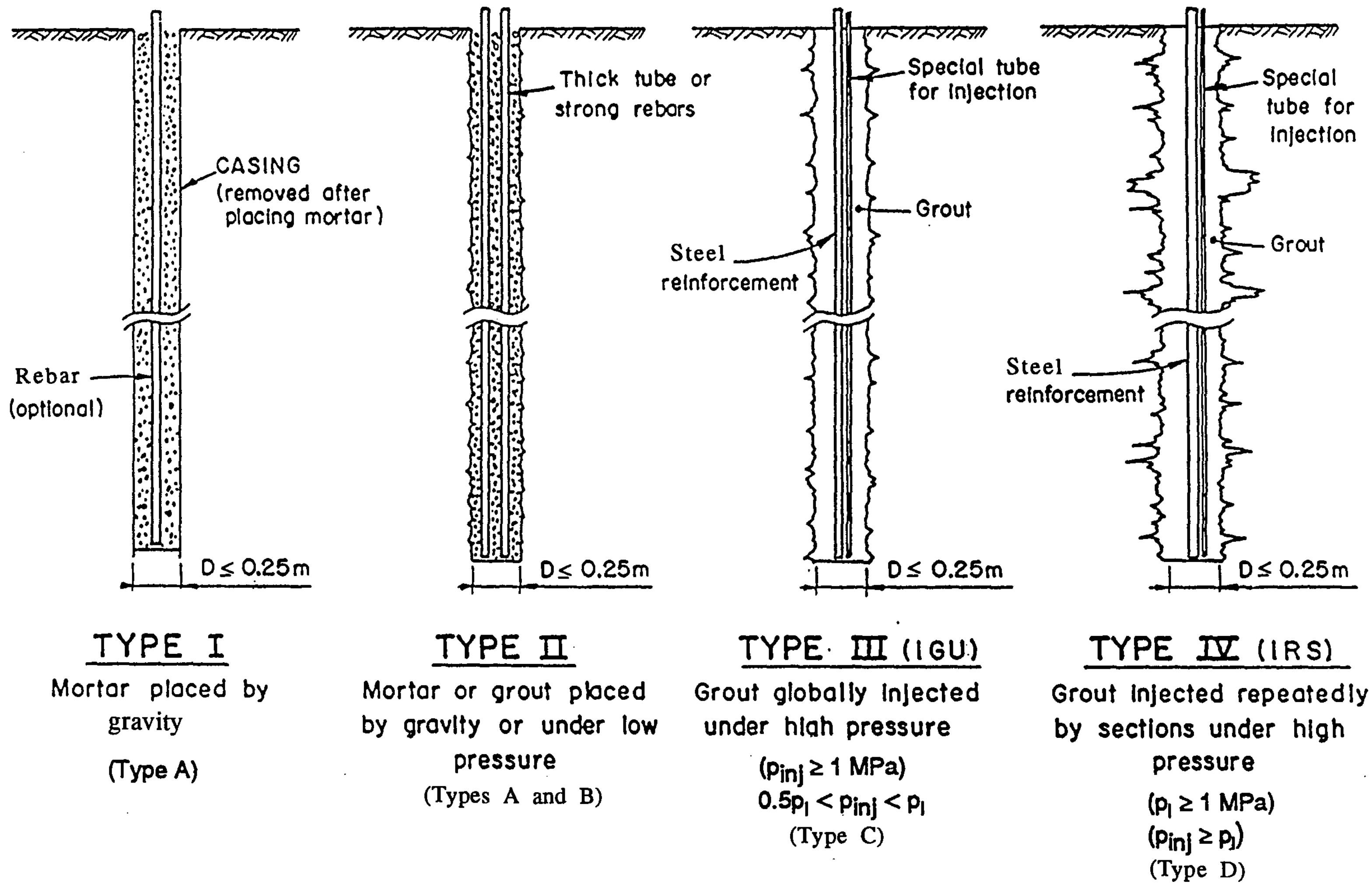


Figure 30. Current French classification of bored micropiles (Schlosser, 1994).

Type II. A casing is installed again only if pressure grouting is to be undertaken or if the ground specifically dictates. The reinforcement, in this case a thick tube or a high-strength rebar or group of bars, is then installed, and grouting is conducted through a tremie pipe. If soil or loading conditions do not dictate the need for subsequent pressure grouting, the drill casing, if present, is removed at this time, and the borehole is topped-off with additional grout as necessary. Otherwise, the drill casing is extracted in conjunction with pressure grouting. Pressures up to "hydrofracture" may be applied, but in reality never exceed 1 MPa. Neat cement grout is used with w/c ratios of 0.45 to 0.50. The allowable elastic limit for reinforcement is set at 500 MPa for tubes and 1100 MPa for bars. Such piles are forbidden in artesian conditions. A Type II pile is equivalent to a medium high-capacity Type A or B micropile.

Type III. (Injection Globale et Unitaire [IGU]). The borehole is formed with or without a casing. The heavy steel reinforcement is then placed, along with a post-grouting tube. The reinforcement may consist of a high-strength bar, a group of bars, or a tube. If tube reinforcement is used, it can be perforated and equipped with sleeves to serve jointly as the post-grouting tube. After the reinforcement has been installed, the borehole is filled with neat cement primary grout via tremie (w/c = 0.50). The casing is totally removed at this time. After 15 to 25 minutes, post-grouting is undertaken once using global techniques, i.e., without the use of packers, from the top of the hole. Injection pressures typically exceed 1 MPa. The code suggests that the IGU pile can be useful in artesian conditions.

Type IV. (Injection Répétitive et Sélective). The Type IV micropile is constructed similarly to the Type III except that post-grouting is conducted after the primary grout has set (typically at least 6 hours after placing), via a system permitting several phases of injection. This is accomplished with a double packer placed in a separate sleeved pipe or in the steel tube reinforcement itself if suitably sleeved. Due to the higher grout/ground bond capacity that can be generated, these piles can be more heavily reinforced and thus can sustain high service loads.

Table 11 provides a summary of borehole diameters, reinforcement characteristics, and nominal service loads. Although this table was produced by one contractor for one concept of pile (Type D), the details are reflective of general practice. Many applications are of smaller diameters (<150 mm) and low to moderate service loads (<500 kN). GEWI type bars of 32- to 40-mm in diameter are common. However, there is a significant number of larger capacity installations (up to a 2000-kN service load) demanding larger diameters and groups of bars, a common selection is three 32-mm GEWI bars.

Corrosion protection is provided in the form of applied coatings (e.g., paint or epoxy) or encapsulating plastic sleeves. Alternatively, it is also common to find the design philosophy of permitting a sacrificial steel loss over the service life of the pile.

Other National Practices

The practice in Europe has generally driven developments in the rest of the world. This has been accomplished through two major mechanisms:

(1) European specialty contractors have set up overseas branches or joint ventures or licensing agreements to install micropiles in specific countries, particularly in Southeast Asia, the Middle East, South Africa, and South America.

(2) European specialists have published key papers at national or international conferences, thereby stimulating the development of native technologies by local contractors. This has happened in the United States, for example.

As a consequence, although micropiling has truly a worldwide application, the technologies used in Europe and the United States cover the range of methods that can be found, local variations in materials (e.g., table 10) or equipment specifications notwithstanding.

Table 10. Types of TUBFIX micropiles used in South Africa (Rodio, 1980).

	Seamless Pipe St-52						
Pipe diameter (mm)	48.3/38.3	60.3/44.3	76.1/60.1	88.9/68.9	101.6/76.6	101.6/66.6	114.3/79.3
Working load (tons)	14	28	36	52	72	87	111
Steel area (cm^2)	6.80	13.14	17.11	24.78	62.00	46.21	53.20
Weight of steel (kg/m)	5.34	10.30	13.40	19.90	27.60	36.20	41.70

Overview of International Practices

The construction of a micropile involves a succession of processes, the most significant of which are drilling, placing the reinforcement, and grouting. There are a large number of drilling systems available for both overburden and rock, but the particular environmental needs of micropile construction naturally focus attention on a more limited range. This narrower range then tends to be utilized worldwide as a result of the comprehensive international marketing and sales efforts of the drill rig equipment manufacturers and the ongoing exchange of data and experiences in trade and professional organizations and their related journals. The newer, more innovative drilling equipment and methods are common among the five countries under closest review in this report, with other countries outside Europe or Japan tending to use more traditional methods, more appropriate to local resources and skills.

The placing of reinforcement is also a fairly standard process, although different countries use different grades, sizes, and configurations. Only in the United States is it common practice to have the drill casing left in place from

Table 11. Details of micropile types used by Soletanche (1980).

REINFORCEMENT		GEOMETRIC CHARACTERISTICS				NOMINAL SERVICE CAPACITY PROPORTIONAL TO SAFETY FACTOR	
REINFORCEMENT TYPE	Dimensions (mm)	Elastic limit σ (MPa)	Minimum borehole diameter (mm)	Area of steel S (cm^2)	Inertia of steel (cm^4)	2/3 σ. S (kN)	1/2 σ. S (kN)
SECTION	IPE 100 × 55 × 4	240	150	10	171/16	160	120
TUBES	Ø 46/60	390 530	100	12	43	310 420	230 310
	Ø 70/89	390 530	120	23	189	600 820	450 620
	Ø 97/114	390 530	150	28	394	730 1000	550 750
	Ø 109/127	390 530	170	34	584	880 1200	660 900
	Ø 157/178	390 530	200	50	1728	1300 1760	980 1320
BARS AND BAR GROUPS	Ø 20 T Ø 32 T Ø 40 T Ø 26 DY Ø 33 DY Ø 36 DY	400 400 400 800 800 800	60 to 250 mm depending on the number of bars in the group	3 8 13 5 8 10	depending on number of bars and diameter of group	80 210 340 280 450 530	60 160 260 210 330 400
	6 Ø 32 T 4 Ø 36 DY	400 800	150 150	48 40	129 129	1290 2120	970 1600

the surface down to the top of the "bond zone," although isolated examples have been recorded by certain British companies, both domestically and internationally.

It is, however, in the process of grouting that the most extreme range of practices and preferences is evident. This, of course, promoted the nature of the grouting method as the basis for the construction-based micropile classification adopted in volume I of this report. A summary of practices is provided in table 12. Practice is influenced partly by contractor capability and experience, but mainly by the provisions of codes and regulations, including those related to site investigation parameters.

Types A and B would seem to be the most popular choices, reflecting still the original concepts of Lizzi's root piles. Type C is only common in France, while Type D is becoming more popular in countries with access to sophisticated post-grouting technologies. Underreaming of micropiles is not now routinely conducted anywhere given the fact that skin friction, not end bearing, is the prime load transfer mechanism, and that developments are focused on bond improvement instead. Indeed, it would seem that the "Expanded Base" pile described by Lizzi (1982) is the sole viable member of this type, and even then, few applications have been reported.

This trend towards increased bond capacity, and so higher individual pile capacity, is in line with the fact that most micropiles are conceived to work in a CASE 1 fashion. Only in Italy and Japan are the low-pressure Type A piles common, this being in step with their prime use as CASE 2 elements.

Regarding the subject of corrosion protection, it would seem that German practice addresses it most stringently, although its importance to the French, in particular, is becoming paramount as evidenced in the national research program name FOREVER. Elsewhere, an increasing degree of attention is being paid, although nowhere is the approach as rigorous as the corresponding requirements imposed on tensile elements including ground anchors and soil nails.

Table 12. Summary of single micropile types used internationally.

Country	Pile Type A	B	C	D
United States	Relatively rare; only for bond zones in rock and stiff cohesive soils	Most common	No knowledge of use	Rare, but becoming more popular
United Kingdom	Very common	Common	No knowledge of use	Not used
Italy	Common	Common	No knowledge of use	Common
Germany	Very common	Common	No knowledge of use	Very common
France	Very common	Common	Common	Common
Others	Common	Common	No knowledge of use	Rare, but becoming more popular

Additional Considerations for Micropiles as In Situ Reinforcement

The individual elements comprising groups or reticulated networks are fundamentally no different than those used for direct axial load bearing and are constructed much the same as described above, except that the temporary drill casing is completely extracted and no post-grouting is conducted (or necessary). Type A is most common, with Type B used if a greater degree of interpile ground treatment or improvement is sought. Figure 31 illustrates the overall construction sequence of such a network. The concrete capping beam may be cast either before or after construction of the piles. However, evidence indicates that if the beam is cast before drilling, less movement will occur in the reinforced soil mass both during and after construction (Bruce, 1989a).

If the beam is constructed before drilling commences, plastic sleeves are embedded within the forms at planned pile locations and battered at appropriate angles before the concrete is poured to provide openings for subsequent drilling. Once the beam is poured, it can serve as a mat upon which drilling equipment can operate. In cases where the beam is constructed after the piles, a mud mat can be formed to serve as a working platform for pile placement (Palmerton, 1984).

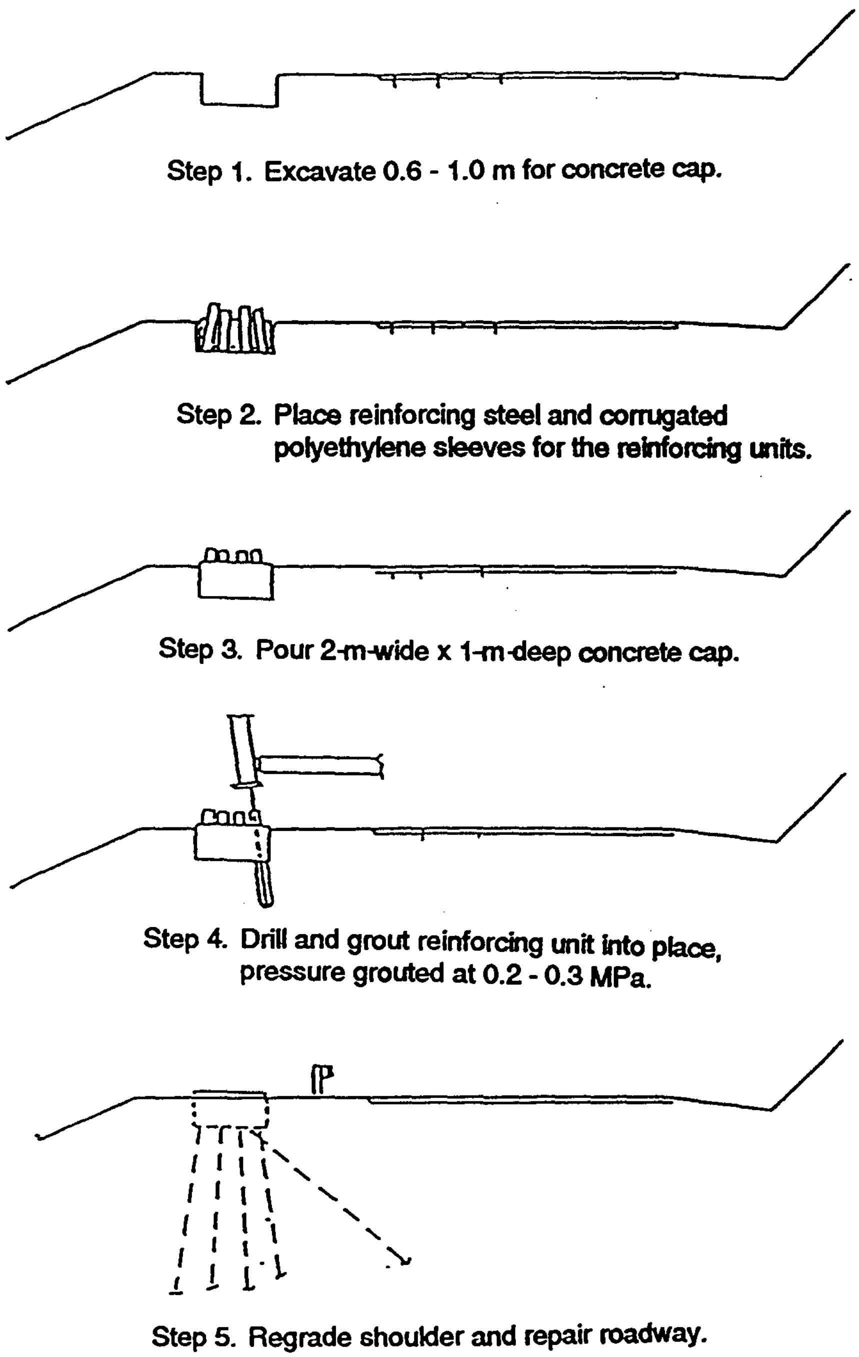


Figure 31. Typical steps in reticulated micropile wall construction (Bruce, 1992).

For relatively shallow slide planes (i.e., within about 6 m of the ground surface), the beam provides added stiffness and is therefore an essential component of the CASE 2 pile wall. If the slide plane is deeper or the sliding surface is overlain by stiff soil, the piles may be of sufficient length that the bending moments and axial forces in the CASE 1 piles are dissipated into the overlying soil prior to reaching the surface. In these cases, analyses seem to indicate that neither a cap beam nor full extension of the pile reinforcement to the ground surface may be required (Pearlman et al., 1992).

CHAPTER 2. INTRODUCTION TO SPECIFICATIONS, QUALITY CONTROL, QUALITY ASSURANCE, AND LOAD TESTING

INTRODUCTION

Micropiles are relatively slender elements that often have to be formed through random fills or heterogeneous soils underlying sensitive or delicate structures. They employ a variety of specialized drilling and grouting techniques and a wide range of constituent materials. Despite their long and successful record of use in Europe and along the eastern seaboard of the United States, micropiles remain a relatively new technology in many parts of the United States.

There is no comprehensive national standard or code of practice to be compared to those available in France or Germany, for example, although the subject is addressed specifically in certain local guidelines (e.g., Massachusetts State Building Code, 1988) or partially or indirectly in national standards, issued by the American Society for Testing and Materials (ASTM), the American Concrete Institute (ACI), or the American Association of State Highway and Transportation Officials (AASHTO) in the 1992 edition of "Standard Specifications for Highway Bridges." Due to the various unique aspects of design and construction, it is not appropriate to broadly enforce standards developed for driven, or even large-diameter, drilled piles for micropile works.

In the case of prestressed ground anchors – a similar technique from many viewpoints – every installation is routinely tested to a demonstrated factor of safety. As with other bored pile systems, however, micropile capacity can only be definitively proven by a load test, although dynamic testing may be used to demonstrate pile integrity. Although micropile load testing is relatively quick and inexpensive, it is not practical or economic to test every element installed, and indeed, there are many projects where no load testing is or can be conducted at all for technical or economic reasons.

It is therefore essential that close attention is paid to the quality of the materials and the construction at all stages of the work. The purpose of this chapter is to highlight issues that should be addressed when preparing construction and testing specifications. **This chapter is not a manual or a specification per se**: the information it contains still affords the specialist the opportunity for innovation and resourcefulness at the same time that it provides a process control framework ensuring a high-quality product. It does not address design issues (volume II) nor does it address contractor prequalification and selection methods (volume I), both topics that should be an integral part of a micropile specification. The information is provided in the following sequence:

- Materials.
- Construction.
- Pile testing.

It should also be emphasized that although micropiles are often formed in large groups, and the consequences of a single member being substandard will probably not be severe, a systematic construction defect because of inattention to detail may result in the integrity of a large number of piles being compromised. For example, Turner (1994) reports that at a recent site in England, uncontrolled drill flushing water was able to contaminate and dilute freshly grouted micropiles within a considerable area. The resultant grout washout and bleed caused the rejection of a large percentage of the piles installed, leading to major project delays and cost overruns.

Reference should also be made to the details provided in chapter 1 of this volume, especially as they relate to materials selection and application.

MATERIALS

Grout

Cement

Cements for use in micropiling are typically ordinary Type I or II or rapid-hardening (Type III) portland cements to accepted national standards. If aggressive ground and/or groundwater conditions dictate, sulfate-resisting cement (Type V) is used. Barley and Woodward (1992) note that sulfate-resisting cement is often routinely used anyway as this limits the alkali content of the grout to below 3 kg/m^3 (as recommended by BS8110, 1989) to avoid the risk of alkali-silicate reaction. Cement replacement materials such as fly ash are not used, as they reduce strength. In the United States, the following ASTM standards apply:

C150-92	Specification for Portland Cement
C595-92	Specification for Blended Hydraulic Cements, excluding Types S and SA, which are not intended as principal cementing constituents of structural concrete.

AASHTO M85 is also relevant.

Fillers

The use of an inert filler to reduce the unit cost of the grout is common in certain countries. The most common filler is sand, in proportions between 0.5:1 and 2:1 of sand/cement (reflecting early Italian regulations for reinforced concrete). Around 1:1 is the most common proportion. Sand fillers should comply with recognized national standards for aggregates used in concrete, and in the United States, these are in conformance with ASTM standards:

C33-90	Specification for Concrete Aggregates
C144-90	Specification for Masonry Mortar Aggregate

Admixtures

It is not common to use admixtures with a neat cement grout except where long pumping distances and/or high temperatures favor a fluidifier/plasticizer. Other additives are typically not necessary and should be avoided for technical, economic, and practical reasons. When using sanded grouts, suitable plasticizers, anti-shrink, or anti-bleed admixtures are more commonly used to improve both fluid and set grout properties. Such admixtures must comply with national and recognized standards and, prior to their use, tests should be undertaken or other substantiated evidence provided to demonstrate that their use improves the properties of the grout without detriment to its strength or durability characteristics. Admixtures containing chloride should not be permitted. Relevant ASTM standards are:

C260-86	Specification for Air-Entraining Admixtures for Concrete
C494-86	Specification for Chemical Admixtures for Concrete
C1017-85	Specification for Chemical Admixtures for Use in Producing Flowing Concrete

Water

Water used for grout mixing must not contain impurities harmful to the steel or the grout. It is normal to stipulate that water be of drinkable quality and comply with recognized national or local standards for water for making concrete. As an example, the British standard for ground anchorages [BS8081 (1989)] requires that water for anchor grouting should not contain oil, organic matter, or deleterious substances, and that it should not contain more than 500 mg of chloride ions per liter (500 parts per million). ACI Building Code 31811 (1992), "Building Code Requirements for Structural Plain Concrete," confirms similar requirements, noting that excessive impurities may affect not only setting time, strength and stability, but may also cause efflorescence or corrosion of reinforcement.

Reinforcing Steel

Deformed bars are exclusively used for micropile reinforcement, rather than smooth bars, and should generally conform to appropriate recognized standards. In the United States, the following ASTM standards apply for deformed reinforcement:

A36-90	Specification for Structural Steel
A82-90a	Specification for Steel Wire, Plain, for Concrete Reinforcement (for spiral reinforcement)
A615-90	Standard Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement
A706-90	Specification for Low-Alloy Steel Deformed Bars for Concrete Reinforcement
A722-90	Standard Specification for Uncoated High-Strength Steel Bar for Prestressing Concrete
A242-89	Specification for High-Strength Low-Alloy Structural Steel
A572-88C	Specification for High-Strength Low-Alloy Columbium-Vanadium Steels of Structural Quality
A588-88a	Specification for High-Strength Low-Alloy Structural Steel With 50-ksi Minimum Yield Point to 4 in Thick

Steel bars used for pile cages or bar groups should comply with national standards for hot-rolled or cold-worked steel bars for the reinforcement of concrete. Any coupling systems should also comply with such standards or have adequate and sufficient test or supporting data to allow their evaluation and use in the piling system. It should be realized that most micropile systems utilize readily available bar or coupling systems that have been developed for general use in reinforced concrete or concrete piles and drilled shafts. Thus, any such additional data are likely to have been routinely provided for the much larger construction industry market.

Steel tube and threaded or welded connections should similarly comply with relevant standards. Lap splicing is never permitted. Further data from various countries are provided in tables 2, 3, 4, 6, 7, 8, 9, and 10.

Centralizers and Spacers

Centralizers are required, where appropriate, to ensure that cage or bar reinforcement is centered within the grout column with a minimum cover of grout. The Federation of Piling Specialists (FPS) (1987) calls for a cover to "all steel reinforcement" of at least 30 mm, while the Massachusetts State Building Code (1988) specifies a minimum cover of 25 mm in soil and 13 mm in rock. It also notes that these requirements may be reduced when the steel is provided with a "suitable protective coating." This code also notes that piles subjected to sustained tensile loading in corrosive environments should have their reinforcement protected by "a suitable protective coating or encapsulation method." Similarly, German DIN4128 (1983) for small-diameter injection piles specifies various covers (table 13), depending on degree of soil aggressiveness, longevity, and grout or concrete type.

Spacers are provided to ensure adequate separation between the individual bars of a group or cage. The minimum value ranges from zero (i.e., touching) to 5 mm, although for efficient load transfer and proper grout penetration, the high end of this range should be specified as a minimum.

Centralizers should be formed from inert, non-metallic materials and be designed such that they have sufficient bearing area in soft cohesive materials that they will not penetrate into the walls of the drill hole. In addition, they should have a low cross-sectional area so that they present as small an obstruction as possible to the flow of the pile grout.

Spacers may be made of steel, except that the minimum grout cover requirement would apply to the largest dimension of the spacer.

It should be noted that where coupled monobar or multibar reinforcement is used, the couplers themselves also require the minimum cover specified.

Typical spacers and centralizers are illustrated in figure 20.

Corrugated Plastic Sheathing

Corrugated plastic sheathing for corrosion protection is usually of the type used for soil and land drainage, e.g., ASTM D2665-91b, "PVC Plastic Drain, Waste and Vent Pipe and Fittings." This material should therefore comply with an accepted national or other standard relating to such pipes. In addition,

Table 13. Minimum dimensions of concrete cover of reinforcement for the steel load-bearing member (DIN4128, 1983).

Line	Degree of aggressiveness: Attack on concrete in accordance with DIN4030	Degree of aggressiveness: Permissible aggressiveness to steel in accordance with DVGW-Datenblatt (Data sheet) GW 9	Concrete cover (1),(5) in mm
1	not aggressive	aggressive, slightly aggressive, or barely aggressive (4)	30
2	not aggressive, but with a sulfate content classified in DIN4030 as slightly aggressive		30 (2)
3	slightly aggressive		35(3)
4	very aggressive		45(3)

(1) The figures apply to concrete; where cement mortar is used, they may be reduced by 10 mm.
(2) A high-strength cement shall be used for forming the pile shaft.
(3) Piles may be inserted only if an expert in matters of corrosion of steel and concrete confirms that the long-term load-bearing behavior is not affected by a time-dependent reduction of the skin friction. In the zone outside the stress-transmitting length, other protective measures, in place of an increase in the concrete cover, may be taken (see DIN1045, December 1978 edition, subclause 13.3), but the concrete cover shall at least conform to table 1, line 1.
(4) In the case of injection piles for temporary purposes, piles may also be formed in soils that are strongly aggressive to steel provided that it can be shown by an expert that the load-bearing behavior is not impaired.
(5) In the case of piles for temporary purposes, the figures may be reduced by 10 mm.

however, other points of interest to the geotechnical application should be checked. A minimum wall thickness should be specified, the material should be resistant to ultraviolet light, and the pitch and amplitude of the corrugations should be sufficient to provide a good mechanical interlock between internal and external grout.

BS8081 (1989) recommends a minimum wall thickness of 0.8 mm, a pitch of between 6 and 12 times the duct wall thickness, and an amplitude of not less than 3 times the duct wall thickness. It also suggests that continuous diffusion-impermeable polypropylene or polyethylene sheaths are preferable, but that Poly-Vinyl Chloride (PVC) sheathing is quite acceptable with the proviso that in the event of exposure to fire, corrosion-promoting chlorides may be released. The latter problem is of particular importance to high-tensile ground anchorage tendon steels, but is generally considered less of a factor with conventional mild or high-yield steels used for micropile reinforcement.

Table 14 outlines properties of such plastics that have been specified by the Geotechnical Control Office in Hong Kong.

Table 14. Properties of plastic sheaths specified by Geotechnical Control Office, Hong Kong (from BS8081, 1989).

Property	Test method	Acceptance criteria		
		Units	PVC	PP and PE
Density	Method 620A of BS2782: Part 6 (1980)	kg/m^3	≥ 1.35	≥ 0.93
Tensile strength at yield at 23 °C, testing speed 50 mm/min.	Method 320C of BS2782: Part 3 (1976)	MPa	≥ 45	≥ 29 (PE) ≥ 30 (PP)
Softening point	Method 120A of BS2782: Part 1 (1976)	°C	≥ 75	≥ 110
Hardness (Shore D)	Method 365B of BS2782: Part 3 (1981)	—	≥ 65	≥ 65
Brittleness temperature	ASTM D746	°C	≤ -5 °	≤ -5 °
Environmental stress-cracking resistance	ASTM D1693	hour	200 (no cracking)	
Fungal resistance	ASTM G21	—	Rating 1 or less (see note 3 below)	
Bacterial resistance	ASTM G22 procedure 'B'	—	No bacterial growth on surface of specimen	
Water absorption 23 ± 1 °C	ASTM D570 Long-term immersion	% increase in mass	Max. 0.5%	
Hydrostatic pressure resistance	BS6437	—	No localized swelling, leakage or weeping	

NOTE 1. ASTM is the American Society for Testing and Materials.
NOTE 2. PVC = polyvinyl chloride; PP = polypropylene; PE = polyethylene.
NOTE 3. Unless otherwise specified, the latest issue of referenced document applies.
NOTE 4. Observed traces of fungal growth shall not cover more than 10% of the surface area.

Epoxy and Galvanic Coating

It is extremely rare to encounter galvanized reinforcing bars in micropile work, due to concerns about the effectiveness of the protection. Likewise, stainless steel bars are not used due to cost. Conversely, epoxy-coated bars are becoming more popular in the United States following progressive improvements to the quality of the coating processes. In addition to the previously listed standards for steel, the following ASTM standards also apply:

A767-90	Specification for Zinc-Coated (Galvanized) Steel Bars for Concrete Reinforcement
A775-90	Specifications for Epoxy-Coated Reinforcing Steel Bars

CONSTRUCTION

Drilling

Drilling Methods

As outlined in chapter 1, a wide range of methods can be used for micropile drilling, usually based upon rotary or rotary percussive techniques, either with or without temporary casing through overburden. Most drilling techniques are likely to be acceptable, provided they can form a stable hole of the required dimensions within permitted tolerances, and without detriment to their surroundings.

It is important in this regard not to exclude a particular drilling method because it does not suit a pre-determined concept of how the project should be executed. On the other hand, it is equally important that a drilling contractor should be aware not only of the ground conditions on the job, but also of the possible wider effects of the method chosen. Drilling within a congested urban site surrounded by historic buildings on deteriorating foundations has very different restraints than drilling the foundations for a new development on a green-field site.

It is common to find that the groundwater surface is very close to the pile head elevation. The drilling methods described in chapter 1, if employed with water, mud, or foam flush by an experienced overburden drilling specialist, will ensure that a stable (or stabilized) hole can be formed with minimal disruption to the surrounding soil mass. This is a prelude to placing the reinforcement and placing the cement grout, as described below.

On very rare occasions, significant artesian head is encountered. The simplest solutions, namely installing temporary dewatering wells or raising the pile head elevation may not be practical or economical, in which case, other technological solutions must be used. These include placing a "blow-out preventer" at the pile head so that excess flush (and grout) pressures can always be maintained on the pile hole. This is time consuming, technically challenging, and costly, and is very rarely adopted. Instead, drilling can be conducted using a viscous and heavy cement-bentonite flush, if geological conditions permit, and this "self-hardening drilling slurry" can be substituted by a stronger mix after the placing of the reinforcement, or can be subject to a post-grouting operation as in the case of Type C or D piles. This is a more common solution to this uncommon problem.

Ground Disturbance

The act of drilling and forming a hole may disturb the surrounding ground for a certain time, and to a certain distance. It is important, therefore, that a drilling method be selected that will cause the least disturbance to the ground or, perhaps more realistically, that will not cause an unacceptable level of disturbance to the ground. Conversely, it is likely that the removal of these fines from mixed soils by rigorous flushing can aid subsequent grout penetration and, therefore, increase micropile load-holding capacity by enlarging the effective bond zone diameter.

The use of high pressures in poorly controlled flushing operations should nevertheless be viewed with extreme care, because of the dangers of either overzealous flushing (which may cause voiding and collapse) or hydrofracture of the ground should blockages occur (which may lead to ground upheaval).

Tolerances

Piles may deviate somewhat from their designed inclination and position. Normal tolerances suggested by the Federation of Piling Specialists (FPS) (1987) are typical of those proposed for larger diameter elements:

In plan:	75 mm in any direction at commencing surface
Vertical:	1 in 75
Between vertical or inclined up to 1:6:	1 in 25
Inclined greater than 1:6:	1 in 15

FPS also states that these tolerances are realistic for most sites. However, where the ground contains service ducts, tolerances may have to be locally tightened and appropriate allowances made in the design of piles, caps, and ground beams. FPS further suggests that the need for close tolerances generally diminishes with capped pile groups having three or more piles, and that in such cases, it is more important to achieve alignment near the head of the pile than near the pile toe.

BS8081 (1989), related to anchor drilling, suggests a setting-up allowance of ± 2.50 degrees in addition to the deviation tolerance. The Massachusetts State Code (1988) requires that, for piles drilled without casing, the minimum design diameter of the hole be verified immediately prior to placing the reinforcement and grouting, although this is typically not a source of concern for holes in rock or cohesive soils.

Hole Support

As detailed in chapter 1, either temporary or permanent casing or some other approved method of support to the pile hole should be used for piles installed through unstable or potentially unstable ground, so as to permit the pile shaft to be formed to its full cross-sectional area.

When temporary casing is to be fully extracted during the grouting process, it should be removed in such a way that the pile reinforcement is not disturbed, damaged, or allowed to contact the soil. It should also be kept full of grout during the extraction operation in order to minimize the danger of collapse of the hole sides into the pile hole.

Continuity of Operations

The drilling, installation of reinforcement, and grouting of any given pile should be completed in a series of continuous processes as expeditiously as possible. Some materials, such as over-consolidated clays and weak mud-rocks, can deteriorate and soften rapidly on exposure, providing a consequential loss of interfacial bond capacity. Where such a phenomenon may occur, it is usual to require, as a minimum, that drilling of the load transfer length should only be undertaken on the same day as reinforcement installation and grouting. If this cannot be done, due to equipment malfunction for example, then the hole may be grouted and later redrilled just before placing and grouting the reinforcement. Care should be taken that this pre-grout is not allowed to achieve a strength markedly in excess of the native ground prior to redrilling, as this may force the redrilling activity to deviate off the original hole alignment.

Hole Plugging

Unless pile construction is continuous, it is usual that when a pile hole has been drilled to final elevation, it should be temporarily plugged at the surface to prevent debris from falling into the hole and for the safety of personnel working on the site.

It is also usual to require that any permanent or temporary casing should be left projecting above the head elevation to prevent fluid cuttings and flushing water from other operations from flowing down and contaminating the completed hole.

Grouting

Purpose

The importance of a successful 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 also extend beyond the confines of the drill hole by permeation, densification, or fissuring, or a combination of all these processes.

The grout, therefore, needs to have adequate properties of fluidity, strength, stability, durability, and impermeability. These are, to some extent, mutually contradictory goals (figure 32), although of all the factors that influence fluid and set grout performance, the water/cement ratio is the dominant one. Figure 32 illustrates why such ratios are limited to the range of 0.36 to 0.50, although even then, additives may be necessary to ensure adequate pumpability for grouts less than 0.4 in certain conditions.

Grout Tightness of Pile Hole

It is essential to the integrity of the pile that upon completion of the grouting operation, there is no significant loss of grout from any part of the pile that will be relied upon for load bearing or corrosion protection. This can be achieved by grouting to refusal during the pile construction, or in extreme cases in certain types of rock mass, by undertaking some form of permeability testing immediately after completing the drill hole and carrying out pre-grouting work if this is necessary, prior to placing the reinforcement.

For example, Mitchell (1985) described the installation of micropiles for the Pan Pacific Hotel in Kuala Lumpur that were installed in pinnacled and cavernous limestone. The micropiles had service loads of 1250 kN. The main foundation members were 1.2-m-diameter bored piles or 0.6-m x 2.7-m barrettes, and the micropiles were used as underpinning if cavities were found within the influence zone beneath these main units. For this work, a simple falling head water test was used as a basis for accepting the pile hole prior to placing reinforcement. A criterion of less than 3.2 L/min under an excess head of 0.1 MPa over 10 minutes was used.

Current practice may be summarized as:

(1) Where pressure grouting through the casing is carried out, a controlled flow rate coupled with an appropriate back-pressure and a slow fall-off in pressure on cessation of pumping are taken to indicate a satisfactory grouting operation. Where high grout takes are recorded, a sand-cement grout may be employed in order to limit travel.

(2) Where gravity filling techniques are used, without pressure grouting, the level of the grout after filling is monitored until it becomes steady. If the head continues to fall, it should be topped-off, the reinforcement removed (and cleaned), and the borehole redrilled after the grout has hardened and retested similarly. This process is known as a falling head grout test.

(3) The watertightness of a hole in rock can be assessed by a falling head water test conducted through the casing or via a packer, or by a pump-in test. The routine acceptance criterion is 5 L/min at an excess head of 0.1 MPa over a period of 10 minutes. These flow rates are related to excess head and consequently, a knowledge of the position of the local water table is necessary. Permeability testing of this type is extremely rare in micropile practice.

If there is a measured gain in water, under artesian pressure for instance, then this pressure must be stabilized by a back-pressure prior to (or during) grouting. This may occur when working within a cofferdam or during deep excavation at a level below the local water table. This situation is also addressed in Drilling Methods.

Grouting Equipment

As a general statement, any plant suitable for the mixing and pumping of fluid cementitious grouts may be used for the grouting of micropiles. This includes either "colloidal" or paddle-type mixers. In particular, however, it is usual that high-speed, high-shear, colloidal mill mixers are recommended for grout mixing and that a positive displacement pump, typically a rotary screw (Moyno) or ram pump, should be used for placing and directly pressurizing the grout. The use of air for direct grout pressurization should be avoided since it may lead to the entrainment of voids in the grout column.

The use of colloidal mill mixers produces a well-wetted, uniform cement grout that resists dilution and dispersion in water-bearing ground and is also less prone to bleeding since it permits the use of lower water/cement ratios than paddle mixers. A general discussion of mixers and pumps is provided in chapter 1.

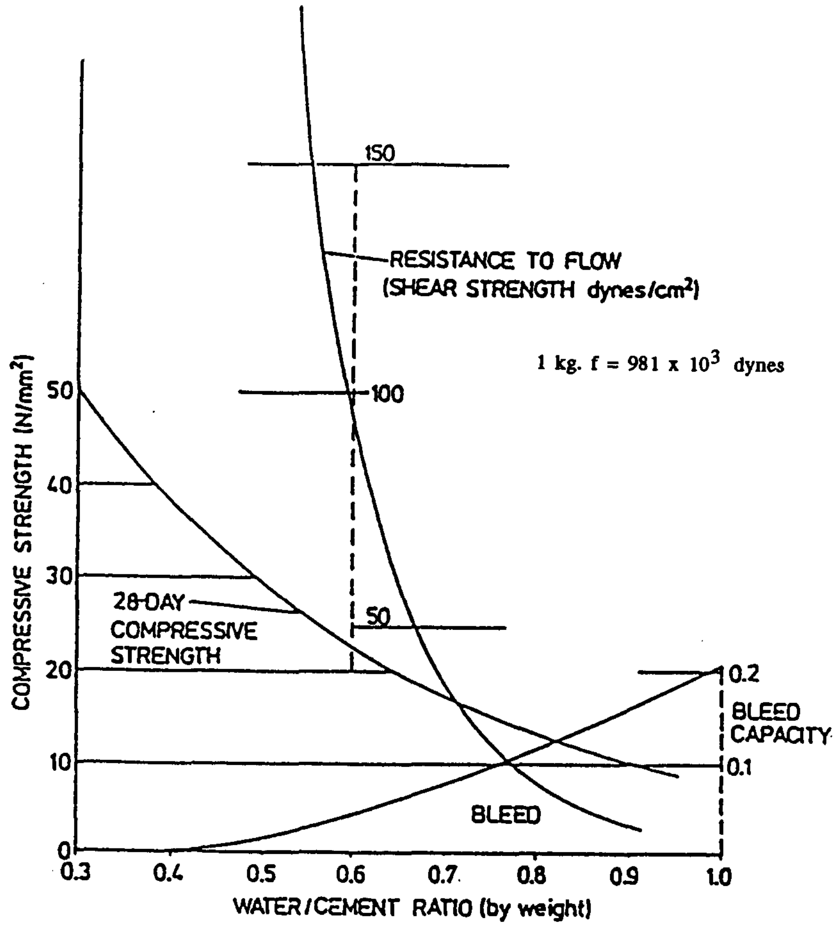


Figure 32. Effect of water content on cement grout properties (Littlejohn and Bruce, 1977).

Grout Mixing

The measured volume of water is usually added to the mixer first, followed by cement and then aggregate or filler. Admixtures are added as directed by the admixture supplier, but typically they immediately follow placing of the water.

It is generally recommended that grout should be mixed for a minimum of 2 minutes and that thereafter the grout should be kept in continuous slow agitation in a storage or holding tank prior to being pumped to the pile location. Only in extreme cases, for example where exceptionally large takes are anticipated, should "ready-mix" supplies be considered.

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 less than 1 hour in most conditions.

Grout Batching

Water should be batched into the mixer by means of a calibrated tank or a flow meter.

Cement grout is usually batched by weight, either in bags or in bulk from a silo. When using bagged cement, it is usual to require that only whole-bag mixes are used.

Sand (or inert fillers) should also be batched by weight -- either by using pre-weighed and bagged materials or, more commonly, by using a gauge box that has previously been checked and weighed. A conventional weigh-batching system may be used for larger operations. When using fillers, allowance has to be made in the water calculations for the moisture content of the filler.

Admixtures are usually provided ready-proportioned to a single bag of cement or the dosage can be adjusted by the mixer operator.

Grout Pumping and Injection -- General

Primary grout is placed by pumping the grout through a tremie pipe exiting at the bottom of the borehole. This ensures that the grout is not diluted or contaminated during placing, and that it progressively displaces water and other fluid debris out of the hole. Although the tremie pipe may be withdrawn as grouting proceeds, it should at all times remain below the level of the rising grout column. A minimum embedment of 3 m is typically required. Grouting should be continued until the grout exiting at the head of the pile is of the same consistency as that being injected.

Grouting operations should be performed in one continuous operation. During pumping, the suction lines of the pump should be airtight and the level of grout in the holding or supply tank should not fall below the crown of the inlet pipe to the pump to avoid drawing air into the injected grout.

After each section of temporary drill casing is removed, the level of grout should be checked and topped up through the tremie pipe if necessary to ensure proper continuity of the grout column.

Pressure Grouting

During casing withdrawal (primary grouting)

Pressure may be applied to the primary grout prior to or during casing withdrawal as is the case with Type B piles. Care must be taken with such pressures to avoid distress to the ground or adjacent structures, and the process is discontinued if grout is observed escaping at the surface around the casing.
When high pressure is applied to fluid grouts in certain situations, hydrofracture can possibly occur. The pressurized fluid is then no longer traveling through the strata by permeation through the pore spaces between the soil particles. Rather, the fluid pressure acts to physically separate an interface or line of weakness within the soil (perhaps a fissure plane or a soil layer or lamination) in a manner reminiscent of a hydraulic flat-jack. Once

initiated, the injected fluid pressure then rapidly exploits the widening interface, and grout will travel along the propagating line of weakness in an uncontrolled and indeterminable manner. A common rule of thumb in grouting to avoid hydrofracturing the ground is to limit grout injection pressures to 20 kPa per meter depth of ground cover at the point of injection, although many studies suggest this is over-conservative by a factor of at least two, and the exact limit can be established for each site.

In American and British practices, the use of hydrofracture techniques at this grouting stage is discouraged because of the fear of causing damage, although in Europe where there is a tendency to use higher pressures, no catastrophic occurrences have been reported. Careful logs of grout volumes and pressures must be maintained, and any structures considered susceptible to uplift damage should be monitored. In general, however, grout usually breaks through to surface around the casing before hydrofracture pressures can develop lower in the ground, and so grouting at that elevation is suspended.

For piles installed with a hollow stem auger, the grout should be pumped under continuous pressure, at controlled and monitored withdrawal rates. This is to ensure that the volume of grout injected is at least the theoretical hole volume and that no "necking" occurs of the pile shaft during extraction of the auger.

Mitchell (1985), describing the Kuala Lumpur micropiles, cites grouting at pressures of between 0.5 and 1.2 MPa causing several instances of intrusion of air bubbles and grout into newly grouted piles at other locations. Air bubbles and grout also appeared at ground level up to 3 m from the pile being grouted, although this was probably a result of near-surface travel only. This phenomenon must always be monitored, and steps taken to prevent any reoccurrence.

It should be noted that where high pressures are to be used, sanded grouts are not practicable. The pressure-filtration effect can cause the water to be squeezed out of the mix at points of pressure change, such as at joints or elbows in the relatively small-diameter pipework, so that lines become blocked. This largely explains why post-grouting operations use only relatively fluid, neat cement mixes.

Post-Grouting

Post-grouting for Type C or D micropiles is most usually undertaken using neat cement grouts of water/cement ratios between 0.6 and 1.0. The technique invariably requires initial grout pressures in excess of hydrofracture values. The grout breaks through the primary grout and exploits the interface between grout and soil, before breaking into the soil at its weakest point. Grout pressures and volumes must be carefully monitored throughout the injection of each sleeve to prevent dangerous or needless over-injection, and to progressively verify the effectiveness of the treatment.

For ground anchor practice, Jorge (1969) recorded grout injection pressures of up to 4.0 MPa, and Ostermayer (1974) prepared skin friction design curves based on injection pressures of between 0.5 and 4.0 MPa (figure 33). Bustamante and Doix (1985) recorded results on ground anchors using post-grout pressures of up to 6.5 MPa, while Ostermayer and Scheele (1977) extended the work of Ostermayer up to 6.0 MPa. Rarely are post-grouting pressures in excess of this, and a typical range during injection is 2 to 4 MPa.

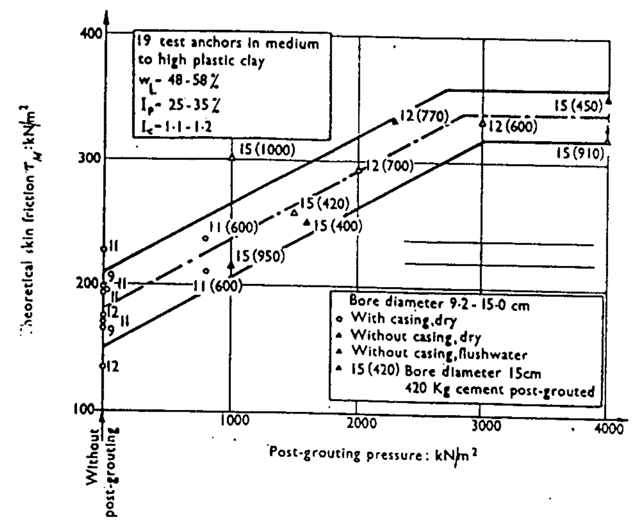


Figure 33. Influence of post-grouting pressure on skin friction in a cohesive soil (Ostermayer, 1974).

Quality Control

Because the grout is such a vital component of the micropile, close attention is paid to the control and quality of the product. Traditionally, this has been achieved by taking cubes or cylinders of the grout and demonstrating quality by the attainment of preset crushing strengths. A drawback with this approach is that it is a retrospective test. Additional quality control measures related to the batching and mixing process are therefore also used by some contractors, especially if admixtures are to be used.

It is good practice that such tests be made on the design grout mix in pre-construction trials. Measurements of fluidity and density of the fluid grout should be taken together with Vicat setting times. Bleed measurements for the setting grout and cube strengths at 7 and 28 days for the hardened grout should also be recorded. Such fluidity measurements are taken by flow cone (often called the Marsh cone) or by a flow trough (e.g., the Colcrete flow trough).

Grout density (a direct reflection of the water/cement ratio) is measured by Baroid mud balance, and is a very quick, easy, and reliable test. Bleed capacity should ideally be measured in a metal or glass cylinder, 100 mm in internal diameter, with a grout depth of 100 mm, although for routine testing, a standard 1000-mL graduated cylinder (75 mm diameter) is considered satisfactory. During the test, the container should be covered to avoid evaporation.

BS8081 (1989) recommends that fluidity, bleed capacity, density, and strength should be tested daily. Vicat setting time is normally a laboratory-based test and therefore is not usually undertaken as a routine monitoring test, although simple grout stiffening and hardening time observations can easily be made in the field.

Grout crushing strengths for a neat cement grout of 0.5 water/cement ratio are typically greater than 24 MPa in 3 days. A 28-day cube strength for such grout would typically be expected to be in excess of 40 MPa. Barley and Woodward (1992) quote characteristic crushing strengths at 28 days for 1:1 and 1.5:1 sand/cement micropile grouts of 25 to 40 MPa.

Typical bleed requirements in anchor standards for neat cement grouts are that bleeding of tendon bonding grout at 20° C should not exceed 2 percent of the volume 3 hours after mixing, and should be a maximum of 4 percent upon final hardening.

FPS (1987) specification limits quality tests on micropile grouts to workability (fluidity), measured by a suitable flowmeter or other approved means, and unconfined compressive strength of the hardened grout, measured by test cubes. FPS suggests that 1 set of 4 cubes be taken for every 10 piles installed. Relevant ASTM standards for grout testing are as follows:

C243-85 (1989)	Bleeding of Cement Pastes and Mortars
C109-92	Compressive Strength of Hydraulic Cement Mortars (Using 2-Inch Cube Specimen)
C188-89	Density of Hydraulic Cement
C359-89	Early Stiffening of Portland Cement (Mortar Method)
C451-89	Early Stiffening of Portland Cement (Paste Method)
C191-92	Time of Setting of Hydraulic Cement by Vicat Needle
C807-89	Time of Setting of Hydraulic Cement by Modified Vicat Needle

Installation of Reinforcement

Reinforcement may be placed either prior to grouting or placed into the grout-filled borehole before the temporary casing is withdrawn. It must be free of deleterious substances such as surface soil and mud that might contaminate the grout or coat the reinforcement, thus impairing bond development. Suitable spacers should be firmly fixed to maintain the specified cover, as described above.

Pile cages and reinforcement groups, if used, must be sufficiently robust to withstand the installation and grouting process and the withdrawal of the drill casings without damage or disturbance.

Records

Comprehensive records of the pile installation operation are of vital importance in establishing the basis of payment and highlighting any deviation that may be significant to pile performance at a later stage. The FPS specification (1987) recommends that the following records be kept for every pile:

Pile number.
Inclination, where appropriate.
Piling platform level related to datum.
Commencing surface level related to datum.
Nominal diameter.
Details of steel reinforcement.
Type of cement and type of any cement replacement material (where used).
Type and quantity of admixtures (where used).
Grade, quality, and slump of concrete or grout placed.
Date and time constructed.
Length from commencing surface to toe.
Length of temporary casing.
Length of permanent casing.
Final set or driving resistance, weight, and drop of hammer or the equivalent energy per blow (if driven).
Quantity of grout placed.
Details of any obstructions encountered and time taken in overcoming them.
Details of major interruptions to the construction process.

It is equally useful to record any changes approved for the particular pile by site supervision and the reasons for these changes.

Ancillary Operations

The pile construction process is not complete until the pile has been incorporated into the structure. Fleming et al. (1985), discussing conventional piling practice, suggested from their research that a large proportion of pile damage necessitating repair or replacement occurs _after_ the piles have been constructed, and are a result of operations associated with other contractors on the site. It is important, for example, that all operators are aware of the situation when moving or working around the completed piles.

Because of their small size, particular care must be exercised to prevent damage to the micropiles during the trimming process or during excavation in their immediate vicinity. Mechanical, excavator-mounted trimmers would normally cause unacceptable damage to the piles.

Connection to Structure

Piles constructed through existing structures have to develop their load by the bond between the structure, the pile grout, and the steel reinforcement. It is important that any smearing or contamination of the drill hole through the existing foundation is cleaned prior to final grouting and that the interface is mechanically roughened, if necessary (chapter 1), to enhance the bond. High-strength, non-shrink special grouts are often used to enhance the bond, and shear connectors can be fixed to the reinforcement for the same reason.

Safety and Training

There appears to be no separate set of safety regulations related specifically to micropiles. However, micropile construction is subject to all relevant national and trade regulations and guidelines. In the United States, for example, sites generally fall under the provisions of the Occupational Safety and Health Administration (OSHA), as supplemented by the guidelines proposed by the Association of Drilled Shaft Contractors (ADSC). Such guidelines may be further supplemented on a local scale by provisions governing the control and disposal of drill flush and other construction-related waste, including excess grout.

There are no special training provisions on a national basis. However, since operator training is a key to good productivity and quality, as well as safety, contractors typically pay close attention to this. Intensity and formality vary throughout the world: in the United States, the "buddy system" predominates in most companies, in contrast to the much more formal apprenticeship programs run by European contractors, usually with some form of Federal financial subsidy.

PILE TESTING

General

Exactly as is the case with other pile systems, it is common practice to subject a certain proportion of micropiles to some type of acceptance testing on each site. This testing may be in the form of preliminary testing, perhaps to failure, prior to commencing the installation of production piles. Such testing is especially common where ground conditions are unfamiliar or otherwise not fully understood, and where design assumptions have to be clearly confirmed. Less rigorous testing of selected production piles is then common to verify and demonstrate the quality of the routine product.

Micropile testing is typically undertaken by static load testing of individual elements regarded as acting in a CASE 1 manner. Tests of this type invariably feature incremental axial loading until the pile either sustains a predetermined load (typically substantially higher than the service load), or reaches a predetermined movement threshold, or reaches a predetermined creep threshold indicative of grout/ground interface failure. With the current trend towards higher capacity CASE 1 piles, failure may also be typified by a sudden, explosive loss of load, associated with a materials failure within the pile section. Most static micropile testing is of the compressive type, reflecting the most common mode of loading, although tensile testing is often conducted, typically on the same pile with a minimum of modification to the compression test setup.

Lateral testing of individual units is not common, and its interpretation is subject to many variables. This form of testing is, however, being requested more often, mirroring again both a new trend in application and the concerns engineers demonstrate over the use of such relatively small-diameter elements in certain applications. Combined loading tests (e.g., axial and lateral) have not been recorded in micropile practice to date, although there is no practical restraint on this.

Standard load-test procedures for driven piles in non-axial modes are scarce, a notable exception being ASTM D3966-81, and for drilled shafts alone they are non-existent since they are grouped with driven piles. Given the differences between these two pile groups in terms of construction, aspect ratios, and capacities, care is warranted in adopting driven pile standards without modification.

Static load testing of groups or networks of piles, either in CASE 1 or CASE 2 applications, is extremely unusual on the grounds of cost and conservatism, although this aspect of pile performance remains tantalizingly under-evaluated and under-exploited.

There are two other basic types of testing that are becoming increasingly popular for piles in general, if rather less so for micropiles to date:

Dynamic testing techniques work by imparting a major high-strain energy source to the pile, and then analyzing the resulting load/movement characteristics. The energy source can be a falling weight or an explosive charge. Various computer-based solutions are available for analysis.

Integrity testing is, in contrast, a low-strain test and has also been used infrequently on simple, centrally reinforced micropiles. These tests aim to investigate the structural continuity of a pile by monitoring electronically its response to a light hand-held hammer blow on the pile head. Analyses of data are complicated by the peculiar nature of many micropile types, which may have a variety of structural cross sections in any one pile (e.g., figure 18).

There is a large body of literature on the testing and performance of piles, which covers the entire range of activities from set up and execution, to the definition of failure and the selection of appropriate service loads. In the case of drilled and injected piles alone, the data available are voluminous and frequently inconsistent or contradictory. In the following sections, the philosophy is to provide details and procedures relevant to micropiles against a background provided by the comprehensive synopses funded by the Electric Power Research Institute (EPRI) in the last few years. In particular, reference will be made throughout to EPRI EL-5915 [Volume 1, 1988, *Conduct and Interpretation of Load Tests on Drilled Shaft Foundations (Detailed Guidelines)*, written by Hirany and Kulhawy, and its companion report (Volume 2, 1991), a condensed *User's Manual* version by Kulhawy, Hirany, and Dunnicliff.]

Static Load Tests

Planning and Documentation

Advance planning, proper organization, and comprehensive documentation are essential for a successful, efficient, and informative load-testing program. The site's geotechnical conditions influence the selection of test pile location, number of tests, loading procedure, equipment type, instrumentation, and interpretation of the results. Therefore, geotechnical site characterization is essential, and Hirany and Kulhawy (1988) recommend that the data of table 15 be collected. The details of the pile itself must also be recorded, as described and illustrated in table 16.

Regarding the items that should be recorded or measured during a test, table 17 provides a summary. Since pile behavior may be affected by the rate of loading, the load-increment schedule must be planned before selection of the test equipment and instrumentation. This selection depends on the anticipated maximum micropile load and movement, which are governed by the geotechnical conditions, test shaft geometry, and rate of load application and economics. So while figure 34 and tables 18 and 19 illustrate the range of conventional types of instrumentation, it is typical to find only dial indicators and a wire-mirror scale for head-movement monitoring, a hydraulic pressure gauge (connected into the jack circuit) and load cell to record load. Table 20 summarizes the list of items to be documented.

Table 15. Checklist of items for geotechnical site characterization (Hirany and Kulhawy, 1988).

General considerations
- site location map
- site description and plan
  - existing conditions
  - proposed construction

Regional geology
- physiography
- surficial geology
- bedrock geology

Subsurface exploration
- boring location plan
- boring logs
  - stratification, identification, classification
  - ground water elevations and water chemistry
- in situ tests
- simple field tests
- subsurface profile

Laboratory tests
- index tests
- strength tests
- compressibility tests
- any special tests

Presentation of field and laboratory test results

Load-Testing Equipment and Setups

For axial and lateral testing, by far the most common and convenient contemporary system is the hydraulic jack and reaction arrangement, as shown in figure 35 (a-e). The reaction shaft may simply be an adjacent micropile or may be replaced by a post-tensioned ground anchor. Hirany and Kulhawy's studies show that for axial loading conditions, the reaction element has a negligible influence on the test pile if it is placed at a clear distance equal to three times its diameter away. The setup is very adaptable to specific site conditions, but must always be sufficiently safe, strong, and rigid.

A useful guide to aspects of the arrangement is provided in table 21. Such arrangements are usually far cheaper and certainly more convenient to use than the use of dead weight as applied by concrete or masonry blocks.

Data Recording and Presentation

The minimum information required from a load test is the load acting on the pile head and the corresponding movement. To obtain this information, at least eight types of observations have to be recorded periodically: (1) clock time, (2) hydraulic jack pressure, (3) applied load, (4) head movement (and rotation if appropriate) measured by gages, (5) head movement and rotation determined from surveying methods, (6) elevations of the reference beam, (7) elevation of the surveying instrument referred to a benchmark, and (8) ambient temperature. However, the elevation of the surveying instrument, ambient temperature, and reference beam elevation do not have to be measured as frequently as the other items. If the test pile and soil are instrumented for evaluating load distribution, pore water stress, soil total stress, and soil displacement, a significantly greater number of observations will have to be recorded.

Table 16. List of essential additional information to accompany pile test records (Fleming et al., 1985).

Item	Information needed
Dates	Date pile was installed Date of test
Location	Sufficient detail to permit the site to be located and also the position of the pile relative to the works. The pile number should always be stated
Pile type	Various categories of pile type are set out in Report PG1
Pile installation details	If a bored pile (i.e., non-displacement), the depth of temporary casing, how much concrete, mix details, how placed (e.g., tremie or chute into dry bore), and any special circumstances, particularly regarding groundwater. If driven (i.e., displacement), type of hammer, weight, drop, final set. The driving record should accompany pile test report
Pile dimensions	Nominal diameter (or section) weight per meter run as applicable. Size of underream, bulb, 'wings' etc. Length of pile -- include the entire pile length
Installed level (driven), Concreted level (bored), Trimmed level, and Toe level	Give full data so that there is no doubt regarding the reduced level of the pile toe, the ground level *at the time of the test*, and the level of the top of the pile, either at the end of installation, after trimming, or both (above or below ground level)
Orientation	State whether vertical or raking (with degree of rake if applicable)
Design load of pile	State design load or indicate on plot
Type of test setup and settlement measuring system	Whether kentledge or tension pile test. Leading plan dimensions of the setup, including reference system. Method of measuring settlement, subsidiary leveling of reference beams, temperature corrections
Weather	Brief comment on weather conditions and extremes of temperature during test
Soil information	It is not generally feasible to provide full soil information. If a report exists, the name of the company who produced the report should be given, together with a reference number. The position of relevant site, investigation boreholes should be given on the pile location plan. Summary logs of nearby boreholes with SPT N values and cohesion values alongside are useful. In some cases, a Bored Piling Contractor will log the boring -- such logs should be given with an indication of their source. The *reduced levels* of boreholes must be given

A typical sheet for manually recording the basic data is shown in figure 36. While it is possible to record such data automatically, this is rare in micropile testing and in any event should always be supplemented by manual data recording. Table 22 lists the kinds of plots required by selected agencies for presenting the results of axial tests, and figures 37 through 41 illustrate the most common presentation forms. Lateral test data can be presented in similar format, but may require more sophisticated analyses, including depth versus coefficient of subgrade reaction, or versus soil stress (volume II).

Load-Testing Procedures

Axial load tests are commonly conducted by incremental loading (figure 42), using one of several procedures, as listed in table 23. The slow-maintained procedure is the traditional ASTM procedure, which according to Kulhawy et al., (1991) "has received much criticism during the last two decades given the long duration (30-70 hours) and restricted data yield." They do not recommend this test "unless special circumstances warrant."

Combination or hybrid versions are also used, and this is typical in micropile testing where the ASTM D1143, Quick Load Test is often enhanced by adding cyclic loading provisions. The rate of loading may influence performance, and the severity of the effect depends on the geotechnical conditions. Hirany and Kulhawy's (1988) recommendations are shown in table 24. For layered soil profiles, the more stringent soil type recommendations should be followed.

Table 17. Recommendations for items to be measured during testing (Hirany and Kulhawy, 1988).

Item To Be Measured	Axial Uplift (u) or Compression (c) Loading	Lateral or Moment Loading
Butt load	1	1
Butt displacement, vertical	1	1
Butt displacement, horizontal	2	1
Butt slope	-	1
Shaft load	2	2
Shaft moment	-	2
Shaft displacement, vertical	2	3
Shaft displacement, horizontal	-	3
Tip load	2(c), 3(u)	-
Tip displacement	2	-
Pore water stress	3	3
Soil stress	4	3
Ground displacement		
surface, vertical	3	3
surface, horizontal	4	3
subsurface, vertical	4	4
subsurface, horizontal	4	3

Legend: 1 - always
2 - in most cases
3 - if financial payback can be justified
4 - rarely

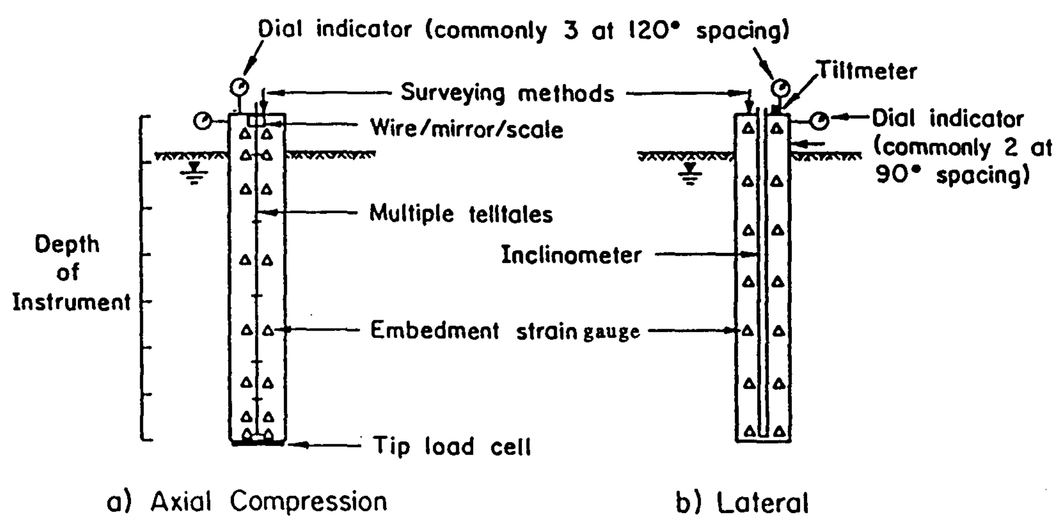


Figure 34. Illustrative instrumentation layouts for static load tests (Hirany and Kulhawy, 1988).

Table 18. General recommendations for drilled shaft instrumentation during testing (Hirany and Kulhawy, 1988).

	Instrument	
Item	Primary[a]	Backup
Butt load	Electrical resistance load cell (vibrating wire load cell)	Calibrated hydraulic jack
Shaft load	Vibrating wire sister bar [fixed (series) extensometers]	Multiple telltales
Shaft moment	Fixed (series) extensometers (sliding micrometer)	Vibrating wire sister bar
Tip load	Vibrating wire load cell (hydraulic load cell)	Multiple telltales
Butt displacement	Three dial gauges (three DCDT's)	Mirror-wire scale; surveying methods
Butt slope	Electrical tiltmeter (bubble level-pivot clinometer)	Axial steel rod extending from shaft with dial gauge at top and bottom of rod
Shaft displacement	Telltales - vertical; Inclinometer - horizontal	Same as primary devices
Tip displacement	Telltales	Same as primary device
Soil total stress	Hydraulic cell with vibrating wire (pneumatic) transducer	Same as primary device
Pore water stress	Vibrating wire piezometer (pneumatic piezometer)	Same as primary device
Soil surface displacement	Timber stakes and surveying methods	Same as primary devices
Soil subsurface displacement	Subsurface settlement points (probe extensometers) - vertical; Inclinometer - horizontal	Same as primary devices

a - Second choice for primary device is given in parentheses.

Table 19. Instrumentation commonly used for drilled shaft load tests (Hirany and Kulhawy, 1988).

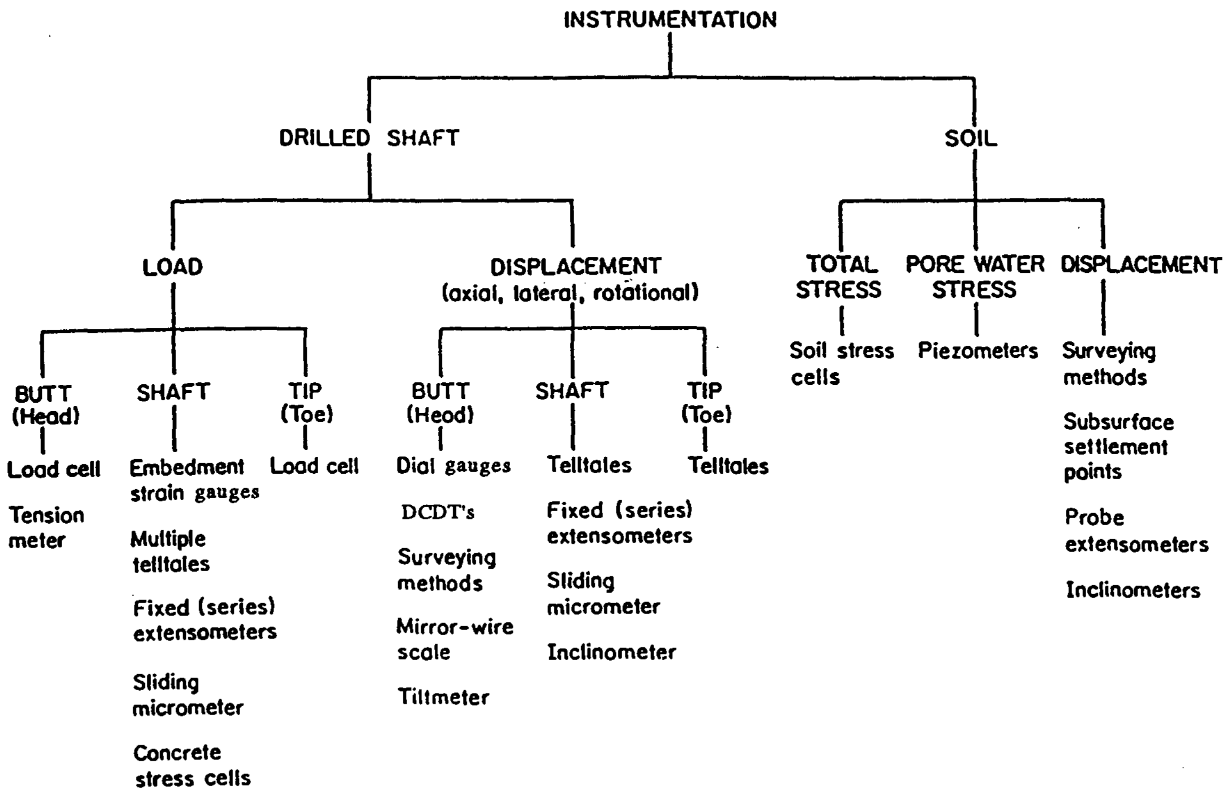


Table 20. Checklist of items to be documented during testing (Hirany and Kulhawy, 1988).

Overall load-test plan

Geotechnical conditions

Observation of geotechnical conditions during construction
- description, identification, stratification
- ground water elevation
- construction difficulties

Load-test arrangement with dimensions and instrument locations
- plan view
- cross section

Test shaft record

Load application schedule
- procedure
- magnitude and duration of load increment

Equipment for load application
- hydraulic jack and calibration
- hydraulic pump
- spherical bearings
- steel bearing plates
- load cell to measure load

Reaction arrangement

Instrumentation, including backup instruments

Reference beam

Proper shading of test area

Weather, temperature, and general information

Readings from instruments and hydraulic jack pressure gauge

Personnel conducting tests and their responsibilities

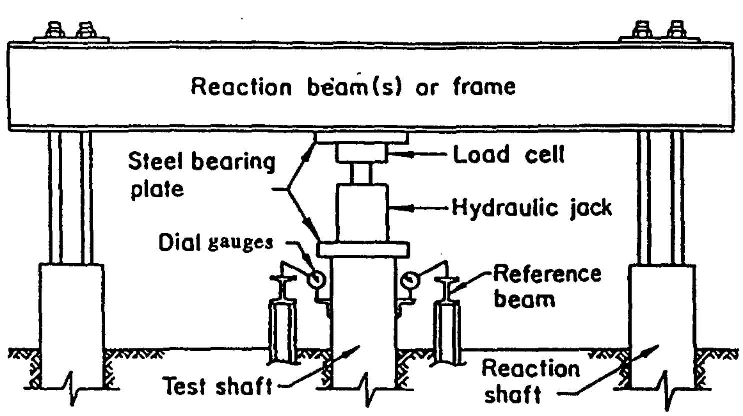


Figure 35a. Reaction arrangement for axial compression load test (Hirany and Kulhawy, 1988).

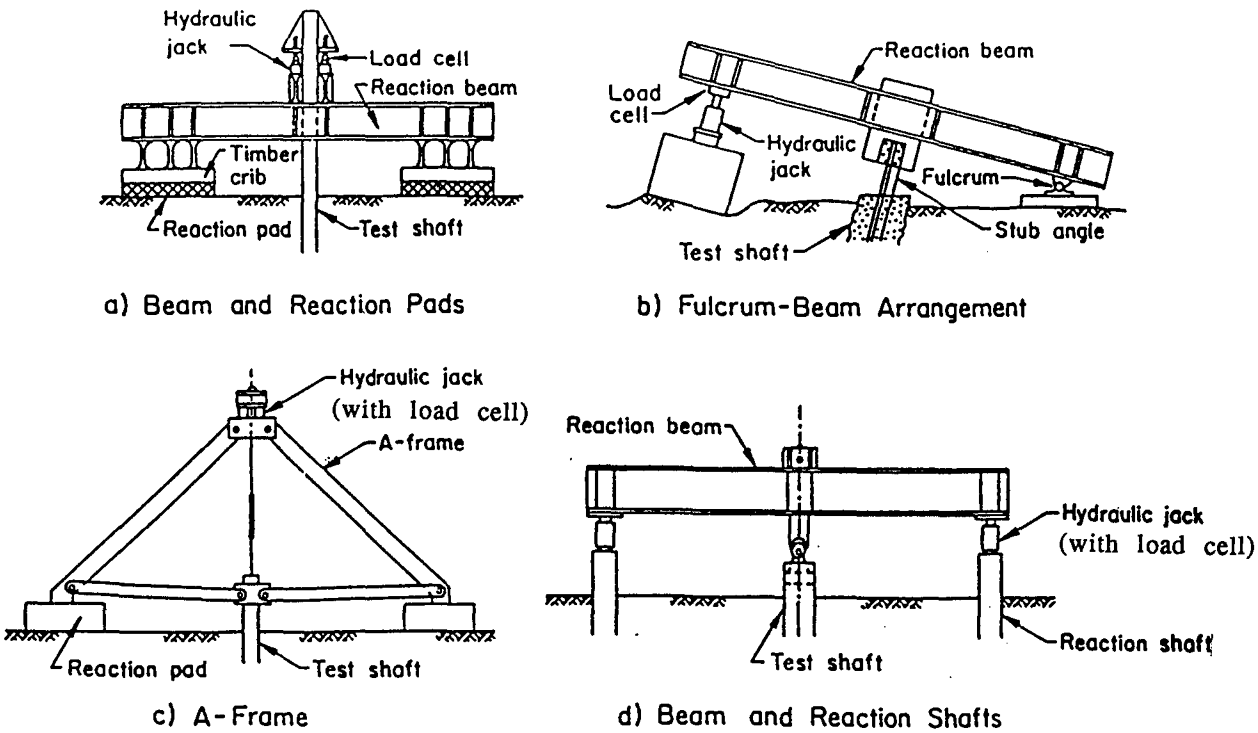


Figure 35b. Reaction arrangements for axial tension load tests (Hirany and Kulhawy, 1988).

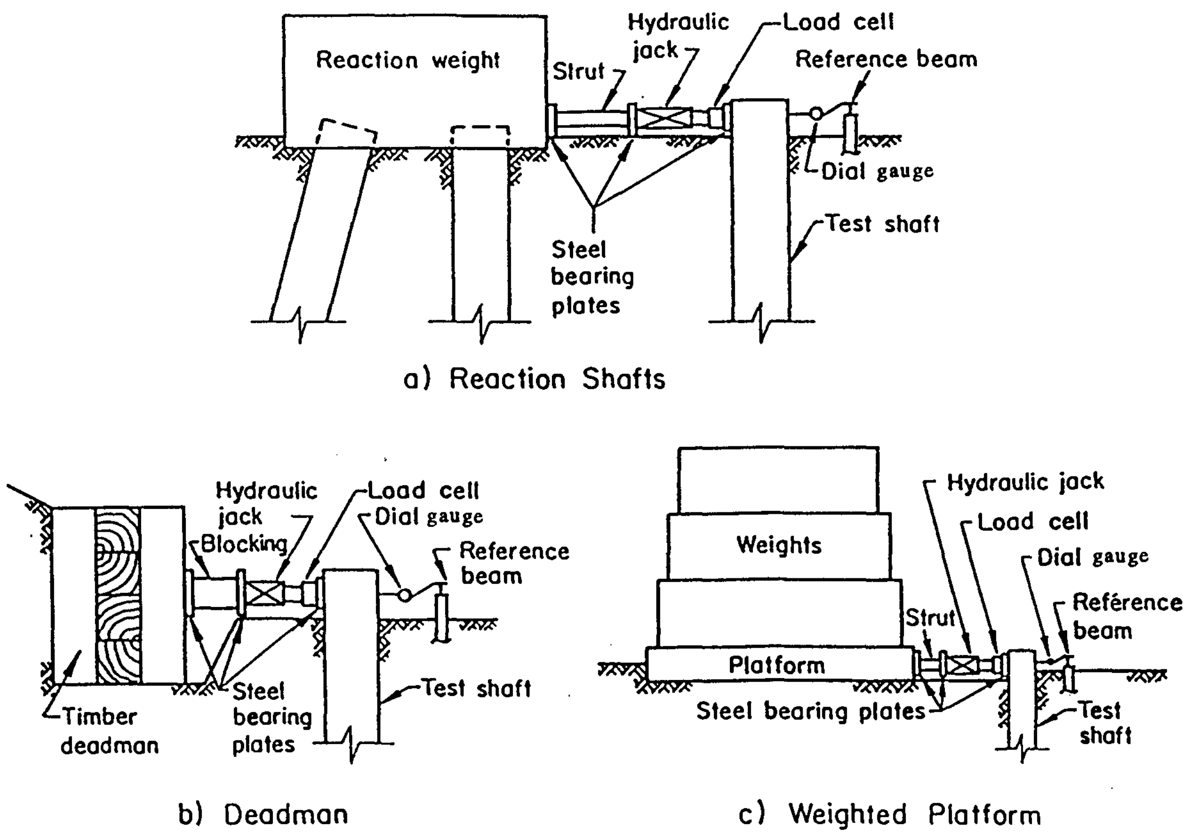


Figure 35c. Reaction arrangements for lateral load tests (Hirany and Kulhawy, 1988).

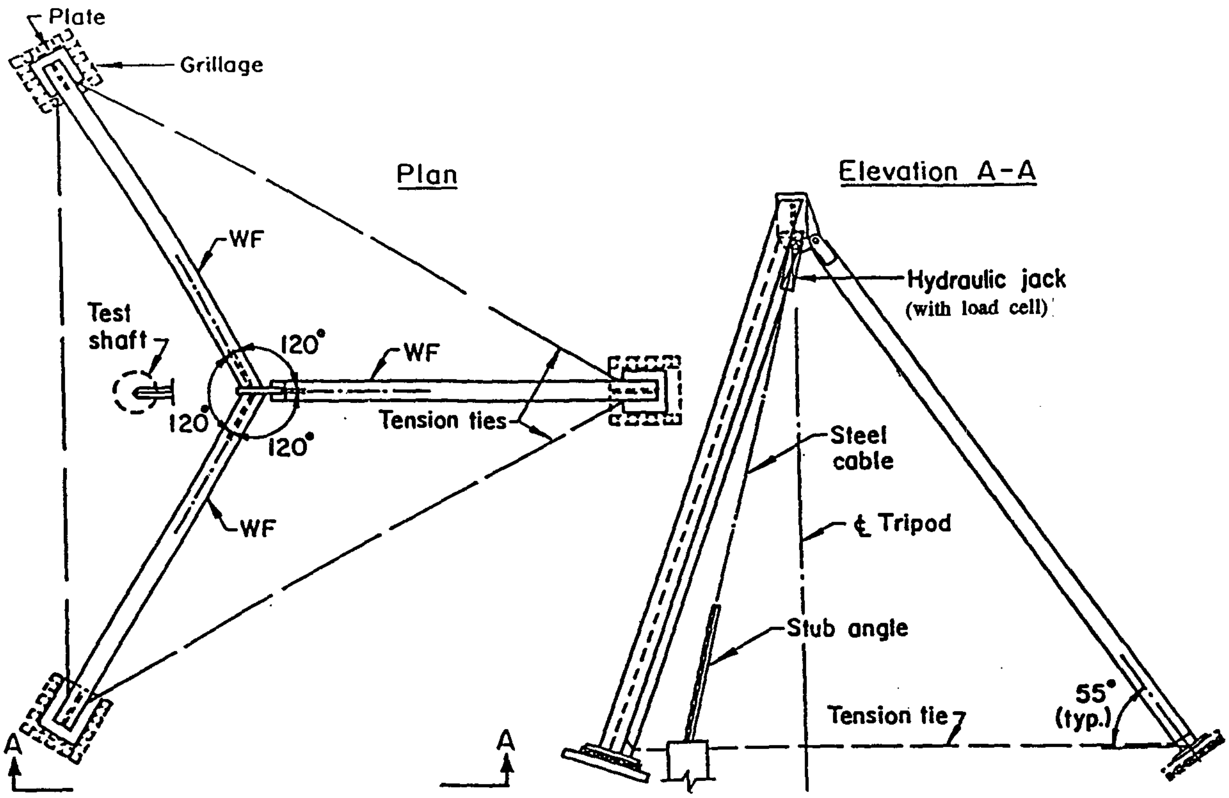


Figure 35d. Reaction arrangement for combined axial tension and lateral load test (Hirany and Kulhawy, 1988).

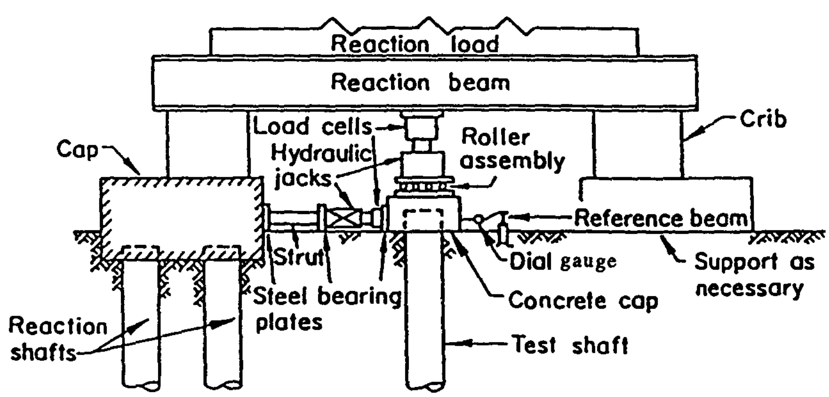


Figure 35e. Reaction arrangement for combined axial compression and lateral load test (Hirany and Kulhawy, 1988).

For evaluating maximum load, the effect of loading rate is insignificant in sand, while the effect may be substantial in cohesive soil: granulometry must therefore be considered during process selection. For load-movement response, creep does not become significant until the load is close to 50 percent of maximum load (Mitchell, 1976) and so only at higher loads should the rate of loading be considered.

For lateral load tests, the procedures of figure 42 can be used, as well as two-way incremental, reciprocal cyclic, reverse cyclic, and surge loading (figure 43), although these are not common in micropile practice. The effect of load duration appears comparable to that observed for axial loading in similar soil types and so the procedures of table 24 should also be used.

In all cases, the specific testing procedure should replicate the field-loading case. Tests involving combined axial and lateral loading are not common.

Interpretation of load test data

Kulhawy et al. (1991) have devoted considerable effort to examining the different criteria proposed for interpreting the results of axial tests, and to developing guidelines for evaluating the failure load. They avoid using the term "ultimate capacity" because it suffers from the lack of a universally accepted definition. They advocate the term "interpreted failure load."

For compression tests alone, 41 different interpretation methods have been cited in current practice for (large-diameter) drilled shafts. These are based on one or more of the following:

- Movement limitation (36 cases).
- Graphical construction (4 cases).
- Mathematical modeling (4 cases).

Micropile experience shows a similar range of criteria, although one graphical construction method (Davisson, 1972) is occasionally used in the United States. Even this method, however, may not be applicable to tests in which load increments are held over 1 hour, and may be overly conservative for holding periods of 24 hours (Peck et al., 1973). In general, however, axial micropile acceptability is simply based on (1) the ability to sustain a certain target test load (within a certain creep criterion, and/or (2) the ability to sustain a certain target test load within a certain total (and/or permanent) movement target. Lateral acceptability is based on pile head deflection at a certain load. These criteria should be project-specific and should closely reflect the expectations of the pile in service.

Other Aspects Specific to Micropiles

Throughout this report, the close links between ground anchor and micropile practice have been noted and it is understandable that these similarities include load testing also. With ground anchors, every installation is load tested and so considerable attention is paid in the numerous national codes of practice to testing and acceptance criteria. This level of attention is not apparent in those micropile-specific documents that do exist, notwithstanding the comprehensive guidance provided by Kulhawy et al. (1991) on drilled shaft

Table 21. Recommendations for arrangement for axial, lateral, and moment load tests (Hirany and Kulhawy, 1988).

Item	Recommendation
Loading system (axial and lateral; low moment)	Hydraulic jack and reaction arrangement with adequate steel bearing plates, spherical bearings or seats, and load cell
Loading system (high moment)	Moment pole with cable rigging, winch, and load cell
Hydraulic jack	Capacity: $\geq$ 1.5 times Q_{ult}
	Stroke: axial compression $\geq$ 0.1B axial uplift $\geq$ 50 mm (2 in) lateral $\geq$ larger of 0.1B or 2^o butt rotation
	Calibration: apply hydraulic pressure to jack and measure piston force with load cell or in testing machine
Load cell	Gauged to compensate for eccentric loading; Capacity same as hydraulic jack; Calibrated prior and subsequent to use
Reaction arrangement	Any arrangement that is safe, has a working load capacity equal to at least 1.5 times Q_{ult} or M_{ult}, and has a minimal influence on the test results
Reaction shaft distance from test shaft	Axial: $\geq 3B_r$ and $\geq$ 1.5 m (5 ft) Lateral: 20B or 6.1m (20 ft) minimum
Reference beam	Follow Table 3-1.

B - test shaft diameter
Br - reaction shaft diameter
Q_{ult} - estimated maximum axial or lateral capacity of the test shaft
M_{ult} - estimated maximum moment capacity of the test shaft

Project ____________ Test shaft no. ____________ Sheet ____ of ____
Location ____________ Shaft diameter ____________ Temp. ____________
Date ____________ Bell diameter ____________ Weather ____________
Engr/Techn. ____________ Butt elevation ____________ Hyd. jack no. ________
Remarks ____________ Tip elevation ____________ Pres. gauge ________
____________ Ground elevation ____________ Load cell no. ________

Clock time	Elaps. time ( )	Reqd. load ( )	Hyd. jack pressure ( )	Load cell K=		Butt displacement									Remarks
						Gage 1		Gage 2		Gage 3		Mean (mm)	Optical level/ mirror-wire scale ( )		
				Gage	Load ( )	reading	Σchange ( )	reading	Σchange ( )	reading	Σchange ( )		reading	Σchange	

Figure 36. Typical data sheet for butt (head) load and displacement (Hirany and Kulhawy, 1988).

Table 22. Plots of data recommended by selected agencies (Hirany and Kulhawy, 1988).

Agency	Requirements
ASTM	Not specified
ICE	Load-displacement time
CIRIA	Load-displacement time
NYSDOT	Load-displacement time Load-net displacement Load-telltale displacement
SPC	Load-displacement Load-tip displacement Load-creep displacement Displacement-number of cycles

ASTM - American Society for Testing and Materials (D1143 - 1987)
ICE - Institution of Civil Engineers (1978)
CIRIA - Construction Industry Research and Information Association (Weltman, 1980)
NYSDOT - New York State Department of Transportation (1977)
SPC - Swedish Pile Commission (1980)

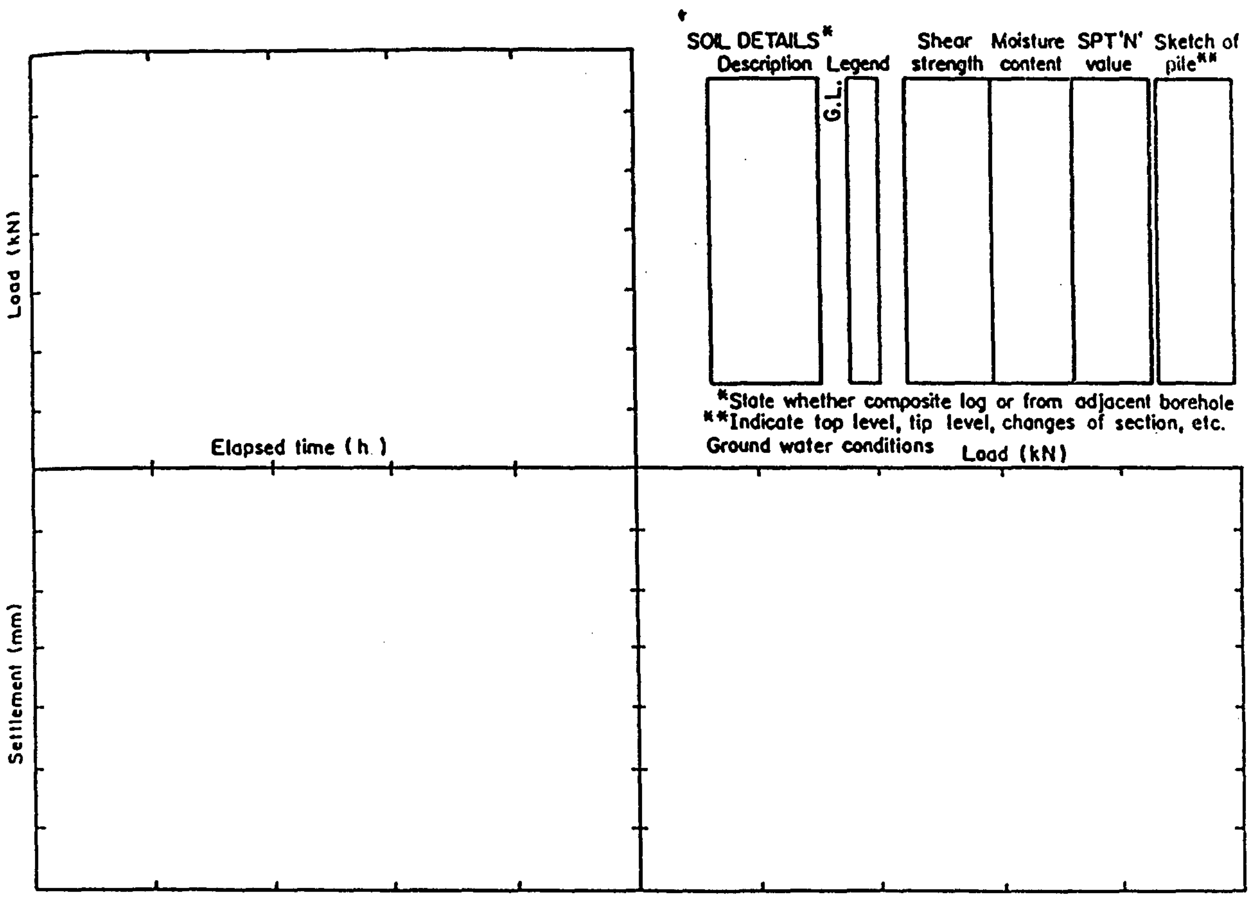
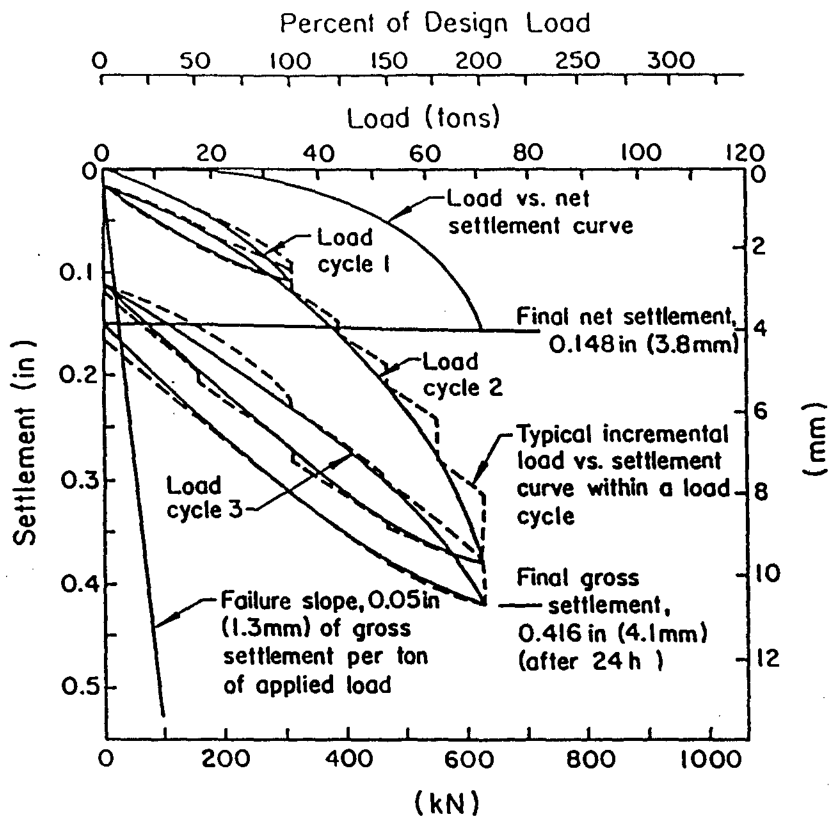
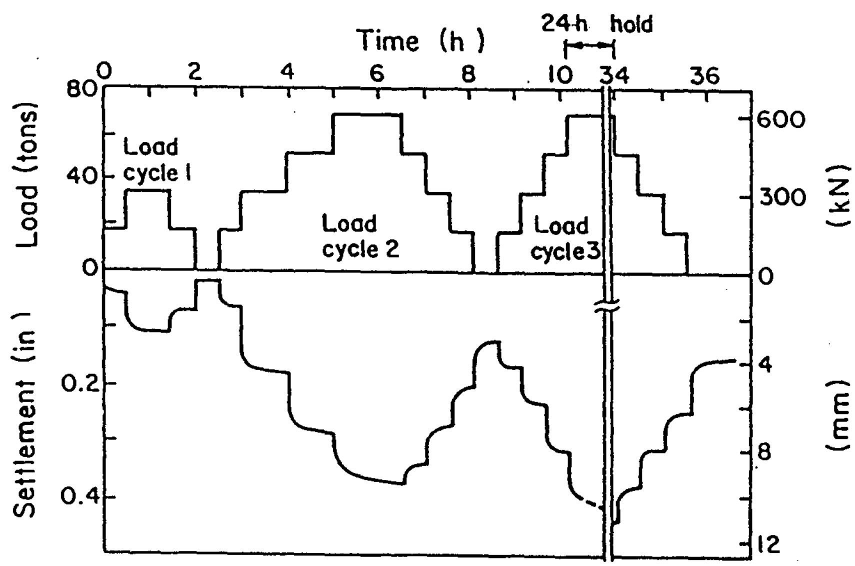


Figure 37. Load/movement plots recommended by CIRIA (1980) (Hirany and Kulhawy, 1988).



a) Load/movement curves for a typical incremental static load test.



b) Load/movement versus time for an incremental static load test with a design load of 35 tons (typical for all design loads).

Figure 38. Plots recommended by NYSDOT (1977) (Hirany and Kulhawy, 1988).

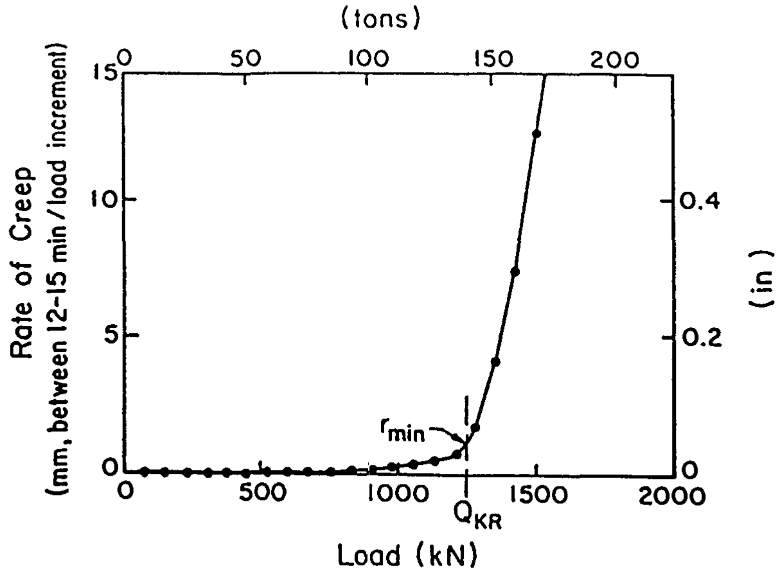


Figure 39a. Load/movement plot recommended by SPC (1980) (Hirany and Kulhawy, 1988).

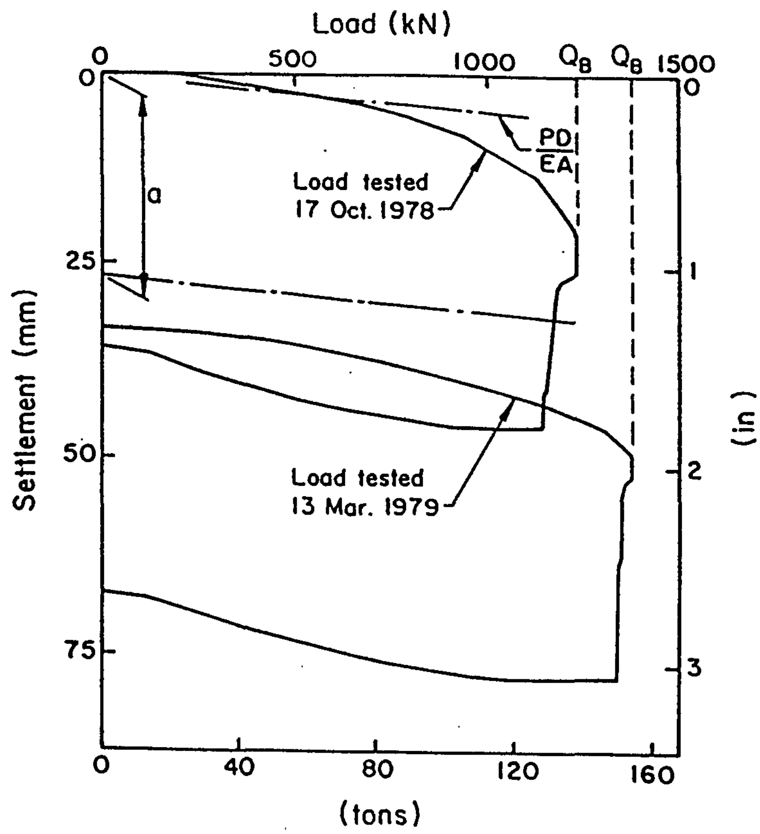


Figure 39b. Rate of creep versus load (SPC, 1980) (Hirany and Kulhawy, 1988).

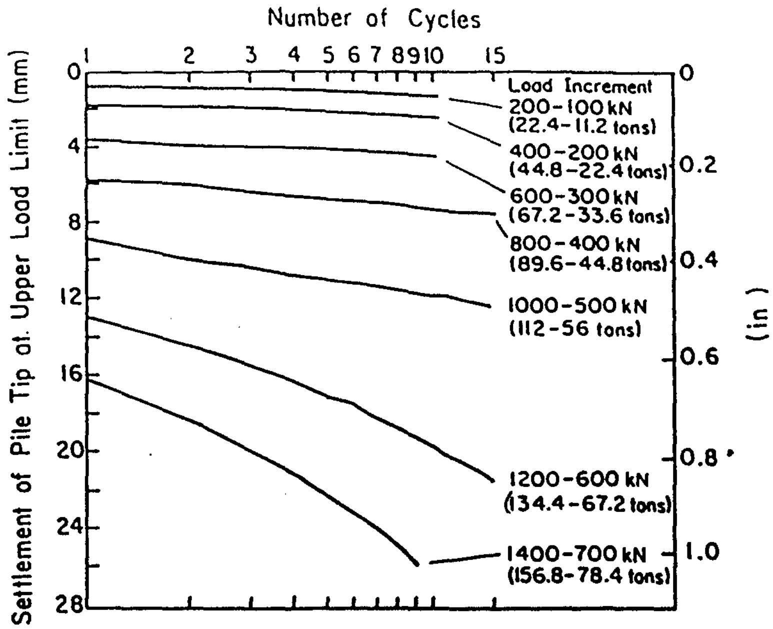


Figure 40. Movement versus number of cycles (SPC, 1980) (Hirany and Kulhawy, 1988).

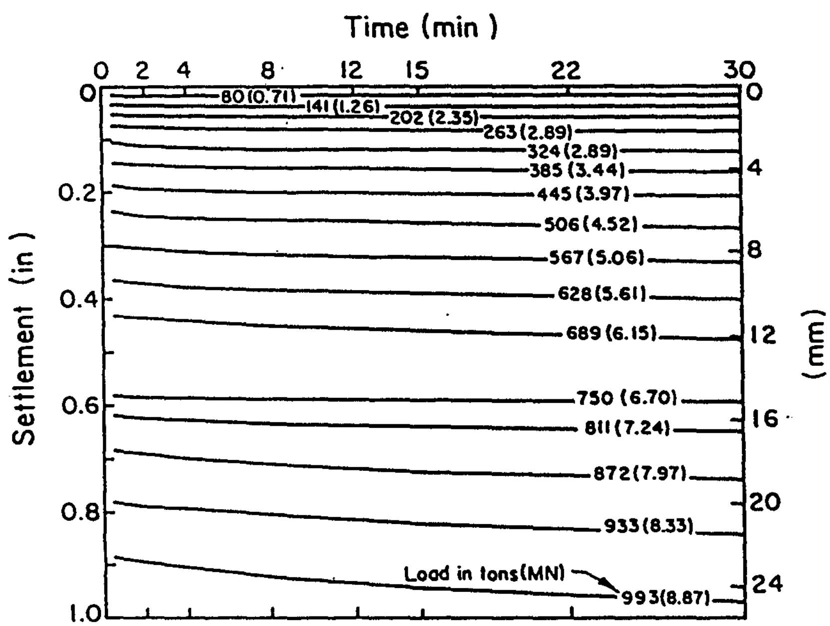


Figure 41. Movement versus time (Beckwith and Bedenkop, 1973, in Hirany and Kulhawy, 1988).

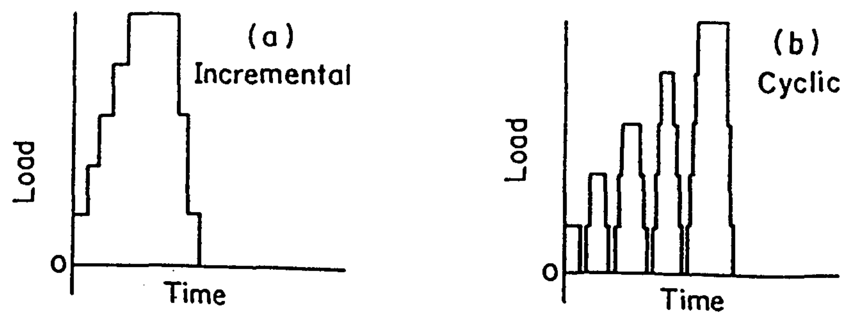


Figure 42. Incremental and cyclic load-testing procedures (Hirany and Kulhawy, 1988).

Table 23. Procedure for axial uplift tests (Hirany and Kulhawy, 1988).

PROCEDURE	DESCRIPTION
Maintained Load	Each of eight equal load increments is maintained for up to 2 hours until the rate of displacement at a maximum load over 12-24 hours reaches a specified value. (ASTM D1143 Compression, ASTM D3689 Tension*)
Constant Time Interval	Each of eight equal load increments is maintained for a specified duration (usually 1 hour).
Equilibrium	After applying pressure to the jack, the pressure supply is turned off and the equilibrium load and displacement are recorded over a 5-15 minute period. Up to 10 increments.
Cyclic	The foundation is loaded and unloaded after each load increment.
Constant Rate of Penetration/Uplift (CRP)	A constant rate of displacement is applied to the shaft.
Quick Test	Each of 10 to 20 equal increments is maintained for 2-5 minutes until failure.
Quick-Maintained Load	As for maintained load test, but each load increment is maintained for about 15 minutes.

*ASTM D3966-81 applies for lateral loads

Table 24. Recommended load-testing procedures as related to soil conditions (Hirany and Kulhawy, 1988)

Soil Type	Test Purpose: Maximum Load	Load Displacement	Load Distribution
Clean sand and gravel	Quick	Quick	Quick
Micaceous and carbonaceous sand and gravel	Quick Maintained	Quick Equilibrium	Quick Equilibrium
Sand and gravel with > 20 percent fines	Quick	Quick Equilibrium	Quick Equilibrium
Lightly cemented and weakly structured sand and gravel	Quick Maintained	Quick Equilibrium	Quick Equilibrium
Silt	Quick	Quick	Quick
Normally consolidated and lightly overconsolidated clay	Quick Maintained	Quick Equilibrium	Quick Equilibrium
Heavily overconsolidated clay	Quick	Quick	Quick

piles in general. It is, therefore, common to find much of the testing philosophies and terminologies associated with anchoring being adopted for micropiles.

For example, in the United States, the influence of the Post-Tensioning Institute's Recommendations (1986, reissued 1996) is strong. Three major classes of test are distinguished and this approach is reflected broadly in micropile practice:

(1) Pre-Production Tests.
(2) Performance Tests.
(3) Proof Tests.

(1) Pre-Production Tests are tests performed to demonstrate to the engineer, client, or other interested party, including the piling contractor himself/herself, the detailed performance of the micropile system in the particular ground conditions, including the suitability and interaction of materials, components, methods of construction, and workmanship. In this respect, an enhanced range of performance data may be required from the test: repeated load cycling at gradually increasing load levels to examine the load/movement characteristics of the pile shaft, or the method of load transfer into the ground or extended creep testing, for example. Extensive instrumentation may be associated with such tests. Such piles are often tested to failure and are not incorporated into the permanent works.

(2) Performance Tests, by contrast, are site-specific and are required to demonstrate in some detail the adequacy of the design and performance of the pile constructed in the same manner and located in the same strata as the production piles. In conventional piling practice, it is often required that measures are taken with such piles to ensure that they transfer the test loads

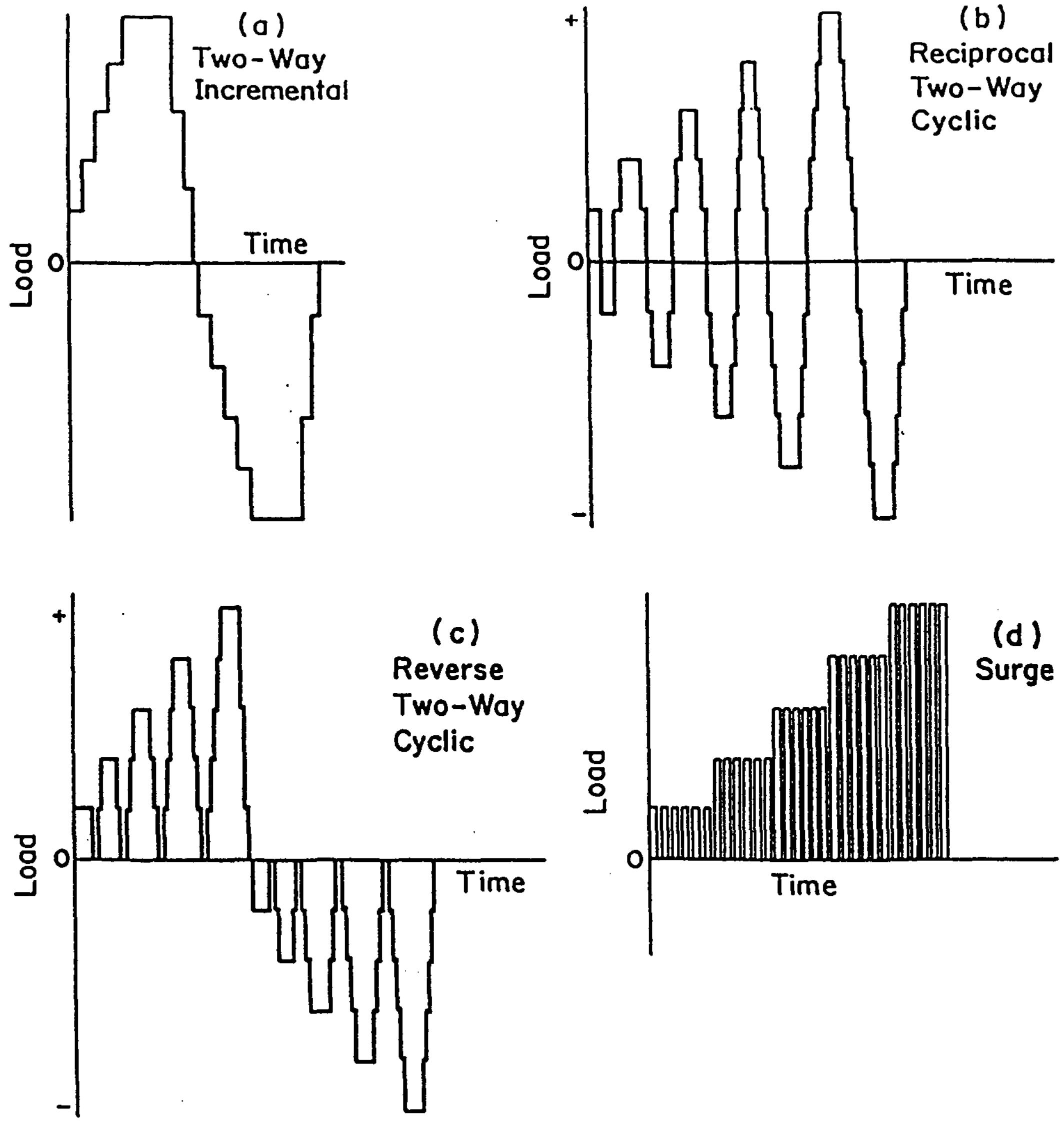
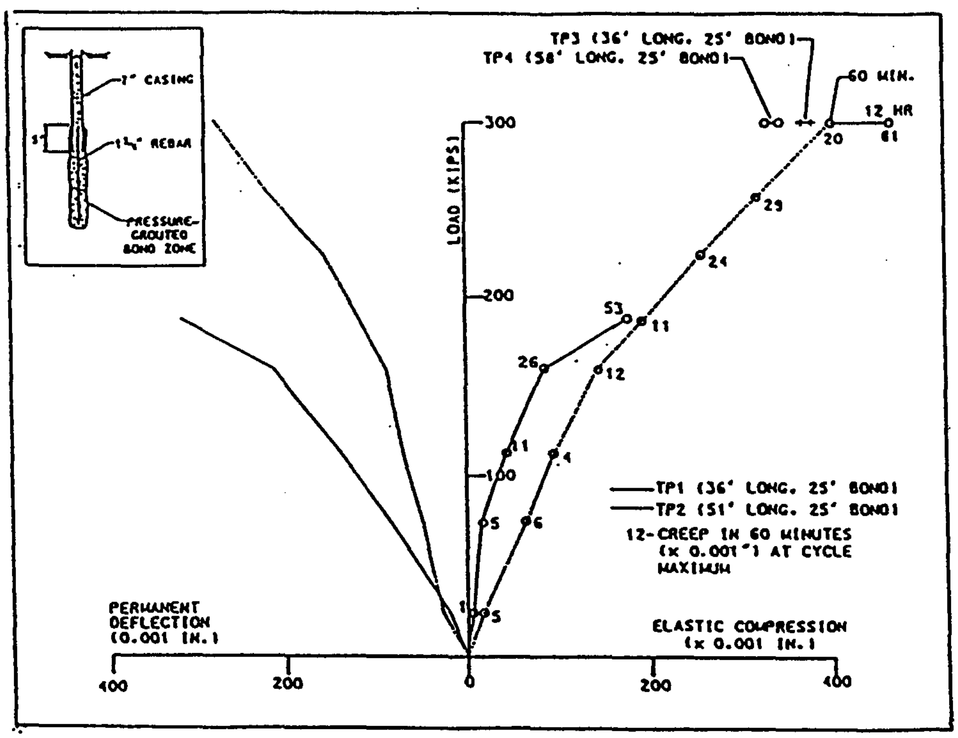


Figure 43. Lateral load-testing variations (Hirany and Kulhawy, 1988).

into the design-bearing stratum, by sleeving through the upper levels or otherwise. Such measures are not particularly common in micropiling practice, given the high slenderness ratio. In general, however, Performance Tests are required to act as a standard against which the performance of the production piles may be referenced and evaluated. Thus, if production piles are not sleeved, then preliminary piles are similarly left unsleeved. Performance-tested piles may be incorporated into the works (assuming no failure has been achieved), although they are quite often tested to a higher load than required under the contract and may therefore contain additional reinforcement to allow this load to be attained safely by the pile as a structural element. Test loads to twice service load are typically applied, and progressive cyclic loading and/or extended creep testing are characteristic. Typically, the first few piles installed on the project, or in each distinctly different stratum, are subjected to such rigorous tests. Likewise, if questions arise about the integrity of any given pile, as a result of geological or constructional difficulties, or questions remain after Proof Testing, then a Performance Test can be called for on those specific piles. Cyclic loading allows the total movements to be partitioned into permanent and elastic components and this is very useful in assessing load transfer or failure mechanisms (figure 44).

(3) Proof Tests are carried out on service piles installed routinely under the contract. They may be randomly selected or chosen where a particular pile or area of the site gives cause for concern. The purpose of a Proof Test is to confirm the performance of the pile as a foundation element. The pile cannot normally be loaded higher than the maximum load that can be safely applied to the pile as a structural element. Test loads are therefore normally limited to about 1.5 times service load. Such tests are typically quicker and simpler than Performance Tests, featuring progressive loading to Test Load, and only short duration creep testing.



1 in = 25.4 mm
1 ft = 0.305 m
1 kip = 4.48 kN

Figure 44. Elastic/permanent movement performance of Test Pile 1 and Test Pile 2, Postal Square, Washington, D.C. (Bruce, 1992a).

A review of American practice suggests the following current trends:

(1) Most axial compressive pile testing is undertaken in accordance with ASTM D1143-81.

(2) Tensile testing to ASTM D3689-87 is undertaken only to determine performance of the piles under tensile loads (i.e., not to predict performance in compression, although the Massachusetts State Code, 1988, permits this).

(3) Typically, routine testing by the ASTM D1143 method is to load equivalent to twice service load by incremental steps, but without load cycling. However, on many occasions, this ASTM test has been modified to allow load cycling, in a similar manner to that used for Performance Testing of ground anchors.

(4) Special test piles are often debonded through the upper horizons by external casing, or by external greasing of the load-carrying tube, above the design bond length, to ensure that the bond length is exclusively tested.

(5) "Telltales" placed at the top and bottom of the design bond length also give valuable information on pile behavior and load transfer, and are not uncommon.

(6) A soft plug is often placed at the base of the test pile to eliminate any end-bearing contribution.

(7) Lateral testing may be conducted to ASTM D3966-81.

The German DIN 4128 (1983) calls for "trial loadings" on not less than two piles, but on at least 3 percent of all piles. It highlights that tests should be carried out in those areas where it is believed that the soil conditions (for "bearing capacity") are least favorable.

The FPS (1987) specification calls for testing at "an early stage in the contract, and the test loads should not normally be greater than the Design Verification Load plus 50 percent of the Specified Working Load."

The testing of groups or networks of piles is not specifically addressed in any code. This reflects the fact that micropile testing is invariably treated as an individual unit CASE 1-type demonstration: the performance of a composite CASE 2 structure is not considered, and the possible advantages of a CASE 2 network or group are not currently exploited at the design stage. With the growing awareness of the potential for CASE 2 structures, it can be anticipated that such testing will become an integral part of their development.

Dynamic Testing

This is a method of predicting the load-carrying capacity and load-movement characteristics of a pile by measuring its response to the impact of a heavy weight, or explosive charge, on the head of the pile. It is a high-strain testing method, of which the most widely accepted method in the United States (40 States) is the Pile Driving Analyzer (PDA) method. Although such methods are becoming more common, especially in Great Britain and France, for testing of conventional, driven piles, it has yet to be used for micropiles. Given the micropiles' mode of construction and operation, and their often complex, cross-sectional compositions, it is doubtful if dynamic testing will have widespread application. A comprehensive review was undertaken by Turner (1994a).

Integrity Testing

This is a much lower strain method designed to confirm the integrity of the installed product and especially the grout or concrete. It is a valuable tool for prefabricated, driven elements, or for certain types of uncased grout or concrete-filled bored piles, such as continuous flight auger piles. However, as for dynamic testing, micropiles do not typically prove amenable to this technique and, in addition, are largely comprised of pretested load-bearing elements (e.g., casing, reinforcement) with well-defined properties. Only in rare cases, such as when the potential for a major defect has been recorded during construction and the pile cannot be replaced for technical and/or cost reasons, is the mobilization of an integrity testing setup justified. A comprehensive review of integrity testing methods was presented by Turner (1989) who estimated that between 10 and 20 percent of concrete piles are tested in this manner, as opposed to 2 percent that are subjected to static load tests. He estimated the cost of a static load test to be several hundred times higher than a typical hammer-based integrity test.

EXISTING MICROPILE SPECIFICATIONS

Despite the fact that micropiles have been used since 1952, they are still regarded in many parts of the world as a novel and innovative piling method, and the level of understanding by many engineers is not high. Consequently, they are either not addressed specifically in codes or standards (and thus, because they do not "exist," they cannot be offered), or are included under sections on other drilled and grouted (or concreted) piles. In the latter case, micropiles must then be designed to conventional piling criteria that invariably lead to unnecessarily high levels of both conservatism and cost.

Work may be governed by national codes such as the Uniform Building Code (UBC, 1991), by local State or city codes such as Massachusetts (1988), or by an owner's own specification and code. While most such codes are designed to be flexible and to allow the use of new materials and construction systems, nevertheless, disputes often arise as the result of ignorance and distrust. One major difficulty is often the definition of the pile system within conventional classifications. For example, is a micropile with a central reinforcing bar a reinforced concrete pile or a cased steel column? Alternatively, in a micropile where the main load-carrying member is the permanent steel drill casing, is this a concrete-filled steel pipe pile? Micropiles often fall outside the limits drawn for such conventional systems. For example, concrete-filled steel pipe piles in UBC (1991) are required to have a minimum outside diameter of 200 mm. In addition, the allowable minimum specified yield strength might be limited to a ceiling value that is well below the values applicable to the high-strength steel used in micropiles. No (or very little) contribution from the grout may be allowed. Similarly, drilled micropiles with a single central bar or small pile cage reinforcement might exceed normally accepted steel/concrete area ratios or length/diameter ratios for such piles. Lizzi (1982) outlined some of the problems in the design and acceptance of the original root pile from its initial development. He notes that the development of micropiling systems was spurred by the large size of conventional bored piles and the associated equipment that meant they could often not be installed in the confined and restricted working areas. He notes that contemporary national and local building codes in Europe often specified that bored cast-in-place piles could not be less than 400 mm in diameter -- several times that of his innovation.

However, as micropiles have grown in acceptance in Europe, codes and specifications have been modified or proposed to specifically accommodate the unique features of their design, construction, and performance.

In France, Bustamante and Doix (1985) proposed guidelines applicable to both ground anchors and micropiles. The French foundations code (CCTG, 1992) was principally based upon the work of the same researchers. The Unified Technical Document (Document Technique Unifié) on deep foundations (DTU 13.2, September 1992) specifically includes micropiles and deals with their classification, construction, and design parameters (appendix 1). In addition, the French also observe Norme Française NFP 94-150 (Static axial testing, 1991) and NFP 94-151 (Static lateral testing, 1993).

In the United Kingdom, the Federation of Piling Specialists (1987) proposed a specification for micropiles (termed "minipiles" in their document). A copy of this specification is attached in appendix 1.

In the United States, the Commonwealth of Massachusetts Building Code (1984) has proposed amendments to cover design and construction procedures associated with micropiles (September, 1988). This revision covered grouted cast-in-place piles of less than 300 mm in diameter and in which all or a portion of the piles are cast directly against the soil without permanent casing. A copy of this document is also attached in appendix 1.

In Germany, DIN 4128 (1983) relates to small-diameter injection piles (cast-in-place concrete piles and composite piles) of up to 300 mm in diameter (appendix 1).

The Guide to Pile Design and Construction (Geoguide 6), issued by the Hong Kong Geotechnical Engineering office, refers to "minipiles" under "special pile types," this being typical of general piling regulations. Diameters of 100 to 250 mm are included. Interestingly, a small contribution to load-holding capacity by the grout is now allowed – in line with "national...overseas practice."

APPENDIX 1. SPECIFICATIONS AND CODES

The following documents are included in this appendix:

PROJET DE FASCICULE 62 - TITRE V DU C.C.T.G.

REGLES TECHNIQUES DE CONCEPTION ET DE CALCUL DES FONDATIONS DES OUVRAGES DE GENIE CIVIL

SEPTEMBRE 1992

FASCICULE 62 - TITRE V

SOMMAIRE

ANNEXES AU TEXTE

ANNEXES AUX COMMENTAIRES

Specification for the construction of mini piles (CFPS)

Preface

Where the term 'Engineer' is used in the following clauses, this shall be taken to mean the person appointed by the Employer to supervise and control the engineering aspects of the work. In some forms of contract the equivalent term may be the Architect or Supervising Officer.

Introduction

Mini piles are generally smaller than those in traditional use for structural support and are often constructed using small boring or driving equipment. The term micro pile is sometimes used.

Mini piles are frequently employed in the same manner as traditional piles but they have a special application where piles are required for underpinning, in restricted working areas, adjacent to sensitive structures or where difficult ground conditions exist (e.g. boulders, fissured rock, old foundations etc.) (See Note 1).

Design

1. The general design of piles shall be in accordance with the British Standard Code of Practice for Foundations BS 8004 where appropriate (See Notes 2-10).
2. The piles shall be designed to carry the loads specified on the Engineer's drawings or in the provided schedules taking into account the required minimum Factor of Safety, settlement, pile spacing, pile head restraint, downdrag, the overall bearing capacity of the ground beneath the piles and any other relevant factors. (See Note 2.3).
   Values of design parameters assumed by the pile designer shall be supplied prior to the commencement of the works. The allowable pile capacity or design load shall not be less than the specified working load.
3. The axial compressive stresses shall be designed in accordance with BS8110 Clauses 3.8.4.3 and 3.8.4.4 – Short Braced Axially Loaded Columns and Clause 5.2.3.4 for pile sections located in fully restrained ground conditions. (e.g. Rock, stiff clay, or dense granular soil).
4. In the case of piles required to act in tension or bending, the stresses in the reinforcement shall be in accordance with the appropriate parts of BS8110 (See Note 10).
5. The ultimate pile bearing capacity shall be taken as the load at which the resistance of the soil to the motion of a pile becomes fully mobilised. Pile capacity may also be limited by the sectional detail and properties.
6. Safe working load shall be calculated by modifying the ultimate bearing capacity to eliminate the effect of downdrag forces and dividing by a suitable factor of safety.
7. The cover to all steel reinforcement, including links, shall be not less than 30mm unless otherwise approved. Where additional protection is required, approved sleeving shall be provided over an upper specified pile length.

Materials

8. Cement shall be ordinary or rapid hardening Portland cement complying with BS 12 or sulphate-resisting cement complying with BS 4027. Cements other than these shall be to the approval of the Engineer and shall comply with the relevant British Standard.
9. Cement replacement material and the proportion in which it is used shall be to the approval of the Engineer.
10. Aggregates shall comply with BS 882 or BS 1200, as appropriate.
11. Clean water free from acids and other impurities and in accordance with BS 3148 shall be used in the Works.
12. Grout mixes shall be designed to be readily pumpable and to give the specified cube strength.
    The cement content including any replacement material shall be not less than 0.5 of the total aggregate weight per unit volume unless otherwise approved by the Engineer.
13. The workability of grout shall be measured by a suitable flowmeter or other approved means and the method and frequency of monitoring this shall be to the approval of the Engineer.
14. The slump of concrete mixes shall be designed to give the specified cube strength and shall be such as to permit pumping or placing without segregation. They shall comply with the following requirements:
    Piling Mix D Slump 175mm or greater
    Minimum cement content 350kg/m^3
15. The workability of concrete where used shall be measured by the slump test or, where approved, the flow test described in BS 1881 part 105 and shall conform with Piling Mix D requirements throughout.
16. The use of admixtures in concrete or grout shall be stated prior to commencement of piling. Evidence shall be given when required by the Engineer that these admixtures will allow pumping to be carried out successfully, will yield concrete or grout of the required strength and durability and have been used successfully in previous practice.
17. Test cubes shall be prepared and tested in accordance with BS 1881 (See Note 11).
18. All steel shall be in accordance with the appropriate British Standard unless otherwise agreed by the Engineer.

Piles employed in underpinning or adjacent to sensitive structures

19. The selection of drilling equipment shall take into account *inter alia* the necessity to drill through concrete or masonry foundations without causing undue or potentially harmful disturbance and vibration.
20. The bond of piles constructed through existing foundations must be able to transmit the load from the structure to the piles with an adequate Factor of

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Safety. (See Note 3).

21. Pile capacities and spacing shall be appropriate to the condition of the structure to be underpinned (See Note 3).
22. Piles shall be constructed in such a manner as to ensure that no damage is sustained either by previously formed piles or by adjacent structures.
23. Temporary or permanent casing or other approved method of pile bore support shall be used in all piles installed through potentially unstable ground to permit the pile shaft to be formed to its full cross-sectional areas.
24. Care shall be exercised to prevent loss of ground from below existing adjacent structures and to avoid ingress of drilling fluid into adjacent basements, sewers or the like.
25. The grout shall be placed through a tremie pipe extending to the bottom of the pile until uncontaminated grout flows from the pile bore.
26. Temporary casing shall be extracted in stages ensuring that, after each length of casing is removed the grout level is brought back up to ground level before the next length is removed. Additional grout shall be placed by the use of a tremie pipe at all times. The tremie pipe shall always extend below the level of existing grout in the pile bore.
27. The grout shall be brought up to the commencing surface level upon completion.
28. The reinforcement shall be placed before temporary casing is extracted. Suitable spacers shall be provided to maintain the specified cover.

Driven piles

29. Piles shall be installed in such a sequence that their construction does not damage any piles already constructed.
30. When required by the Engineer, levels shall be taken to determine the movement of previously installed piles resulting from the driving process. Adequate measures shall be taken to overcome any detrimental effect on the piles from ground heave (See Note 12).
31. Where significant changes of driving characteristics are anticipated or observed across a site, additional records of driving resistance over the full length of piles shall be taken as agreed with the Engineer.
32. Where piles are driven to a set, the final set shall normally be recorded either as the penetration in millimetres per 10 blows or as the number of blows required to produce a penetration of 25mm.
33. Where solid core, tube or other steel section piles are installed double corrosion protection techniques shall be employed at the head of the piles and elsewhere where corrosion is a high risk.

General

34. Individual piles shall be constructed within the following normal tolerances (See Note 13).

In plan	75mm in any direction at commencing surface
Vertical	1 in 75
Between vertical and raking up to 1:6	1 in 25
Raking greater than 1:6	1 in 15

35. Any failure of a pile to reach the required depth shall be reported to the Engineer without delay and the reasons shall be stated. If there is any reason to amend pile depths this shall be agreed with the Engineer.
36. The head of each pile shall be trimmed to the specified cut off level as shown on the drawings. Particular care shall be exercised to prevent damage to mini piles during the trimming process or during excavation in their immediate vicinity (See Note 14).
37. The following records shall be kept of every pile:—
    Pile number
    Rake, where appropriate
    Piling platform level related to Ordnance Datum
    Commencing surface level related to Ordnance Datum
    Nominal diameter
    Details of steel reinforcement
    Type of cement and type of any cement replacement material (where used)
    Type and quantity of admixtures (where used)
    Grade, quality and slump of concrete or grout placed
    Date constructed
    Length from commencing surface to toe
    Length of temporary casing
    Length of permanent casing
    Final set or driving resistance, weight and drop of hammer or the equivalent energy per blow.
    Quantity of grout placed
    Details of any obstructions encountered and time taken in overcoming them
    Details of major interruptions to the construction process.

EXPLANATORY NOTES

1. Main applications

Driven mini piles are mainly employed for foundations to domestic dwellings, lightly loaded buildings or structures.

The bored injection grouted mini pile is mainly used for underpinning and in other situations requiring the piles to be installed through foundations, obstructions including steel reinforcement, old grillages, timber etc. Other applications include foundations adjacent to sensitive structures and soil reinforcement.

2. Pile design—general

Pile design should ensure:

2.1 that an adequate Factor of Safety is provided against reaching the ultimate load of the pile (as BS 8004);

2.2 that the required load settlement characteristics are achieved at or near to the design load.

2.3 that even where the Engineer does not carry out the pile design, he should provide all information to enable the pile designer to make due allowance for additional loading. This may be caused by downdrag, resulting from ground and ground water level changes and direct surcharge loads, or tension forces resulting from, inter alia, the swelling of clay soils following the removal of trees from the site or bulk excavation. Design should take account of uplift forces which may be transmitted to piles via pile caps, ground beams or slabs. The deformability of the soil and the relationship of anticipated settlement to time shall be taken into account.

3. Pile design for underpinning

When selecting piles for underpinning special consideration must be given to the transfer of load from the structure to the piles. High concentrations of stress should be avoided, and it is usually desirable to minimise total and differential settlements.

In piled underpinning the structure continues to rest on its original foundation and will only call on the piles to assist if further settlement occurs and then only to the extent induced by that settlement. The pile design should take into account the additional movement of the foundation required to generate the necessary pile reaction.

4. Preliminary test piles and re-assessment of pile design

When possible, and particularly where soil conditions are unfamiliar, a preliminary test pile or test piles should be installed to check the pile design. These should be constructed under the closest supervision in an area where the soil conditions are known, and tested to a specified load of not less than the Design Verification Load plus the Specified Working Load. The information from pile tests or from other data gained during the contract may require the initial pile design to be modified.

5. Proof tests on working piles

Where there is a requirement to test working piles, the test programme should be initiated at an early stage in the contract and the test loads should not normally be greater than the Design Verification Load plus 50% of the Specified Working Load.

6. Testing piles in underpinning

It is not normally practicable to proof test piles when they are constructed through existing foundations. In these circumstances additional test piles should be specified if required.

7. Testing piles installed through obstructions

In cases where cast-in-place piles installed through obstructions, made ground, large boulders etc., are required to be tested, they should be doubly sleeved to below the obstruction or made ground to minimise friction and prevent misleading results.

8. Data requirements

When piles are specified by the Engineer, the specialist piling contractor will require the following information — site investigation, drawings showing layout, levels, and borehole positions, loads, pile details and other particular requirements.

When the specialist piling contractor is required to design the piles, or when alternatives are permitted, the enquiry documents should in addition state the required minimum design Factor of Safety, and the acceptable settlement of individual piles under test.

9. In-situ soil testing

Where in-situ soil testing is a contractual requirement, the type and anticipated number of tests should be indicated in the enquiry document and included as a measured item in the bill of quantities.

10. Loads

Loads supplied for the purpose of pile design should be 'service loads' and not ultimate loads as used in limit state design practice.

11. Concrete or grout test cubes

Opinions vary as to the number of test cubes which should be required on a piling contract but it is suggested that a set of four cubes be taken for every ten piles installed.

The anticipated number of test cubes should always be included as a measured item in the bill of quantities.

12. Heave of driven piles

The acceptable amount of heave depends upon whether the piles are designed to carry the majority of their load by shaft friction or by end bearing. Heave due to pile driving has generally a minor effect on an adequately reinforced pile which carries its load mainly by shaft friction or on an end bearing pile which had not been unseated. Where an end bearing pile has been unseated by heave, the effects of heave may be reduced by pre-boring or alternatively the contractor may elect to re-drive piles where this is a practicable solution.

The particular measures required will vary with each site and those adopted should be a matter for discussion and agreement with the specialist piling contractor concerned.

13. Positional and alignment tolerances

Tolerances should be related to the construction conditions and the design should take this into account.

The tolerances quoted are realistic for most sites but where the ground contains obstructions, tolerances may have to be increased and appropriate allowances made in the design of piles, caps and ground beams. The need for close tolerances generally diminishes with capped pile groups having three or more piles and it is more important to achieve alignment near the head of the pile than near the pile toe.

Tolerances should be further considered in the light of specific requirements for work in marine and shoreline conditions.

14. Trimming of piles

The trimming of pile heads to a final cut-off level and the cutting of steel to required projections are normally carried out by others and do not fall within the scope of the specialist piling contractor's work. Clause 36 of this specification should be included in the general contract specification, where appropriate.

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1987

The Commonwealth of Massachusetts

Executive Office of Public Safety

STATE BOARD OF BUILDING REGULATIONS AND STANDARDS
One Ashburton Place - Room 1301
Boston, Massachusetts 02108
(617) 727-3200

CODE REVISION PROPOSAL FORM

(please type or print)

TE: 21 September 1988

ilding Code Section No. 742.0, 743.0 Code Change No. ____

oponent (NAME) Geotechnical Advisory Committee, S.B.B.R.S.

DDRESS) One Ashburton Place, Rm. 1301, Boston, MA 02108

lease check type of amendment proposed.)

X Change Section as follows: ____ Delete Section and substitute as follows:

____ Add new Section as follows: ____ Delete Section - no substitute.

OPOSED CODE AMENDMENT: 1) 742.0 SPECIAL PILES AND CAISSONS; change to SECTION 743.0

!) 743.0 LATERAL SUPPORT; change to SECTION 744.0

3) Insert new SECTION 742.0 SMALL DIAMETER GROUTED PILES (6 pages attached).

IPPORTING STATEMENT(S): The proposed changes cover design and construction procedures which are considered current state of the art. Draft copies have been circulated to the local engineering community and reviewed at a forum sponsored by BSCES on 14 June 1988.

The provisions were unanimously voted by the Geotechnical Advisory Committee on 14 September 1988.

SECTION 742.0 SMALL DIAMETER GROUTED PILES

742.1 General: This section covers grouted cast-in-place piles which are less than twelve (12) inches in diameter and in which all or a portion of the pile is cast directly against the soil without permanent casing.

742.2 Installation: The pile may be formed in a hole advanced by rotary or rotary percussive drilling methods (with or without temporary casing), by a hollow-stem auger, or by driving a temporary casing. The pile shall be grouted with a fluid cement grout. The grout shall be pumped through a tremie pipe extending to the bottom of the pile until grout of suitable quality returns at the top of the pile.

742.2.1 Piles grouted with temporary casing: For piles grouted inside a temporary casing, the reinforcing steel shall be inserted prior to withdrawal of the casing. The casing shall be withdrawn in a controlled manner with the grout level maintained at the top of the pile, to insure that the grout completely fills the drill hole. During withdrawal of the casing, the grout level inside the casing shall be monitored to check that the flow of grout inside the casing is not obstructed.

742.2.2 Piles grouted without temporary casing: For a pile or portion of a pile grouted in an open drill hole in soil without temporary casing, the minimum design diameter of the drill hole shall be verified by a suitable device immediately prior to grouting. The reinforcing steel shall be inserted prior to grouting.

742.2.3 Piles grouted with hollow-stem augers: For piles installed with a hollow-stem auger, the grout shall be pumped under continuous pressure, and the rate of withdrawal of the auger shall be carefully controlled to insure that the hole is completely filled with grout as the auger is withdrawn. The actual volume of grout pumped for each one(1) foot of withdrawal of the auger shall be recorded and must be equal to or greater than the theoretical volume. The reinforcing steel shall be inserted prior to withdrawal of the auger.

742.2.4 Piles designed for end-bearing: For piles designed for end-bearing a suitable means shall be employed to verify that the bearing surface is properly cleaned prior to grouting.

742.2.5 Protection of grouted piles: Subsequent piles shall not be drilled or driven near piles that have been grouted until the grout has had sufficient time to harden.

742.3 Pile diameter: The design pile diameter shall be taken as:

1. The outside diameter of the temporary casing, or

2. The diameter of a full circumferential drill bit attached to the bottom of the temporary casing, or

3) The outside diameter of the hollow stem auger, or

4) The borehole diameter verified by suitable measurements made immediately prior to grouting.

742.4 Allowable design stresses: Except as provided in the fourth paragraph of Section 734.3, the design stresses shall not exceed the following allowable values:

1. For compression loads: The allowable stress on the cement grout shall be thirty-three (33) percent of the twenty-eight (28) day unconfined compressive strength, but not exceeding sixteen hundred (1,600) psi. The allowable stress on the steel

reinforcing, including permanent steel casing, shall be forty (40) percent of the minimum specified yield strength, but not exceeding twenty-four thousand (24,000) psi.

2. For tension loads: The allowable stress on the steel reinforcing shall be sixty (60) percent of the minimum specified yield strength. The allowable stress on the cement grout shall be zero.

742.5 Minimum reinforcing: The steel reinforcing shall be designed to carry the following minimum percentage of the design compression load:

1. For a pile or portion of a pile grouted inside a temporary casing, grouted inside a hole drilled into rock, or grouted with a hollow-stem auger, the reinforcing steel shall be designed to carry not less than forty (40) percent of the design compression load.

2. For a pile or a portion of a pile grouted in an open drill hole without temporary or permanent casing, the pile shall be designed to carry the entire design compression load on the reinforcing steel. If a steel pipe section is used for reinforcing, any portion of the cement grout enclosed within the pipe may also be included at the allowable stress for the grout.

742.6 Corrosion protection:

1. Minimum grout cover: Where steel reinforcing is not enclosed inside a permanent casing, centralizers shall be provided on the reinforcing to insure a minimum grout cover of one (1) inch in soil and one half (1/2) inch in rock. Grout cover requirements may be reduced when the reinforcing steel is provided with a suitable protective coating.

2. Permanent steel casing that is used as structural reinforcing shall be protected in accordance with the provisions of Section 733.3.

3. For piles subjected to sustained tension loading in corrosive environments, the reinforcing steel shall be protected by a suitable protective coating or encapsulation method.

742.7 Allowable load: The load on small diameter grouted piles shall not exceed the allowable load computed on the basis of the allowable stresses given in Section 742.4 and minimum reinforcing requirements given in Section 742.5, nor shall the load exceed the allowable load determined by load test in accordance with section 722.8. Load tests may be waived by the building official based on substantiating data and analyses prepared by a registered professional engineer.

742.8 Alternative load test procedure for friction piles: For piles designed as friction piles, the friction capacity in compression may be verified by load testing in tension. The tension load test shall be performed in accordance with Section 722.8.2, with the following exceptions:

1. The test pile must be cased or left ungrouted down to the top of the bearing stratum in a manner which will insure that no friction resistance is developed above the bearing stratum.

2. The maximum design load shall be taken as fifty (50) percent of the applied test load which results in a movement under load of one half (1/2) inch at the pile tip. The movement at the pile tip shall be a) measured directly by a tell-tale or b) computed by deducting the theoretical elastic elongation of the pile from the displacement measured at the top of the pile.

742.9 Records: The owner shall engage a registered professional engineer to observe the installation of the piles in accordance with 732.10. The engineer or his representative shall make an accurate record of the installation equipment used, pile dimensions, grouting volumes and procedures used and all other pertinent installation data.

722.8 Requirements for Pile Load Tests:

722.8.1 Compression Load Test:

722.8.1.1 Required Test Load: A single pile shall be load-tested to not less than twice the allowable design load. When two (2) or more piles are to be tested as a group, the total load shall be not less than one and one-half (1 1/2) times the allowable design load for the group.

In no case should the load reaching the bearing stratum for a single pile or pile group be less than the following:

Case A - piles designed as end-bearing piles, 100% of the allowable design load.

Case B - piles designed as friction piles, 150% of the allowable design load.

For piles designed as combination end-bearing and friction piles, Case A applies if the pile is designed to support more than fifty (50) percent of its design in end bearing; otherwise, Case B applies.

722.8.1.2 Internal Instrumentation: The test pile shall be instrumented in accordance with the requirements in paragraph 4.4.1 of ASTM D1143 to enable measurement or computation of the load in the pile where it enters the bearing stratum. For piles containing concrete, instrumentation shall be installed in the test pile to permit direct measurement of the elastic modulus of the pile.

This requirement is waived for the following cases:

1.) The test pile is installed within a casing that extends to within 10 ft above the bearing stratum.

2.) The pile to be tested has been functioning satisfactorily under load for a period of one year or more.
3.) The pile is 30 ft long or less and no appreciable load will be supported above the bearing stratum.

722.8.1.3 Loading Procedure: Pile load tests shall be conducted in accordance with ASTM D1143, Standard Method of Testing Piles under Static Axial Compressive Load. except that Section 5 Loading Procedures shall be deleted and replaced by the following provisions:

1) Apply 25% of the allowable design load every one-half hour. Longer time increments may be used, but each time increment should be the same.
2) At 200% of the allowable design load (or 150% for pile groups), maintain the load for a minimum of one hour and until the settlement (measured at the lowest point on the pile at which measurements are made) over a one-hour period is not greater than 0.01 in.
3) Remove 50% of the design load every 15 minutes until zero load is reached. Longer time increments may be used, but each should be the same.
4) Measure rebound at zero load for a minimum of one hour.

In no case shall a load be changed if the rate of settlement is not decreasing with time. For each load increment or decrement, take readings at the top of the pile and on the internal instrumentation at 1, 2, 4, 8 and 15 minutes and at 15 minute intervals thereafter.

A load greater than 200% of the allowable design load (or 150% of the allowable design load for pile groups) may be applied at the top of the pile, using the above loading procedure, to ensure that Section 722.8.1 is fulfilled.
Other optional methods listed in ASTM D1143 may be approved by the Building Official upon submittal in advance of satisfactory justification prepared by a registered professional engineer who is qualified in this field.

722.8.1.4 Selection of Design Load: Provided that the allowable design load does not exceed the load allowed in this section for the type of pile and provided that the allowable design load does not exceed 100% of the load supported in the bearing stratum (or 2/3 of the load upported in the bearing stratum for friction piles) when the maximum test load is applied. then the allowable design load shall be the greater of the following:

1) Allowable Design Load Based on Settlement During Loading:
Fifty (50) percent of the applied test load which causes a gross settlement at the pile cutoff grade equal to the sum of: a) the theoretical elastic compression of the pile in inches, assuming all the load on the butt is transmitted to the tip. plus b) 0.15 inch. plus c) one (1) percent of the pile tip diameter or pile width in inches. If the settlements are so small that the load-settlement curve does not intersect the failure criterion, the maximum test load shall be taken as the failure load and used to compute the allowable design load.

2) Allowable Design Load Based on Net Settlement After Rebound:
Fifty (50) percent of the applied test load which results in a net settlement at the top of the pile of 1/2 inch, after rebound for a minimum of one hour at zero load

722.8.2 Tension Load Test:

722.8.2.1 Required Load Test: A single pile or a pile group shall be load tested to not less than 200 percent of the design load for transient loads (i.e. earthquake and wind) and 250 percent for sustained loads.

722.8.2.2 Test Setup and Loading Procedure: The load test setup, instrumentation, and loading procedure shall be in accordance with ASTM D3689-83.

722.8.2.3 Selection of Design Load: Provided the allowable design load does not exceed the allowable stresses in the pile materials, the allowable design load shall be the lower of the following:

1) Fifty (50) percent (for transient loads) or forty (40) percent (for sustained loads) of the applied test load which results in a net upward movement of one-half (1/2) inch at the top of pile after removal of the maximum test load. (The gross upward movement minus the rebound movement).
2) Fifty (50) percent (for transient loads) or forty (40) percent (for sustained loads) of the applied test load which results in continuous upward movement with no increase in load.

722.8.3 Lateral Load Test:

722.8.3.1 Required Test Load: A single pile shall be load tested to not less than 200 percent of the design load.

722.8.3.2 Test Setup and Loading Procedure: The load test setup, instrumentation, and loading procedure shall be in accordance with ASTM D3966-81.

722.8.3.3 Selection of Design Load: The design load shall be selected by the responsible registered professional engineer, based upon his interpretation of the load deflection data from the load test.

722.9 Application of pile load test results: The results of the load test can be applied to other piles within the area of substantially similar subsoil conditions as that for the test pile: and providing the performance of the test pile has been satisfactory and the remaining piles are of the same type, shape and size as the test pile: and are installed using the same methods and equipment and are driven into the same bearing strata as the load-tested pile to an equal or greater penetration resistance.

722.10 Settlement analysis: Whenever a structure is to be supported by medium or soft clay (materials of Classes 12 and 13) or other materials which may be subject to settlement or consolidation, the settlements of the structure and of neighboring structures due to consolidation shall be given careful consideration, particularly if the subsurface material or the loading is subject to extensive variation. The building official may require a settlement analysis to be made by a registered professional engineer in case the live and dead loads of the structure, as specified in this article, minus the weight of the excavated material, induce a maximum stress greater than three hundred (300) pounds per square foot at mid-depth of the underlying soft clay layer.

722.11 Settlement analysis computations: Settlement analyses will be based on a computation of the new increase in stress that will be induced by the structure and realistically appraised live loads, after deducting the weight of excavated material under which the soil was fully consolidated. The effects of fill loads within the building area or fill and other loads adjacent to the building shall be included in the settlement analysis. The appraisal of the live loads may be based on surveys of actual live loads of existing buildings with similar occupancy. The soil compressibility shall be determined by a registered professional engineer and approved by the building official.

	Small diameter injection piles (cast-in-place concrete piles and composite piles) Construction procedure, design and permissible loading	DIN 4128

Verpresspfähle (Ortbeton- und Verbundpfähle) mit kleinem Durchmesser; Herstellung, Bemessung und zulässige Belastung

As it is current practice in standards published by the International Organization for Standardization (ISO), the comma has been used throughout as a decimal marker.

This standard is the outcome of many years of consultation within a joint committee of section *Baugrund* of the *Normenausschuss Bauwesen* (Building Standards Committee) of *DIN Deutsches Institut für Normung e. V.* and the *Deutsche Gesellschaft für Erd- und Grundbau* (German Society for Earthworks and Foundation Engineering). It has been recommended to the Laender building inspectorates by the *Institut für Bautechnik* (Institute for Building Technology), Berlin, for inclusion in the official approval procedure.

The term "load" is used for forces acting on a system from outside; the same applies to terms which include the word "load" (see DIN 1080 Part 1).

Because the planning, design and execution of injection piles call for sound knowledge of the method of construction and wide experience, only those contractors and engineers may be entrusted therewith who meet these requirements and ensure expert construction. Only such persons as possess a thorough knowledge of the method of construction and its execution may be appointed as the responsible contractor's agent. Supervision of the work may be exercised only by trained foremen drillers, site foremen or gangers who have already successfully constructed injection piles. Sufficient time shall be allowed for preparing piles.

See DIN 4014 Part 1 for bored piles of conventional type.

Contents

Continued on pages 2 to 7

1 Field of application and scope

This standard serves as a basis for the planning and execution, and for the assessment of the bearing capacity of non-prestressed injection piles (cast-in-place concrete piles and composite piles) with shaft diameters of less than 300 mm with circular shaft cross sections or comparable similar cross-sectional shapes. The required minimum shaft diameters are 150 mm for cast-in-place concrete piles and 100 mm for composite piles. Piles are formed vertically or raked and are usually axially stressed. Piles transmit their loads into the ground by skin friction, unless the pile rests on rock.

Injection piles are used for temporary purposes, normally for not more than 2 years, and for permanent purposes.

The general principles of pile foundations specified in DIN 1054, November 1976 edition, clause 5, shall apply.

2 Designation

The designation of an injection pile (V) shall read:

Injection pile DIN 4128 – V

3 Concepts

3.1 Injection pile

An injection pile is a cast-in-place concrete pile or a composite pile in which the transmission of stress to the surrounding ground is achieved by the injection of concrete or cement mortar.

A cast-in-place concrete pile has a longitudinal reinforcement of reinforcing steel running its whole length. It may be formed of concrete as specified in DIN 1045 or of cement mortar.

A composite pile has a prefabricated load bearing member of reinforced concrete or steel running its whole length. The load bearing member is either placed in a cavity in the ground or inserted into the soil with the aid of a foot, larger than the load bearing member, e. g. in the form of an impact-driven injection pile. In this case, the cavity may already be filled before insertion of the load bearing member. In the process, the grout surrounds the load bearing member over its entire length in the ground. The stress is transmitted through the bond from the load bearing member into the grout along the whole or part of the pile length.

3.2 Internal and external bearing capacity

The internal bearing capacity of an injection pile is determined by the failure of the materials of which the pile is made.

The external bearing capacity of an injection pile is determined by the failure of the ground supporting the pile.

3.3 Stress transmitting length

The stress transmitting length is that length of the pile shaft through which the pile stress is transmitted into the ground.

3.4 Diameter of pile shaft

The diameter of the pile shaft is the greatest outside diameter of the drilling tool, casing pipe, casing or driving shoe. In the case of piling with external flushing, it may be assumed that the diameter of the pile shaft is equal to the outside diameter of the casing pipe plus 20 mm.

3.5 Injection, re-injection

Injection is a method in which grout is placed at a pressure greater than the hydrostatic pressure. Pressure can be applied to the grout by atmospheric pressure or by liquid pressure.

Re-injection is a method in which a single or repeated injection is effected after the first injection has set or hardened.

4 Soil investigation

Before piles are formed, the sequence of strata, the condition of the soil and the groundwater circumstances shall be investigated as described in DIN 1054, November 1976 edition, clause 3, to an adequate depth below the pile base.

In non-cohesive soils, the strength properties shall be determined by soundings (static soundings, standard soundings, drop-penetration soundings or lateral pressure soundings) and the particle size distribution of the individual soil strata ascertained.

In cohesive soils, the particle size distribution, the consistency index, the uniaxial compressive strength and the shear strength of the individual soil strata shall be determined.

In rock and rocky soils, those methods of exploration and investigation shall be selected which, in addition to determining the sequence of strata, the types of rock and the strength of the rock, also permit conclusions to be drawn regarding formation strength, permeability to water and sensitivity to water.

Groundwater and subsoil shall be examined for properties harmful to concrete and other building materials including, where appropriate, those which could affect the mechanical properties of a supporting liquid.

5 Investigation of existing structures

At an early stage of plannning, where work on or in the vicinity of existing structures is involved, investigations shall be made into their depth, the width and height of their foundations and the method of construction of the building materials used and their strength. If no stability computations are available, the loads shall be determined. In this connection, particular attention shall be paid to horizontal loads.

When selecting the method of formation of injection piles, the structural condition, with particular reference to sensitivity to deformation and vibration, shall be taken into account.

6 Design of individual piles

6.1 Cast-in-place concrete piles

Cast-in-place concrete piles shall be reinforced as specified in DIN 1045. Table 1 applies to the concrete cover of the reinforcement. Where these minimum dimensions are not ensured by the method of manufacture, bar spacers shall be fitted as a centre guide; they shall continue to provide adequate cover after withdrawal of the casing or casing pipe, e. g. spring cage bar spacers. Especially in the case of piles inclined at more than 15° to the vertical, bar spacers shall be so constructed that there is no danger of the reinforcement cage sinking down into the freshly mixed concrete.

6.2 Composite pile

The load bearing member of a composite pile shall be located centrally, if necessary with the aid of suitable bar spacers (see subclause 6.1) and shall extend over the whole length of the pile.

DIN 1045 shall be observed regarding the design of load bearing members of reinforced concrete.

In the case of steel load bearing members, the cross section may be in the form of a solid round bar, a pipe or sectional steel. It shall be formed as specified in DIN 1050. In addition to the steels listed therein, other steels approved by the building inspectorates, with a nominal tensile yield point of up to 500 N/mm^2, may be used. Steel load bearing members shall be protected against corrosion over their whole length. Table 1 applies to the concrete cover of load bearing members.

6.3 Measures against aggressive soil or groundwater

Should the soil or groundwater attack concrete within the meaning of DIN 4030 or be aggressive within the meaning of DVGW-Datenblatt (Data Sheet) GW 9, the contents of table 1 shall be observed.

7 Formation of cavity and injection

7.1 Formation of cavity

Boring methods, driving methods and vibratory methods are suitable for producing the cavity for an injection pile. Internal or external flushing may be used to transport drill cuttings. Loosening of the soil by flushing methods only is not permissible. A stable cavity of the intended geometrical cross section shall be created over the entire length. The method of producing it shall be adapted to the subsoil involved.

The spacing between the axes of piles in the area of the stress transmitting lengths shall be not less than 0,80 m. This minimum axial spacing may be reduced if damage to adjacent piles can be excluded during their formation.

Cavities inclined at more than 15° to the vertical may be formed only with sufficiently rigid casings or casing pipes.

Piles shall not be inclined at more than 80° to the vertical.

Cavities shall be checked for position, length and inclination.

Where adequately rigid casings or casing pipes are used, the inclination can normally be measured at the uppermost casing or casing pipe.

Checking of the inclination is especially important in the case of groups of piles.

By way of departure from DIN 4014 Part 1, the joints of the casing pipe may be formed with internal nipples

Table 1. Minimum dimensions of concrete cover of reinforcement or of the steel load bearing member

Line	Degree of aggressiveness: Attack on concrete in accordance with DIN 4030	Degree of aggressiveness: Permissible aggressiveness to steel in accordance with DVGW-Datenblatt (Data sheet) GW 9	Concrete cover 1), 5) in mm
1	not aggressive	aggressive, slightly aggressive or barely aggressive 4)	30
2	not aggressive, but with a sulfate content classified in DIN 4030 as slightly aggressive		30 2)
3	slightly aggressive		35 3)
4	very aggressive		45 3)

1) The figures apply to concrete; where cement mortar is used, they may be reduced by 10 mm.

2) An HS cement shall be used for forming the pile shaft.

3) Piles may be inserted only if an expert in matters of corrosion of steel and concrete confirms that the long-term load bearing behaviour is not affected by a time-dependent reduction of the skin friction. In the zone outside the stress transmitting length, other protective measures, in place of an increase in the concrete cover, may be taken (see DIN 1045, December 1978 edition, subclause 13.3), but the concrete cover shall at least conform to table 1, line 1.

4) In the case of injection piles for temporary purposes, piles may also be formed in soils which are strongly aggressive to steel provided that it can be shown by an expert that the load bearing behaviour is not impaired.

5) In the case of piles for temporary purposes, the figures may be reduced by 10 mm.

(internal sockets) provided that, when the casing pipe is withdrawn, there can be no possibility of damage to the reinforcement or to the load bearing member.

If drilling takes place below groundwater level, overpressure of the flushing or supporting liquids shall be used to prevent soil from entering the cavity.

The borehole shall be cleared of residue from drilling.

7.2 Injection

Concrete or cement mortar are used as grout. To form the shaft, an injection pressure shall be applied which should be not less than 5 bar in the vicinity of the stress transmitting length.

The concrete shall be made up as specified in DIN 1045, December 1978 edition, subclause 6.5.2. Contrary thereto, the minimum cement content of the concrete shall be 500 kg/m^3. The concrete shall conform at least to strength category B 25. Contrary to DIN 1045, December 1978 edition, subclause 6.5.7.1, concrete injected by the tremie method, see DIN 4126 Part 1 (at present at the stage of draft), shall be produced in the conditions applicable to concrete B I. If piles are injected in injectable or fissured ground, the proportion of cement shall be correspondingly increased.

The maximum particle size of the aggregate shall not be greater than half of the concrete cover or of the clear space between reinforcing bars. In the case of piles with a shaft diameter less than 200 mm, the diameter of the maximum particle size of the aggregate shall not exceed 8 mm.

Where cement mortar is used, materials conforming to DIN 1045, December 1978 edition, clause 6, shall be employed. By way of departure from DIN 1045, December 1978 edition, subclause 2.1.3.1, the addition of a concrete aggregate may be dispensed with. The compressive strength of the cement mortar shall conform at least to concrete strength category B 25.

Injection operations in the stress transmitting length shall take place immediately following completion of the cavity. Placing of the grout shall start at the bottom and proceed continuously upwards.

Grout shall be fed in by pumps through pipes, hose or the drill pipe. On withdrawal, its discharge opening shall terminate not less than 3,0 m in the grout.

7.3 Re-injection

Re-injection is always necessary if the cavity has not been injected as specified in subclause 7.2, paragraph 1. The injection equipment should be located symmetrically in the cross section of the pile. The re-injection material, pressures and quantities shall be adapted to the type of subsoil and to local conditions. The re-injection material shall be of such a composition that voids are filled up again.

Piles under load shall not be re-injected.

7.4 Record of formation

Records shall be kept for all injection piles during their formation. For this purpose, the information corresponding to the type of pile used shall be taken from Appendix A.

8 Quality testing

DIN 1045, December 1978 edition, clause 7, applies to the verification of the quality of building materials. Where cement mortar is used, then, contrary to what is stated in DIN 1045, verification of the compressive strength shall be provided as specified in DIN 4227 Part 5. For quality tests, however, 2 sets of 3 cylinders shall be produced and tested on each of 7 working days or for each building site.

9 Design and verification

9.1 Verification of external bearing capacity

The stress transmitting length of injection piles shall be located in adequately firm subsoil, as described in DIN 4014 Part 1, August 1975 edition, subclause 13.1, and shall be not less than 3 m. In rock or soils similar to rock, the stress transmitting length may be appropriately reduced. However, it shall be not less than 0,5 m.

In the case of pressure piles, the thickness of the soil stratum shall be checked to ensure that lower lying strata cannot give rise to harmful settlements as a result of the pile loading.

Where existing foundations are being strengthened, then by way of departure from DIN 1054, November 1976 edition, subclause 5.2.1, injection piles may also be used to transmit partial loads, provided that the compatibility of the deformation behaviour of the foundation elements involved is taken into account.

The permissible pile loading shall be specified on the basis of trial loadings which shall be carried out as described in DIN 1054, November 1976 edition, subclause 5.8 and appendix. Trial loadings shall be carried out on not less than 2 piles, but on at least 3 % of all piles. Tests shall be carried out at those points where, on the basis of soil investigations, the least favourable soil profile for the bearing capacity of the piles can be expected, unless appropriate trial loadings are carried out for each characteristic profile.

When carrying out the trial loading, skin friction shall be eliminated by constructive measures in sectors which are excavated in the working condition of the pile foundation. In those cases in which it is necessary that the pile

Table 2. Safety coefficients η for injection piles

Injection piles used as		η for loading case as defined in DIN 1054: 1	2	3
pressure piles		2,0	1,75	1,5
tension piles with	0 to 45° deviation from the vertical	2,0	1,75	1,5
	80° deviation from the vertical	3,0	2,5	2,0
For tension piles, values between 45 and 80° shall be interpolated.				

5

loads be transmitted to a limited length of the shaft, constructive measures shall be taken during the trial loading to prevent the stress from being transmitted into other sections. In exceptional cases, this proportion of the skin friction may be estimated by calculation.

If structural piles are used as test piles, then it shall be shown that their bearing capacity does not suffer as a result of the test loading.

The safety level shall conform at least to the figures in table 2. The lowest trial value shall be taken for determining the permissible pile loading.

Note. As specified in DIN 1054, November 1976 edition, table 8 and to the *Empfehlungen des Arbeitsausschusses "Ufereinfassungen"* (Recommendations of Technical Committee Bank Surrounds) EAU 1980 – E 26, the safety coefficients η are, in the case of tension piles, for static and geometrical reasons, reduced as the deviation of the tension pile from the vertical increases. In contrast thereto, in this standard, the safety coefficients for injection piles used as tension piles are increased as the deviation from the vertical becomes greater, because of the more extensive utilization made in such cases of skin friction. The intention is to achieve, in comparison with injection anchors as described in DIN 4125 Part 1 and Part 2, a comparable level of safety despite the abandonment of the principle of an acceptance test.

The results of tensile tests on injection piles may be used for assessing the bearing capacity of pressure piles provided that the pile stresses are mainly transmitted into the ground by skin friction over the entire length.

Trial loadings need not be carried out if the results of trial loadings in comparable circumstances are available.

If, in exceptional cases, no trial loadings are carried out, the limit skin friction values given in table 3 may be applied. Point bearing shall not additionally be taken into account.

Table 3. Limit skin friction values for injection piles

Type of soil	Pressure piles MN/m²	Tension piles MN/m²
Medium gravel and coarse gravel	0,20	0,10
Sand and gravelly sand	0,15	0,08
Cohesive soil	0,10	0,05

The permissible skin friction values are obtained by dividing the limit skin friction values given in table 3 by the safety coefficient η shown in table 2.

For individual piles of a total length up to 10 m and without freestanding parts, axial displacements of the pile head of up to 10 mm must be expected under the permissible loading. These figures include elastic and plastic deformations.

9.2 Internal bearing capacity

Cast-in-place piles made with concrete shall be designed as specified in DIN 1045. Evidence of serviceability shall be provided for the design of composite piles and of cast-in-place concrete piles which do not conform to DIN 1045 (e.g. injection piles formed with cement mortar).

In the case of cast-in-place concrete piles, evidence of limitation of the crack width, as described in DIN 1045, December 1978 edition, subclause 17.6.2, shall be provided for a "very small" expected crack width.

If, in the case of composite piles, concrete or cement mortar is used for protection against corrosion, the appropriate procedure shall be adopted.

Other measures shall be regarded as effective protection against corrosion only if it is shown that, for the stresses in question, they provide long-term protection equivalent to that of concrete or cement mortar.

9.3 Evidence of safety against buckling

If the shear strength of an undrained cohesive soil, as defined in DIN 18 137 Part 1 or DIN 4096 is less than 10 kN/m², then, in addition to the specifications of DIN 1054, November 1976 edition, subclause 5.2.10, evidence of safety against buckling shall be provided for a bar which is not laterally supported.

9.4 Bending stress

In addition to the specifications of DIN 1054, November 1976 edition, subclause 5.3.3, the bending stress caused by lateral pressure shall be taken into account in accordance with the DGEG recommendations *"Seitendruck auf Pfähle durch Bewegung von weichen bindigen Böden* (Lateral pressure on piles due to movement of soft cohesive soils)".

In order to avoid bending stresses on individual piles caused by unintentional eccentric loading, piles shall be so arranged that such eccentricities can be regarded as harmless to the individual pile.

Note. This implies, for example, the arrangement of at least three piles under a point load or of two rows of piles under a linear load, unless other structural measures are adopted.

9.5 Stability and deformation behaviour of the system as a whole

Evidence shall be provided of the stability and deformation behaviour of the system as a whole. In so doing, the procedure set out in DIN 1054, November 1976 edition, subclauses 5.2.3 and 5.3, shall be followed as appropriate.

Where injection piles are used for purposes of anchoring, DIN 4125 Part 1, June 1972 edition, subclauses 5.6 to 5.8, shall be taken into account as appropriate.

In the case of repeated unidirectional loading and of alternating stresses, the unidirectional and alternating loading in the working state shall be simulated, in addition to the usual trial loading for determining the breaking load. The number of load reversals in the test shall be sufficient to enable an estimate to be made of the dying down to zero of the increase in deformation. In the case of repeated unidirectional loading, this test may be dispensed with if the unidirectional loading is less than 50 % of the working load.

Appendix A

Statement of the record of the formation of injection piles as specified in DIN 4128

1 General information relating to the building project
1.1 Company
1.2 Site
1.3 Pile plan No.
1.4 Reinforcement plan No.
1.5 Placing plan No.
1.6 Description of the pile system
1.7 Permit No.
1.8 Drilling equipment / pile driver
1.9 Casing pipe outside diameter/inside diameter
1.10 Bit outside diameter / pile shoe dimension
1.11 Internal nipple
1.12 Flushing: air / water / suspension / external / internal
1.13 Placing of grout using hose / pipe / drill pipe
1.14 Injection equipment
1.15 Injection by air / liquid
1.16 Strength category of concrete / cement mortar
1.17 Composition of mixture
1.17.1 Cement: type, strength category, proportion by weight per unit of volume
1.17.2 Aggregate: maximum particle size, proportion by weight per unit of volume
1.17.3 Addition agent: type, proportion by weight referred to the cement weight
1.17.4 Additive: type, proportion by weight per unit of volume
1.17.5 Water / cement ratio
1.17.6 Result of suitability test
1.18 Reinforcement joint: cover / welding / or joint of load bearing member: socket / dowel pin

2 Data relating to the individual pile
2.1 General
2.1.1 Pile number
2.1.2 Pile diameter
2.1.3 Inclination to the vertical
2.1.4 Pile head position relative to the drilling plane and structural zero or absolute
2.1.5 Pile length
2.1.6 No-load length
2.1.7 Stress transmitting length
2.2 Sequence of strata as specified in DIN 4014 Part 1, August 1975 edition, appendix, or driving reports according to DIN 4026, August 1975 edition, appendix
2.3 Pile reinforcement / load bearing member
2.3.1 Length or reinforcing cage / of load bearing member
2.3.2 Top of reinforcing cage / of load bearing member relative to the drilling plane and structural zero or absolute
2.3.3 Number of blows
2.4 Injection, re-injection
2.4.1 Injection pressure in the stress transmitting length (discharge pressure at the pump)
2.4.2 Volume of grout used per valve
2.4.3 Total volume of grout used
2.5 Time needed for
2.5.1 drilling / driving
2.5.2 reinforcing / installation of load bearing member
2.5.3 injecting
2.5.4 re-injecting
2.6 Remarks / special points
2.7 Signatures
2.7.1 Foreman driller / site foreman / ganger
2.7.2 Contractor's agent
2.7.3 Owner's representative
2.8 Date

Standards referred to and other documents

DIN 1045	Concrete and reinforced concrete; design and construction
DIN 1050	Steel in building construction; calculation and constructional design
DIN 1054	Subsoil; permissible loading of subsoil
DIN 1080 Part 1	Terms, symbols and units used in civil engineering; principles
DIN 4014 Part 1	Bored piles of conventional type; manufacture, design and permissible loading
DIN 4026	Driven piles; manufacture, dimensioning and permissible loading
DIN 4030	Evaluation of liquids, soils and gases aggressive to concrete
DIN 4096	Subsoil; vane testing, dimensions of apparatus, mode of operation, evaluation
DIN 4125 Part 1	Earth and rock anchors; injection anchors for temporary purposes in loose rock; design, execution and testing
DIN 4125 Part 2	Earth and rock anchors; injection anchors for permanent anchorages (permanent anchors) in soil; design, execution and testing
DIN 4126 Part 1	(at present at the stage of draft) Mole walls; cast-in-place mole walls; design and execution
DIN 4227 Part 5	Prestressed concrete; injection of cement mortar into prestressing concrete ducts
DIN 18 137 Part 1	(Preliminary standard) Subsoil; examination of soil samples, determination of shear strength, concepts and basic principles of test conditions
EAU (1980)	*Empfehlungen des Arbeitsausschusses "Ufereinfassungen"* of the *Hafenbautechnische Gesellschaft* (Technical Harbour Construction Association) and the *Deutsche Gesellschaft für Erd- und Grundbau* (1981), 6th edition, published by *Ernst & Sohn*
DVGW-Arbeitsblatt GW 9	*Merkblatt für die Beurteilung der Korrosionsgefährdung von Eisen und Stahl im Erdboden* (Data Sheet for the assessment of the risk of corrosion of iron and steel in soil) *ZFGW-Verlag*, Frankfurt am Main

Recommendations of AK 5 of the *Deutsche Gesellschaft für Erd- und Grundbau: Seitendruck auf Pfähle durch Bewegung von weichen bindigen Böden* (Lateral pressure on piles due to movement of soft cohesive soils), *Geotechnik* 1978, pp 100 et seq.

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Preface

THE UNIFORM BUILDING CODE is dedicated to the development of better building construction and greater safety to the public by uniformity in building laws. The code is founded on broad-based performance principles that make possible the use of new materials and new construction systems.

THE UNIFORM BUILDING CODE was first enacted by the International Conference of Building Officials at the Sixth Annual Business Meeting held in Phoenix, Arizona, October 18-21, 1927. Revised editions of this code have been published since that time at approximate three-year intervals. New editions incorporate changes approved since the last edition.

THE UNIFORM BUILDING CODE is designed to be compatible with related publications listed on the following pages to provide a complete set of documents for regulatory use.

Anyone may propose amendments to this code. For more information, write to the International Conference of Building Officials at the address on the copyright page.

Changes in the code are processed each year and published in supplements in a form permitting ready adoption by local communities. These changes are carefully reviewed in public hearings by professional experts in the field of building construction and fire and life safety.

Solid vertical lines in the margins within the body of the code indicate a change from the requirements of the 1988 edition except where the entire chapter was revised, a new chapter was added or the change was minor. Where an entire chapter is changed or new chapter was added, a notation appears at the beginning of that chapter. Vertical lines in the margins that are interrupted by the letter "F" indicate that the provision is maintained under the code change procedures of the Western Fire Chiefs Association. Deletion indicators (➧) are provided in the margin where a paragraph or item listing has been deleted if the deletion resulted in a change of requirements.

An analysis of changes between editions is published in pamphlet form by the Conference.

Part VI

DETAILED REGULATIONS

Chapter 29

EXCAVATIONS, FOUNDATIONS AND RETAINING WALLS

Scope

Sec. 2901. (a) **General.** This chapter sets forth requirements for excavation and fills for any building or structure and for foundations and retaining structures.

Reference is made to Appendix Chapter 70 for requirements governing excavation, grading and earthwork construction, including fills and embankments.

(b) **Standards of Quality.** The standards listed below labeled a "U.B.C. standard" are also listed in Chapter 60, Part II, and are part of this code.

1. **Testing**

A. U.B.C. Standard No. 29-1, Soils Classification

B. U.B.C. Standard No. 29-2, Expansion Index Test

2. **Design**

A. U.B.C. Standard No. 29-3, Treated Wood Foundation System

B. U.B.C. Standard No. 29-4, Design of Slab-on-grade Foundations to Resist the Effects of Expansive Soils

Quality and Design

Sec. 2902. The quality and design of materials used structurally in excavations, footings and foundations shall conform to the requirements specified in Chapters 23, 24, 25, 26 and 27 of this code.

Excavations and Fills

Sec. 2903. (a) **General.** Excavation or fills for buildings or structures shall be so constructed or protected that they do not endanger life or property.

Slopes for permanent fills shall not be steeper than 2 horizontal to 1 vertical. Cut slopes for permanent excavations shall not be steeper than 2 horizontal to 1 vertical unless substantiating data justifying steeper cut slopes are submitted. Deviation from the foregoing limitations for cut slopes shall be permitted only upon the presentation of a soil investigation report acceptable to the building official.

No fill or other surcharge loads shall be placed adjacent to any building or structure unless such building or structure is capable of withstanding the additional loads caused by the fill or surcharge.

Existing footings or foundations which may be affected by any excavation shall be underpinned adequately or otherwise protected against settlement and shall be protected against lateral movement.

Fills to be used to support the foundations of any building or structure shall be placed in accordance with accepted engineering practice. A soil investigation report and a report of satisfactory placement of fill, both acceptable to the building official, shall be submitted.

(b) **Protection of Adjoining Property.** The requirements for protection of adjacent property and depth to which protection is required shall be as defined by prevailing law. Where not defined by law, the following shall apply: Any person making or causing an excavation to be made to a depth of 12 feet or less below the grade shall protect the excavation so that the soil of adjoining property will not cave in or settle, but shall not be liable for the expense of underpinning or extending the foundation of buildings on adjoining properties when the excavation is not in excess of 12 feet in depth. Before commencing the excavation, the person making or causing the excavation to be made shall notify in writing the owners of adjoining buildings not less than 10 days before such excavation is to be made that the excavation is to be made and that the adjoining buildings should be protected.

The owners of the adjoining properties shall be given access to the excavation for the purpose of protecting such adjoining buildings.

Any person making or causing an excavation to be made exceeding 12 feet in depth below the grade shall protect the excavation so that the adjoining soil will not cave in or settle and shall extend the foundation of any adjoining buildings below the depth of 12 feet below grade at the expense of the person causing or making the excavation. The owner of the adjoining buildings shall extend the foundation of these buildings to a depth of 12 feet below grade at such owner's expense, as provided in the preceding paragraph.

Soil Classification—Expansive Soil

Sec. 2904. (a) **Soil Classification: General.** For the purposes of this chapter, the definition and classification of soil materials for use in Table No. 29-B shall be according to U.B.C. Standard No. 29-1.

(b) **Expansive Soil.** When the expansive characteristics of a soil are to be determined, the procedures shall be in accordance with U.B.C. Standard No. 29-2 and the soil shall be classified according to Table No. 29-C. Foundations for structures resting on soils with an expansion index greater than 20, as determined by U.B.C. Standard No. 29-2, shall require special design consideration. In the event the soil expansion index varies with depth, the weighted index shall be determined according to Table No. 29-D.

Foundation Investigation

Sec. 2905. (a) **General.** The classification of the soil at each building site shall be determined when required by the building official. The building official may require that this determination be made by an engineer or architect licensed by the state to practice as such.

(b) **Investigation.** The classification shall be based on observation and any necessary tests of the materials disclosed by borings or excavations made in appropriate locations. Additional studies may be necessary to evaluate soil strength, the effect of moisture variation on soil-bearing capacity, compressibility and expansiveness.

(c) **Reports.** The soil classification and design bearing capacity shall be shown on the plans, unless the foundation conforms to Table No. 29-A. The building official may require submission of a written report of the investigation which shall include, but need not be limited to, the following information:

1. A plot showing the location of all test borings and/or excavations.
2. Descriptions and classifications of the materials encountered.
3. Elevation of the water table, if encountered.
4. Recommendations for foundation type and design criteria, including bearing capacity, provisions to minimize the effects of expansive soils and the effects of adjacent loads.
5. Expected total and differential settlement.

(d) **Expansive Soils.** When expansive soils are present, the building official may require that special provisions be made in the foundation design and construction to safeguard against damage due to this expansiveness. The building official may require a special investigation and report to provide these design and construction criteria.

(e) **Adjacent Loads.** Where footings are placed at varying elevations the effect of adjacent loads shall be included in the foundation design.

(f) **Drainage.** Provisions shall be made for the control and drainage of surface water around buildings. [See also Section 2907 (d) 5.]

Allowable Foundation and Lateral Pressures

Sec. 2906. The allowable foundation and lateral pressures shall not exceed the values set forth in Table No. 29-B unless data to substantiate the use of higher values are submitted. Table No. 29-B may be used for design of foundations on rock or nonexpansive soil for Types II One-hour, II-N and V buildings which do not exceed three stories in height or for structures which have continuous footings having a load of less than 2,000 pounds per lineal foot and isolated footings with loads of less than 50,000 pounds.

Footings

Sec. 2907. (a) **General.** Footings and foundations shall be constructed of masonry, concrete or treated wood in conformance with U.B.C. Standard No. 29-3 and shall extend below the frost line. Footings of concrete and masonry shall be of solid material. Foundations supporting wood shall extend at least 6 inches above the adjacent finish grade. Footings shall have a minimum depth as indicated in Table No. 29-A unless another depth is recommended by a foundation investigation.

The provisions of this section do not apply to building and foundation systems in those areas subject to scour and water pressure by wind and wave action. Buildings and foundations subject to such loads shall be designed in accordance with approved national standards.

(b) **Bearing Walls.** Bearing walls shall be supported on masonry or concrete foundations or piles or other approved foundation system which shall be of sufficient size to support all loads. Where a design is not provided, the minimum foundation requirements for stud bearing walls shall be as set forth in Table No. 29-A.

> EXCEPTIONS: 1. A one-story wood- or metal-frame building not used for human occupancy and not over 400 square feet in floor area may be constructed with walls supported on a wood foundation plate when approved by the building official.
>
> 2. The support of buildings by posts embedded in earth shall be designed as specified in Section 2907 (g). Wood posts or poles embedded in earth shall be pressure treated with an approved preservative. Steel posts or poles shall be protected as specified in Section 2908 (l).

(c) **Stepped Foundations.** Foundations for all buildings where the surface of the ground slopes more than 1 foot in 10 feet shall be level or shall be stepped so that both top and bottom of such foundation are level.

(d) **Footings on or Adjacent to Slopes.** 1. Scope. The placement of buildings and structures on or adjacent to slopes steeper than 3 horizontal to 1 vertical shall be in accordance with this section.

2. **Building clearance from ascending slopes.** In general, buildings below slopes shall be set a sufficient distance from the slope to provide protection from slope drainage, erosion and shallow failures. Except as provided for in Subsection 6 of this section and Figure No. 29-1, the following criteria will be assumed to provide this protection. Where the existing slope is steeper than 1 horizontal to 1 vertical, the toe of the slope shall be assumed to be at the intersection of a horizontal plane drawn from the top of the foundation and a plane drawn tangent to the slope at an angle of 45 degrees to the horizontal. Where a retaining wall is constructed at the toe of the slope, the height of the slope shall be measured from the top of the wall to the top of the slope.

3. **Footing setback from descending slope surface.** Footing on or adjacent to slope surfaces shall be founded in firm material with an embedment and setback from the slope surface sufficient to provide vertical and lateral support for the footing without detrimental settlement. Except as provided for in Subsection 6 of this section and Figure No. 29-1, the following setback is deemed adequate to meet the criteria. Where the slope is steeper than 1 horizontal to 1 vertical, the required setback shall be measured from an imaginary plane 45 degrees to the horizontal, projected upward from the toe of the slope.

4. **Pools.** The setback between pools regulated by this code and slopes shall be equal to one-half the building footing setback distance required by this section. That portion of the pool wall within a horizontal distance of 7 feet from the top of the slope shall be capable of supporting the water in the pool without soil support.

5. **Foundation elevation.** On graded sites, the top of any exterior foundation shall extend above the elevation of the street gutter at point of discharge or the inlet of an approved drainage device a minimum of 12 inches plus 2 percent. The building official may approve alternate elevations, provided it can be demonstrated that required drainage to the point of discharge and away from the structure is provided at all locations on the site.

6. **Alternate setback and clearance.** The building official may approve alternate setbacks and clearances. The building official may require an investigation and recommendation of a qualified engineer to demonstrate that the intent of this section has been satisfied. Such an investigation shall include consideration of material, height of slope, slope gradient, load intensity and erosion characteristics of slope material.

(e) **Footing Design.** Except for special provisions of Section 2909 covering the design of piles, all portions of footings shall be designed in accordance with the structural provisions of this code and shall be designed to minimize differential settlement and the effects of expansive soils when present.

Slab-on-grade and mat-type footings for buildings located on expansive soils may be designed in accordance with the provisions of U.B.C. Standard No. 29-4 or such other engineering design based on geotechnical recommendation as approved by the building official.

(f) **Foundation Plates or Sills.** Foundation plates or sills shall be bolted to the foundation or foundation wall with not less than $^1/_2$-inch nominal diameter steel bolts embedded at least 7 inches into the concrete or masonry and spaced not more than 6 feet apart. There shall be a minimum of two bolts per piece with one bolt located within 12 inches of each end of each piece. A properly sized nut and washer shall be tightened on each bolt to the plate. Foundation plates and sills shall be the kind of wood specified in Section 2516 (c).

(g) **Designs Employing Lateral Bearing.** 1. **General.** Construction employing posts or poles as columns embedded in earth or embedded in concrete footings in the earth may be used to resist both axial and lateral loads. The depth to resist lateral loads shall be determined by means of the design criteria established herein or other methods approved by the building official.

2. **Design criteria. A. Nonconstrained.** The following formula may be used in determining the depth of embedment required to resist lateral loads where no constraint is provided at the ground surface, such as rigid floor or rigid ground surface pavement.

$$d = \frac{A}{2}\left(1 + \sqrt{1 + \frac{4.36h}{A}}\right)$$

WHERE:

$$A = \frac{2.34P}{S_1 b}$$

P = applied lateral force in pounds.

S_1 = allowable lateral soil-bearing pressure as set forth in Table No. 29-B based on a depth of one third the depth of embedment.

S_3 = allowable lateral soil-bearing pressure as set forth in Table No. 29-B based on a depth equal to the depth of embedment.

b = diameter of round post or footing or diagonal dimension of square post or footing (feet).

h = distance in feet from ground surface to point of application of "P."

d = depth of embedment in earth in feet but not over 12 feet for purpose of computing lateral pressure.

B. **Constrained.** The following formula may be used to determine the depth of embedment required to resist lateral loads where constraint is provided at the ground surface, such as a rigid floor or pavement.

$$d^2 = 4.25 \frac{Ph}{S_3 b}$$

C. **Vertical load.** The resistance to vertical loads is determined by the allowable soil-bearing pressure set forth in Table No. 29-B.

3. **Backfill.** The backfill in the annular space around columns not embedded in poured footings shall be by one of the following methods:

A. Backfill shall be of concrete with an ultimate strength of 2,000 pounds per square inch at 28 days. The hole shall not be less than 4 inches larger than the diameter of the column at its bottom or 4 inches larger than the diagonal dimension of a square or rectangular column.

B. Backfill shall be of clean sand. The sand shall be thoroughly compacted by tamping in layers not more than 8 inches in depth.

4. **Limitations.** The design procedure outlined in this subsection shall be subject to the following limitations:

1. The frictional resistance for retaining walls and slabs on silts and clays shall be limited to one-half of the normal force imposed on the soil by the weight of the footing or slab.

Posts embedded in earth shall not be used to provide lateral support for structural or nonstructural materials such as plaster, masonry or concrete unless bracing is provided that develops the limited deflection required.

(h) **Grillage Footings.** When grillage footings of structural steel shapes are used on soils, they shall be completely embedded in concrete with at least 6 inches on the bottom and at least 4 inches at all other points.

(i) **Bleacher Footings.** Footings for open-air seating facilities shall comply with Chapter 29.

> EXCEPTIONS: Temporary open-air portable bleachers as defined in Section 3322 may be supported upon wood sills or steel plates placed directly upon the ground surface, provided soil pressure does not exceed 1,200 pounds per square foot.

Piles—General Requirements

Sec. 2908. (a) **General.** Pile foundations shall be designed and installed on the basis of a foundation investigation as defined in Section 2905 where required by the building official.

The investigation and report provisions of Section 2905 shall be expanded to include but not be limited to the following:

1. Recommended pile types and installed capacities.
2. Driving criteria.
3. Installation procedures.
4. Field inspection and reporting procedures (to include procedures for verification of the installed bearing capacity where required).
5. Pile load test requirements.

The use of piles not specifically mentioned in this chapter shall be permitted, subject to the approval of the building official upon submission of acceptable test data, calculations or other information relating to the properties and load-carrying capacities of such piles.

(b) **Interconnection.** Individual pile caps and caissons of every structure subjected to seismic forces shall be interconnected by ties. Such ties shall be capable of resisting, in tension or compression, a minimum horizontal force equal to 10 percent of the larger column vertical load.

> EXCEPTION: Other approved methods may be used where it can be demonstrated that equivalent restraint can be provided.

(c) **Determination of Allowable Loads.** The allowable axial and lateral loads on piles shall be determined by an approved formula, by load tests or by a foundation investigation.

(d) **Static Load Tests.** When the allowable axial load of a single pile is determined by a load test, one of the following methods shall be used:

Method 1. It shall not exceed 50 percent of the yield point under test load. The yield point shall be defined as that point at which an increase in load produces a disproportionate increase in settlement.

Method 2. It shall not exceed one half of the load which causes a net settlement, after deducting rebound, of 0.01 inch per ton of test load which has been applied for a period of at least 24 hours.

Method 3. It shall not exceed one half of that load under which, during a 40-hour period of continuous load application, no additional settlement takes place.

(e) **Column Action.** All piles standing unbraced in air, water or material not capable of lateral support, shall conform with the applicable column formula as specified in this code. Such piles driven into firm ground may be considered fixed and laterally supported at 5 feet below the ground surface and in soft material at 10 feet below the ground surface unless otherwise prescribed by the building official after a foundation investigation by an approved agency.

(f) **Group Action.** Consideration shall be given to the reduction of allowable pile load when piles are placed in groups. Where soil conditions make such load reductions advisable or necessary, the allowable axial load determined for a single pile shall be reduced by any rational method or formula approved by the building official.

(g) **Piles in Subsiding Areas.** Where piles are driven through subsiding fills or other subsiding strata and derive support from underlying firmer materials, consideration shall be given to the downward frictional forces which may be imposed on the piles by the subsiding upper strata.

Where the influence of subsiding fills is considered as imposing loads on the pile, the allowable stresses specified in this chapter may be increased if satisfactory substantiating data are submitted.

(h) **Jetting.** Jetting shall not be used except where and as specifically permitted by the building official. When used, jetting shall be carried out in such a manner that the carrying capacity of existing piles and structures shall not be impaired. After withdrawal of the jet, piles shall be driven down until the required resistance is obtained.

(i) **Protection of Pile Materials.** Where the boring records of site conditions indicate possible deleterious action on pile materials because of soil constituents, changing water levels or other factors, such materials shall be adequately protected by methods or processes approved by the building official. The effectiveness of such methods or processes for the particular purpose shall have been thoroughly established by satisfactory service records or other evidence which demonstrates the effectiveness of such protective measures.

(j) **Allowable Loads.** The allowable loads based on soil conditions shall be established in accordance with Section 2908.

> **EXCEPTION:** Any uncased cast-in-place pile may be assumed to develop a frictional resistance equal to one sixth of the bearing value of the soil material at minimum depth as set forth in Table No. 29-B but not to exceed 500 pounds per square foot unless a greater value is allowed by the building official after a soil investigation as specified in Section 2905 is submitted. Frictional resistance and bearing resistance shall not be assumed to act simultaneously unless recommended after a foundation investigation as specified in Section 2905.

(k) **Use of Higher Allowable Pile Stresses.** Allowable compressive stresses greater than those specified in Section 2909 shall be permitted when substantiating data justifying such higher stresses are submitted to and approved by the building official. Such substantiating data shall include a foundation investigation including a report in accordance with Section 2908 (a) by a soils engineer defined as a civil engineer experienced and knowledgeable in the practice of soils engineering.

Specific Pile Requirements

Sec. 2909. (a) Round Wood Piles. 1. Material. Except where untreated piles are permitted, wood piles shall be pressure treated. Untreated piles may be used only when it has been established that the cutoff will be below lowest groundwater level assumed to exist during the life of the structure.

2. **Allowable stresses.** The allowable unit stresses for round wood piles shall not exceed those set forth in Table No. 25-E.

The allowable values listed in Table No. 25-E for compression parallel to the grain at extreme fiber in bending are based on load sharing as occurs in a pile cluster. For piles which support their own specific load, a safety factor of 1.25 shall be applied to compression parallel to the grain values and 1.30 to extreme fiber in bending values.

(b) **Uncased Cast-in-place Concrete Piles. 1. Material.** Concrete piles cast in place against earth in drilled or bored holes shall be made in such a manner as to ensure the exclusion of any foreign matter and to secure a full-sized shaft. The

length of such pile shall be limited to not more than 30 times the average diameter. Concrete shall have an ultimate compressive strength f'_c of not less than 2,500 pounds per square inch (psi).

> **EXCEPTION:** The length of pile may exceed 30 times the diameter provided the design and installation of the pile foundation is in accordance with an approved investigation report.

2. **Allowable stresses.** The allowable compressive stress in the concrete shall not exceed $0.33f'_c$. The allowable compressive stress of reinforcement shall not exceed 34 percent of the yield strength of the steel or 25,500 psi.

(c) **Metal-cased Concrete Piles. 1. Material.** All concrete used in metal-cased concrete piles shall have an ultimate compressive strength f'_c of not less than 2,500 psi.

2. **Installation.** Every metal casing for a concrete pile shall have a sealed tip with a diameter of not less than 8 inches.

Concrete piles cast in place in metal shells shall have shells driven for their full length in contact with the surrounding soil and left permanently in place. The shells shall be sufficiently strong to resist collapse and sufficiently watertight to exclude water and foreign material during the placing of concrete.

Piles shall be driven in such order and with such spacing as to ensure against distortion of or injury to piles already in place. No pile shall be driven within four and one-half average pile diameters of a pile filled with concrete less than 24 hours old unless approved by the building official.

3. **Allowable stresses.** Allowable stresses shall not exceed the values specified in Section 2909 (b) 2, except that the allowable concrete stress may be increased to a maximum value of $0.40f'_c$ for that portion of the pile meeting the following conditions:

1. The thickness of the metal casing is not less than No. 14 gauge.
2. The casing is seamless or is provided with seams of equal strength and is of a configuration which will provide confinement to the cast-in-place concrete.
3. The design f'_c shall not exceed 5,000 psi and the ratio of metal yield strength shall not be less than 6.
4. The pile diameter is not greater than 16 inches.

(d) **Precast Concrete Piles. 1. Material.** Precast concrete piles prior to driving and at 28 days after pouring shall develop an ultimate compressive strength f'_c of at least 3,000 psi.

2. **Reinforcement ties.** The longitudinal reinforcement in driven precast concrete piles shall be laterally tied with steel ties or wire spirals. Ties and spirals shall not be spaced more than 3 inches apart, center to center, for a distance of 2 feet from the ends and not more than 8 inches elsewhere. The gauge of ties and spirals shall be as follows:

For piles having a diameter of 16 inches or less, wire shall not be smaller than No. 5 gauge.

For piles having a diameter of more than 16 inches and less than 20 inches, wire shall not be smaller than No. 4 gauge.

For piles having a diameter of 20 inches and larger, wire shall not be smaller than $^1/_4$ inch round or No. 3 gauge.

3. **Allowable stresses.** Precast concrete piling shall be designed to resist stresses induced by handling and driving as well as by loads. The allowable stresses shall not exceed the values specified in Section 2909 (b) 2.

(e) **Precast Prestressed Concrete Piles (Pretensioned).** 1. **Material.** Precast prestressed concrete piles shall develop a compressive strength of not less than 4,000 psi before driving and an ultimate compressive strength f'_c at 28 days after pouring of not less than 5,000 psi.

2. **Reinforcement.** The longitudinal reinforcement shall be high-tensile seven-wire strand. Longitudinal reinforcement shall be laterally tied with steel ties or wire spirals.

Ties or spiral reinforcement shall not be spaced more than 3 inches apart center to center for a distance of 2 feet from the ends and not more than 8 inches elsewhere.

At each end of the pile, the first five ties or spirals shall be spaced 1 inch center to center.

For piles having a diameter of 24 inches or less, wire shall not be smaller than No. 5 gauge. For piles having a diameter greater than 24 inches but less than 36 inches, wire shall not be smaller than No. 4 gauge. For piles having a diameter greater than 36 inches, wire shall not be smaller than $^1/_4$ inch round or No. 3 gauge.

3. **Allowable stresses.** Precast prestressed piling shall be designed to resist stresses induced by handling and driving as well as by loads. The effective prestress in the pile shall not be less than 400 psi for piles up to 30 feet in length, 550 psi for piles up to 50 feet in length, and 700 pounds per square inch for piles greater than 50 feet in length.

The compressive stress in the concrete due to externally applied load shall not exceed:

$$f_c = 0.33f'_c - 0.27fp_c$$

WHERE:

fp_c = effective prestress stress on the gross section.

Effective prestress shall be based on an assumed loss of 30,000 psi in the prestressing steel. The allowable stress in the prestressing steel shall not exceed the values specified in Section 2618.

(f) **Structural Steel Piles.** 1. **Material.** Structural steel piles, steel pipe piles and fully welded steel piles fabricated from plates shall conform to U.B.C. Standard No. 27-1 and be identified in accordance with Section 2701 (b).

2. **Allowable stresses.** The allowable axial stresses shall not exceed 0.35 of the minimum specified yield strength F_y, provided such yield strength shall not be assumed greater than 36,000 psi for computational purposes.

EXCEPTION: When justified in accordance with Section 2908 (k), the allowable stresses may be increased to 0.50 F_y.

3. **Minimum dimensions.** Sections of driven H-piles shall comply with the following:

A. The flange projection shall not exceed 14 times the minimum thickness of metal in either the flange or the web, and the flange widths shall not be less than 80 percent of the depth of the section.

B. The nominal depth in the direction of the web shall not be less than 8 inches.

C. Flanges and webs shall have a minimum nominal thickness of $^3/_8$ inch.

Sections of driven pipe piles shall have an outside diameter of not less than 10 inches and a minimum thickness of not less than $^1/_4$ inch.

(g) **Concrete-filled Steel Pipe Piles.** 1. **Material.** Steel pipe piles shall conform to U.B.C. Standard No. 27-1 and shall be identified in accordance with Section 2701 (b). The concrete-filled steel pipe piles shall have an ultimate compressive strength f'_c of not less than 2,500 psi.

2. **Allowable stresses.** The allowable axial stresses shall not exceed 0.35 of the minimum specified yield strength F_y on the steel plus 0.33 of the ultimate compressive strength f'_c of the concrete, provided F_y shall not be assumed greater than 36,000 psi for computational purposes.

EXCEPTION: When justified in accordance with Section 2908 (k), the allowable stresses may be increased to 0.50 F_y.

3. **Minimum dimensions.** Driven piles of uniform section shall have a nominal outside diameter of not less than 8 inches.

Foundation Construction—Seismic Zones Nos. 3 and 4

Sec. 2910. (a) **General.** In Seismic Zones Nos. 3 and 4 the further requirements of this section shall apply to the design and construction of foundations, foundation components and the connection of superstructure elements thereto.

(b) **Soil Capacity.** The one-third stress increase allowed by Section 2303 (d) may be exceeded for soils for combinations including earthquake when substantiated by geotechnical data. The foundation shall be capable of transmitting the design base shear and overturning forces prescribed in Section 2334 from the structure into the supporting soil. The short-term dynamic nature of the loads may be taken into account in establishing the soil properties.

(c) **Superstructure-to-foundation Connection.** The connection of superstructure elements to the foundation shall be adequate to transmit to the foundation the forces for which the elements were required to be designed.

(d) **Foundation-soil Interface.** For regular buildings, the force F_t may be omitted when determining the overturning moment to be resisted at the foundation-soil interface.

(e) **Special Requirements for Piles and Caissons.** Piles and caissons shall be designed for flexure whenever the tops of such members will be displaced by earthquake motions. The criteria and detailing requirements of Section 2625 (e) for concrete and Section 2710 (e) for steel shall apply for a length of such members equal to 120 percent of the flexural length.

TABLE NO. 29-A—FOUNDATIONS FOR STUD BEARING WALLS—MINIMUM REQUIREMENTS[1 2]

NUMBER OF FLOORS SUPPORTED BY THE FOUNDATION[3]	THICKNESS OF FOUNDATION WALL (inches)		WIDTH OF FOOTING (inches)	THICKNESS OF FOOTING (inches)	DEPTH BELOW UNDISTURBED GROUND SURFACE (inches)
	CONCRETE	UNIT MASONRY			
1	6	6	12	6	12
2	8	8	15	7	18
3	10	10	18	8	24

[1]Where unusual conditions or frost conditions are found, footings and foundations shall be as required in Section 2907 (a).
[2]The ground under the floor may be excavated to the elevation of the top of the footing.
[3]Foundations may support a roof in addition to the stipulated number of floors. Foundations supporting roofs only shall be as required for supporting one floor.

TABLE NO. 29-B—ALLOWABLE FOUNDATION AND LATERAL PRESSURE

CLASS OF MATERIALS[2]	ALLOWABLE FOUNDATION PRESSURE LBS./SQ. FT.[3]	LATERAL BEARING LBS./SQ./FT./FT. OF DEPTH BELOW NATURAL GRADE[4]	LATERAL SLIDING[1]	
			COEF-FICIENT[5]	RESISTANCE LBS./SQ. FT.[6]
1. Massive Crystalline Bedrock	4000	1200	70	
2. Sedimentary and Foliated Rock	2000	400	.35	
3. Sandy Gravel and/or Gravel (GW and GP)	2000	200	.35	
4. Sand, Silty Sand, Clayey Sand, Silty Gravel and Clayey Gravel (SW, SP, SM, SC, GM and GC)	1500	150	.25	
5. Clay, Sandy Clay, Silty Clay and Clayey Silt (CL, ML, MH and CH)	1000[7]	100		130

[1]Lateral bearing and lateral sliding resistance may be combined.
[2]For soil classifications OL, OH and PT (i.e., organic clays and peat), a foundation investigation shall be required.
[3]All values of allowable foundation pressure are for footings having a minimum width of 12 inches and a minimum depth of 12 inches into natural grade. Except as in Footnote No. 7 below, increase of 20 percent allowed for each additional foot of width or depth to a maximum value of three times the designated value.
[4]May be increased the amount of the designated value for each additional foot of depth to a maximum of 15 times the designated value. Isolated poles for uses such as flagpoles or signs and poles used to support buildings which are not adversely affected by a 1/2-inch motion at ground surface due to short-term lateral loads may be designed using lateral bearing values equal to two times the tabulated values.
[5]Coefficient to be multiplied by the dead load.
[6]Lateral sliding resistance value to be multiplied by the contact area. In no case shall the lateral sliding resistance exceed one half the dead load.
[7]No increase for width is allowed.

TABLE NO. 29-C—CLASSIFICATION OF EXPANSIVE SOIL

EXPANSION INDEX	POTENTIAL EXPANSION
0-20	Very low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very high

TABLE NO. 29-D—WEIGHTED EXPANSION INDEX[1]

DEPTH INTERVAL[2]	WEIGHT FACTOR
0-1	0.4
1-2	0.3
2-3	0.2
3-4	0.1
Below 4	0

[1]The weighted expansion index for nonuniform soils is determined by multiplying the expansion index for each depth interval by the weight factor for that interval and summing the products.
[2]Depth in feet below the ground surface.

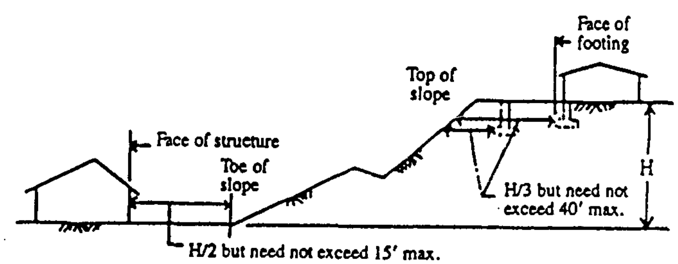

FIGURE NO. 29-1

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