

FACTORS INFLUENCING THE DEVELOPMENT OF MICROPILE CONSTRUCTION TECHNIQUES: A THIRTY YEAR PERSPECTIVE

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ABSTRACT

Although micropiles had been introduced into North American practice via European specialists in the early 1970's, it was more than a decade later when they became more popular and readily accepted in the U.S. Main applications were associated with urban construction and industrial remediations and were originally conducted by U.S. contractors with previous experience in soil and rock anchors. Reflecting that many of these applications related to transportation projects, for example, the seismic retrofits conducted on Interstates in the San Francisco Bay area following the catastrophic Loma Prieta Earthquake of 1989, the Federal Highway Administration (FHWA) in 1993 funded a State-of-Practice Review of micropile technology worldwide. Working in close cooperation with their counterparts in France, and an international advisory board, the Principal Investigators (Drs. Bruce and Juran) wrote a series of four reports from September 1993 to July 1995 subsequently published in July 1997. Volume III (136 pp) dealt with Construction, QA/QC, and Testing, and included a detailed review of construction techniques used in the countries then judged most active in micropiling.

The series of Workshops conducted to disseminate this micropile knowledge to other national groups such as the Nordic Countries and Japan, from 1997 onwards gave birth to the ISM we recognize today.

This paper begins by illustrating the differences in national practices which existed thirty years ago. It continues by considering the current state of international construction techniques, and why they have changed over the years. Factors contributing to the current situation include the development of new overburden drilling and grouting systems, the quality and type of equipment available, evolving QA/QC requirements, and methods and demands imposed by specific ground conditions and environmental concerns.

1. BACKGROUND

A companion paper in this Conference by Bruce et al., refers to the researches conducted on behalf of the Federal Highway Administration in the period 1993 – 1996 by the Principal Investigators Dr. Bruce and Prof. Juran. The product of that project was the publication of a series of four “State of Practice” Reports, in July 1997. Volume III addressed Construction, QA / QC, and Testing, and included a detailed review of construction techniques used in those countries then judged most active in micropile technology. These volumes became the basis for the FHWA “Implementation Manual”, first issued in 2000 and then updated and revised in 2005. This paper begins by reviewing the findings

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of Volume III, as a basis for thereafter illustrating the major changes which have (and have not!) taken place in the intervening 28 years preceding this conference.

2. SITUATION AS AT THE MID 1990's

2.1. Drilling

One of the most frequently quoted mantras associated with micropile construction is “primum non nocere” – firstly cause no damage – and this was assiduously observed by specialists from the beginning of micropile practice in Italy in 1952. The combination of limited drill rig availability and the very delicate condition of the historic structures to be underpinned (Lizzi, 1982) led to reliance on small scale rotary equipment and diamond drilling techniques. Diameters were typically less than 100mm, but gradually increased to 150mm or more as higher pile capacities were required. Bentonite flush was used in certain ground conditions, but with caution, since unless the slurry was completely evacuated from the hole prior to pile grouting, a reduced pile / ground bond resulted. This practice is still found in Italy and France, especially for Type D micropiles, as defined in Section 2.2.2 (below).

Later, throughout North America and Europe, micropile contractors selected the drilling method based on better equipment availability, regional and national preferences, and, of course, (it is hoped!) the specific ground and site conditions. Regarding the ground and site conditions, micropiles tend to be selected in applications where “conventional” driven piles or large diameter bored piles are either not feasible and / or there are overriding access or environmental challenges. In general, the drilling equipment and methods were similar to those used in ground anchor construction, and used similar types of drilling rigs, typically diesel-hydraulic or electro-hydraulic, track-mounted models from European manufacturers. For low head-room applications, short-masted rigs, track or wheel mounted, were developed.

A typical micropile application involves drilling through fills (often with obstructions), non-lithified soils, and into a bearing stratum comprising dense to very dense soils or rock. Hence, most micropiles involved the installation of a temporary steel casing, typically to the full depth of the pile, or at least a short distance into competent rockhead. Table 1 provides a summary of the most common generic overburden drilling methods recognized at the time and Figure 1 provides a simplified illustration. Duplex drilling (i.e., involving the simultaneous advancement of the outer drill casing with the inner drill rod) was introduced into soil anchor practice in the late 1960's. It was soon widely used in micropile practice, especially in urban settings where the returning drill flush had to be captured and returned to the surface within the casing / rod annulus as opposed to being allowed to escape from the drill hole in an indiscriminate and potentially harmful manner. Thus, “end of casing flushing”, also known as “wash boring”, was not a common or indeed allowable method for many micropile projects. Conventional core drilling methods were widely used for the initial hole penetration through reinforced concrete footings, or masonry structures, before switching to the overburden drilling system. For micropiles founded in rock, air-powered down-the-hole hammers, or water flushed top hammer percussion was used, the choice largely dependent on the diameter and depth of the bond zone.

Table 1. Overburden drilling methods (Bruce 1989), as reproduced as Table 1 in FHWA (1997).

DRILLING METHOD	PRINCIPLE	COMMON DIAMETERS AND DEPTHS	NOTES
1. Single tube advancement a) Drive drilling	Casing, with "lost point" percussed without flush.	50 - 100mm to 30m	Obstructions or very dense soils are problematical.
b) External flush	Casing, with shoe, rotated with strong water flush ("Wash Boring").	100 - 200mm to 60m	Very common for anchor installation. Needs high torque drill head and powerful flush pump.
2. Rotary duplex	Simultaneous rotation and advancement of casing and internal rod, carrying flush.	100 - 200mm to 70m	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 are percussed as well as rotated.	89 - 175mm to 40m	Useful in obstructed / bouldery conditions. Needs powerful top rotary percussion hammer.
4. Rotary percussive eccentric duplex	As 2, except eccentric bit on rod cuts oversized hole to ease casing advance.	89 - 200mm to 60m	Somewhat obsolescent and technically difficult system designed for variable overburden.
5. "Double head" duplex	As 2 or 3, except casing and rods are rotated in opposite senses.	100 - 150mm to 60m	Powerful, newer 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 - 400mm to 30m	Obstructions problematical; care must be exercised in cohesionless soils to avoid cavitation and / or excessive loosening. Prevents application of higher grout pressures.

Note: Drive drilling, being purely a percussive method, is not described in the text as it was not appropriate for micropile construction.

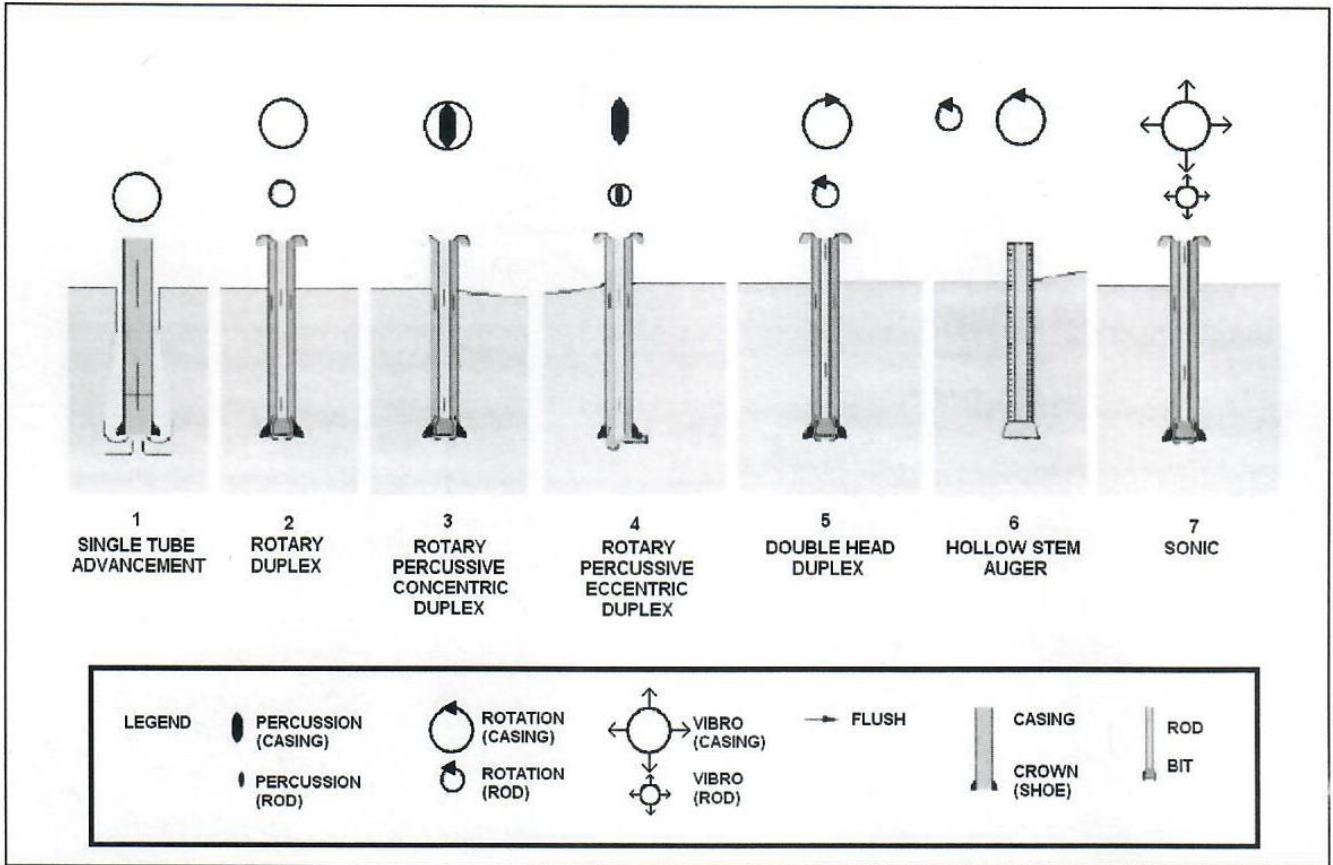


Figure 1. Overburden Drilling Methods (modified after Bruce, 1989), reproduced as Figure 4-9 in FHWA (2005).

Water was found to be the most common cooling, cleaning, and flushing drill medium in overburden, latterly supplemented by foam flush, which has considerable “lifting power” despite the fact that its critical Uphole Velocity is orders less than that of water. Compressed air was used, but with extreme care, given the danger that pneumatic fracturing of the overburden could lead to damage to the soils and overlying structures.

A very common installation method for the typical and popular Type A or B micropile (see Section 2.2, below) is illustrated in [Figure 2](#).

Various devices had been developed to enlarge / underream bond zones in cohesive soils or weak rock, largely associated with soil anchor practice, or for micropiles acting under tension. However, this was found to be a relatively complicated and time consuming process and it was rarely possible or convenient to verify its effectiveness. In addition, the cleaning of the underreams was most efficiently conducted by water flush, which had the perverse effect of softening the founding stratum. For all these reasons, underreaming was not usually a common practice, and was soon discontinued in favor of pressure-grouting technologies (Type B - D micropiles).

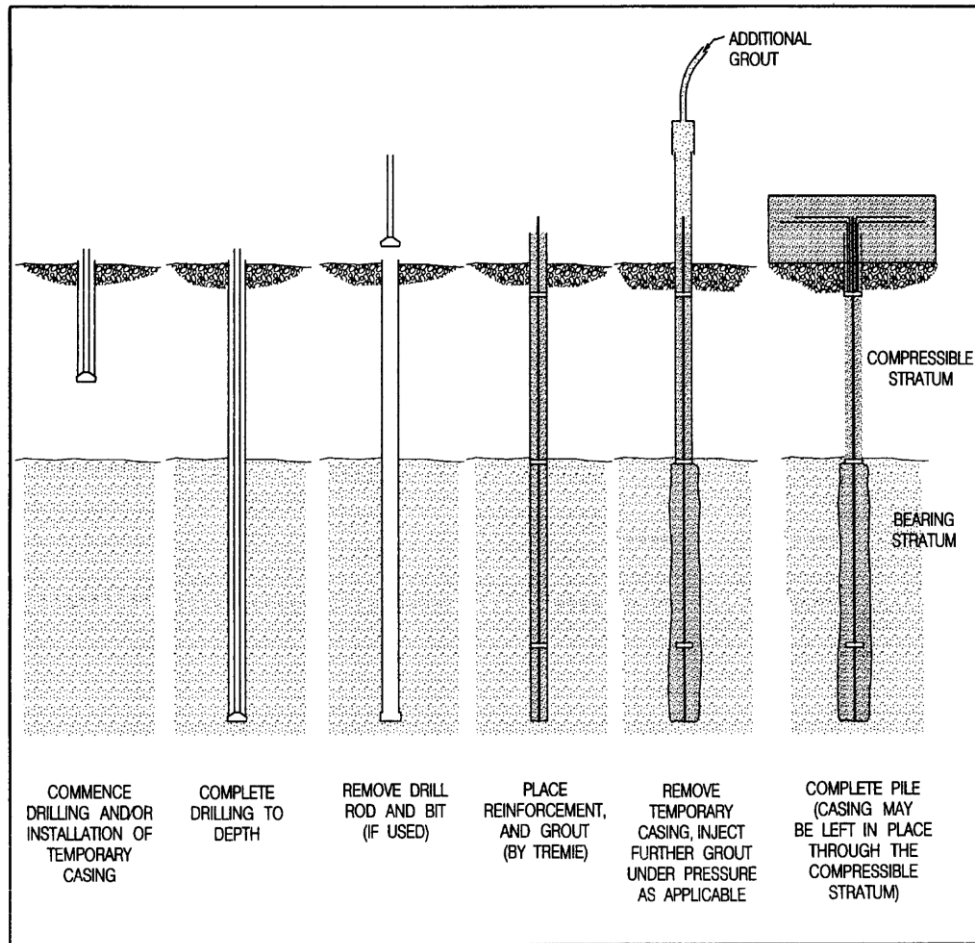


Figure 2. Typical construction sequence for a Type A or B micropile, (Figure 1 in FHWA, 1997).

Typical micropile hole diameters were recorded in the survey as follows:

Country	Common Micropile Diameter	Typical Range of Diameters
USA	150 - 200mm	120 - 300mm
UK	(see Table 5, below)	120 - 250mm
Italy	100 - 150mm	100 - 270mm
Germany	150mm	100 - 200mm
France	60 - 150mm	< 250mm

Specific details of UK practice were provided by Fleming et al., (1985), reproduced as Table 5 in the 1997 FHWA Report.

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	
Displacement piles	Hydro-Technique (micro-pieux)	Range 60 to 300	Range 20 to 600	Versatile with high load transfer characteristics.
	Quickpile (Ground Engineering Ltd.)	76		
	Terresearch Ltd.	120	150	
	GKN Keller Foundations Ltd. (GKN mini piles)	Range 160 to 300	Range 165 to 350	Top-driven steel tube, by drop hammer. Various rigs available. Vibro-driven cast in place.
	Menard Techniques Ltd. (Menard mini pile)	Tapered 160 to 100	160	
	Harison and Co. Ltd.	Tapered 250 to 150	200	Rapid installation.
	The House Piling & Foundation Co.	Range 150 to 300	Range 150 to 300	
	West's Piling & Construction Ltd. (West's mini-shell)	112	400	Driven.
	PM Minipile Ltd.	280	300	
	Roger Bullivant	80 +	40 +	Hydraulically jacked vibration-free system. Driven.
		65	20	

As a final point, the 1997 Report states (Page 45) "Increasing attention is being paid to lower capacity "self-drilling", thick walled piles such as those produced by MAI and Ischebeck", for which details were provided from 1992 and 1993 respectively. This proved to be a very prescient observation, and further commentary is provided in Section 3.2, below.

2.2 Grouting

2.2.1. Commonalities

The processes used in grouting micropiles were soon recognized as exerting major influence over the resultant micropile capacity, (Figure 3). Indeed the micropile classification proposed in the 1997 FHWA Report was based primarily on the grouting methodology, bearing in mind that very similar drilling equipment and systems were readily available and used by the major contractors in Europe and North America: this reflected the influence of equipment manufacturers operating aggressively throughout the world. (Another factor in drilling "uniformity" was the ever expanding markets for renting new and used equipment especially to smaller or newer contractors.) Details of the grouting methods were found to vary between countries, depending on local preferences and quality of resources. However, several commonalities became clear:

- grouts were designed to provide high strength and static stability while remaining easily pumpable (Figure 4). This implies water / cement ratios of 0.45 to 0.50 by weight, higher for post-grouted variants (Types C and D).
- grouts were produced with fresh water, to reduce the danger of corrosive attack to the reinforcement.

- the best quality grouts in terms of both fluid and set properties were produced by high speed, high shear mixers, as opposed to low speed, low energy mixers, such as those that depend on slowly rotating paddles.
- cement was supplied in bagged or bulk form, depending on site conditions, project size, local availability and cost.
- equipment could be driven by air, diesel, or electricity, and was available in a wide range of capacities and sizes from many manufacturers.
- lower pressure injections (up to 2 MPa for Type B micropiles) usually featured constant pressure, rotor-stator pumps, while higher pressure injections (Type C and D micropiles) required a piston or ram pump.
- most grouting featured neat cement mixes, although sand-cement grout was common in certain countries such as Italy and the UK, in ratios of 1: 1 up to 2: 1 sand: cement. The use of higher pressures had reduced the popularity of these mixes, in which the water / cement ratio was often extended to 0.6 to 0.7. Additives were required only in certain conditions e.g., high temperatures, long pumping distances, or to arrest “runaway” takes.

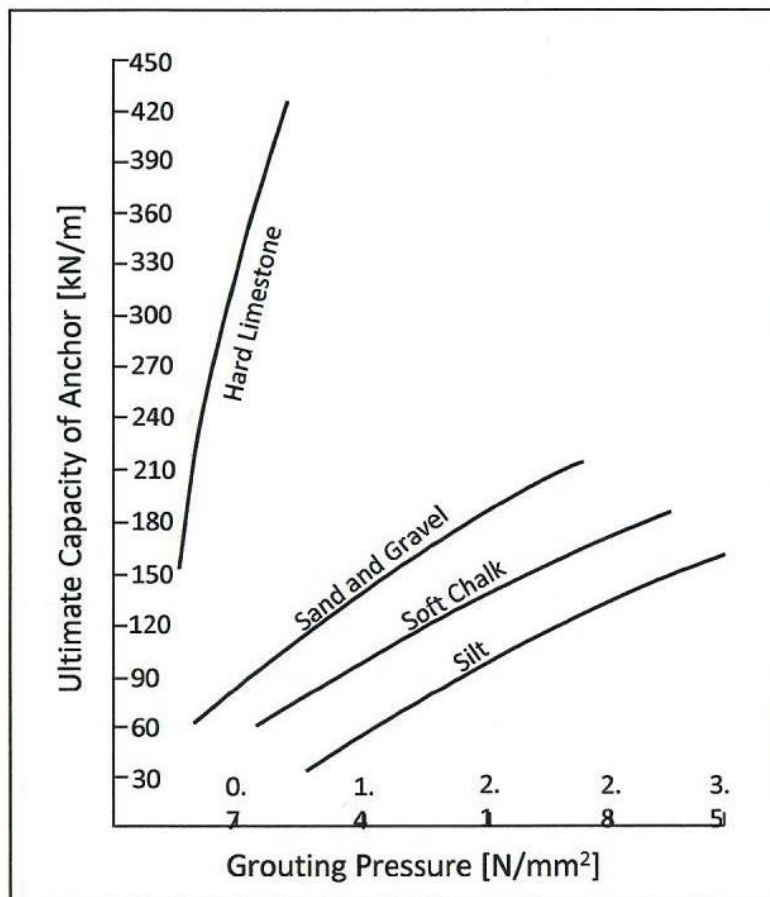


Figure 3. Influence of Grouting Pressure on Ultimate Load Holding Capacity (Littlejohn and Bruce (1977)).

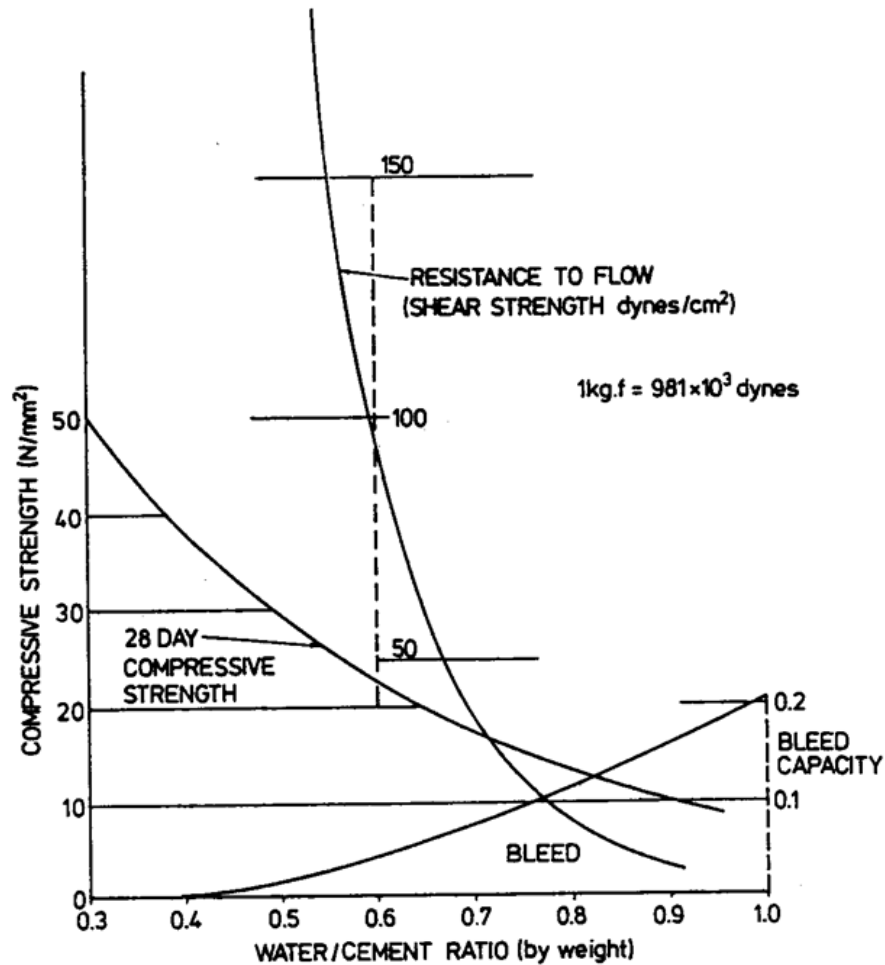


Figure 4. Basic Considerations for the Design of Cement-Based Grouts (Littlejohn and Bruce, 1977)

2.2.2. Micropile Classification Based on Grouting

Bearing in mind that at the time the FHWA study was conducted by 1995, the use of “self-drilling” micropiles was in its early days, European and North American practice was therefore regarded as encompassing four types of micropiles (Figure 5). Type E piles are described in Section 3, below.

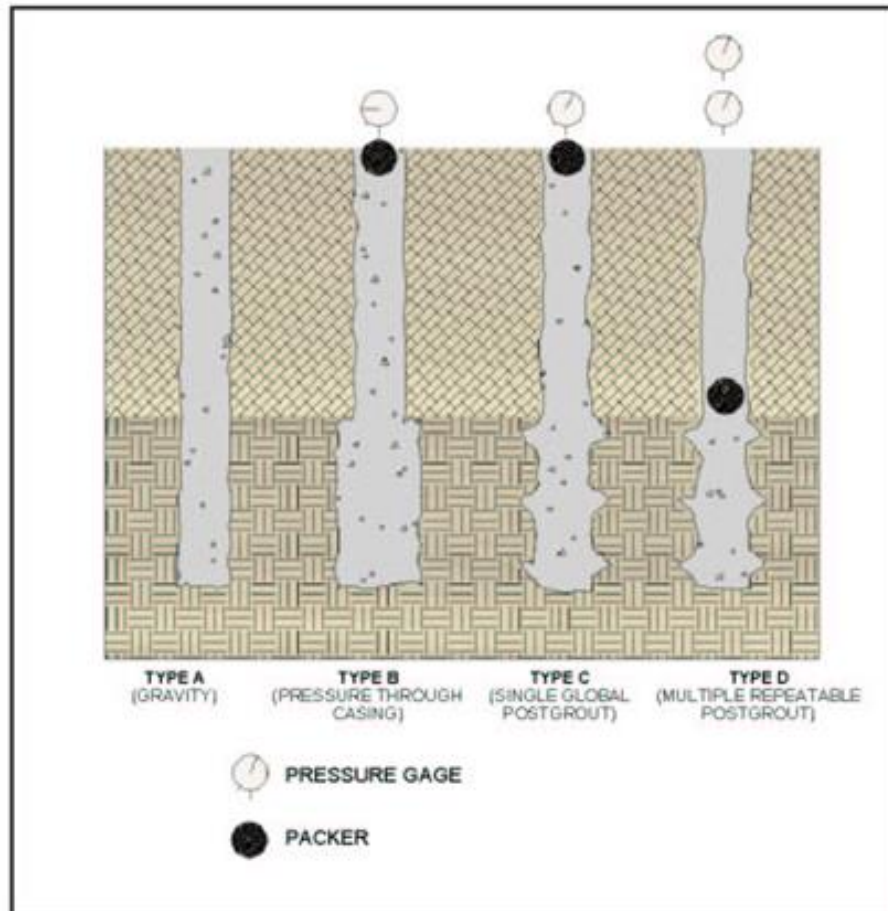


Figure 5. Micropile Classification System Based on Type of Grouting, as shown in the FHWA Implementation Manual (2005) as Figure 2-5.

- **Type A.** Following placement of the permanent reinforcement, the hole is tremied full of stable grout under gravity pressure only (neat grout with sand / cement mixes also common as a tactic to reduce grout costs). This is also referred to as the “Primary” treatment for other types of piles. Type A piles are applicable when founded in rock and / or when relatively low capacity piles are required in stiff / hard cohesive soils where pressure grouting is unnecessary to generate adequate ground / grout bond.
- **Type B.** Typically constructed in cohesionless soils (especially sands and gravels where full length temporary drill casing is required ([Figure 2](#)). Immediately following tremie placement of the Primary grout, additional grout of the same composition is injected into the casing via a pressure cap, or through the drill head, as the drill casing is progressively withdrawn in stages over the bond zone. The aim is to enhance the grout / soil bond capacity by “repairing the damage” to the soil caused during the drilling process (i.e., causing a redensification of locally loosened soil). Another benefit accrues from the phenomenon of pressure filtration of some of the water from the grout into the pores of the soil, hence reducing the water / cement ratio of the remaining mix, rendering it quicker setting

and higher strength. This also encourages development of a cake-like paste along the outside of the lower sections of the casing, so preventing grout simply escaping to the surface along that interface during its injection and withdrawal. In less permeable soils, some lateral compaction or localized improvement of the soil around the bond zone can occur, but to a lesser degree than in coarser soils.

In short, pressure grouting causes a recompaction or redensification of the soil around the bond zone, and, as a result of some amount of permeation into the (coarser) soil, will significantly increase the effective diameter of the bond zone.

In earlier versions of Type B piles in Europe, when the use of sand-cement grouts was common, the mortar was pressurized by applying compressed air through the drill casing. This method was rendered obsolete by developments in grout pump technology and the trend towards using only neat cement grouts which had progressively become more cost-effective in most countries. Indeed, specifications began to ban the use of compressed air to pressurize the grout.

U.S. practice had tended to favor the installation of higher capacity micropiles than in Europe, and so Type B micropiles were found to be the most common type. Excess grout pressures of 0.5 to 1.5 MPa were common, with line pressure losses conceptually balanced by the pressure of the grout column itself. The maximum injection pressures reflected several factors including:

- the need to avoid soil hydrofracture, heave or uncontrolled loss of grout.
- the type of drilling system (pressures are practically limited when using augers, due to leakage around the flights).
- the ability of the soil to form the “seal” around the casing during its extraction and pressure grouting.
- the “groutability” of the soil.
- the required grout / soil bond capacity.

It was also found in the U.S. that for Type B micropiles and for higher capacity Type A micropiles, the temporary drill casing was left in place over the “free length”, following a 1 - 2m “plunge” back into the bond zone. This casing greatly enhanced the buckling stability of the piles, while the costs of extracting the casing were found to exceed the material cost of the casing itself, especially in heavily Unionized areas.

Practices in the UK were similar to those in the U.S., although it was reported that in Continental Europe (and France in particular) injection pressures were even higher.

- Post-Grouted Micropiles (Types C and D)

The benefits of applying significant excess grouting pressures during micropile construction were widely acknowledged. However, it was not always possible to construct Type B piles in conditions where there was grout leakage around the casing, or there were valid project-specific concerns regarding controlled

hydrofracture or “runaway” grout takes during injection. Further, in some soil conditions, contractors did not need, nor use, temporary drill casing at all. In certain countries, the use of the tube à manchette system ([Figure 6](#)) was common practice in ground treatment and soil anchor practice, and so it is not surprising that it became widely used in micropile construction via a number of proprietary systems.

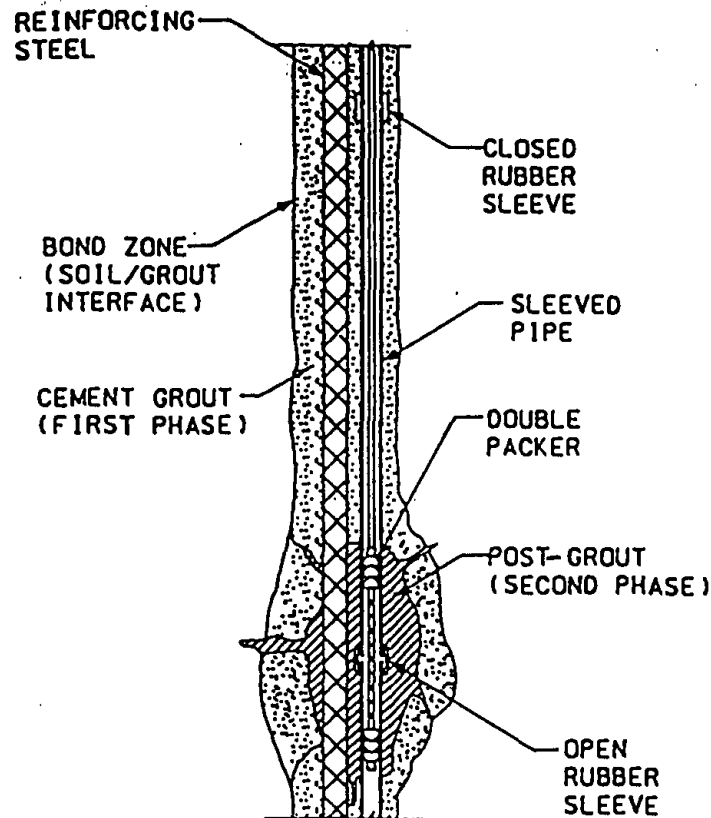


Figure 6. Principle of the tube à manchette method of post-grouting injection (schematic), reproduced as Figure 13 from the FHWA Report (1997).

Post-grouting permits discrete volumes of grout to be placed at specific locations along the bond zone, after the Primary grout has been placed. Post-grouting typically used higher water-cement ratios (0.50 to 0.75 by weight) to improve travel in the relatively small diameter injection pipework, expecting that pressure filtration of these mixes when injected into the soil would reduce the water content of the grout in-situ and hence improve its stability and strength.

A slight word of caution regarding the use of post-grouting was raised during the 1994 FHWA Workshop by Dr. Lizzi. This related to leaving “foreign materials (e.g. post-grouting tubes) in the pile, and contractors being tempted to use low strength (e.g., cement-bentonite) Primary grouts just to reduce “break out” pressures during post-grouting.

The 1997 FHWA study identified two separate concepts of post-grouting, leading to the definition of Type C and Type D micropiles:

- Type C: the Primary grout is placed under gravity head as for Type A. However, between 15 and 25 minutes thereafter (i.e., before initial grout set) similar grout is injected once from the top of the hole without a packer via a 38 - 50mm diameter preplaced tube à manchette, at an excess pressure of at least 1 MPa. This type of post-grouted pile was referred to in France as a Type III or IGU pile (Injection Globale et Unitaire).
- Type D: the Primary grout was typically a neat cement mix, but cement-bentonite and Self Hardening slurries were common in some countries (such as Italy) and also for use in Type C piles. The Primary grout was placed as for a Type A micropile, and allowed to set for 24 - 72 hours. Thereafter, grout was injected from the bottom of the bond zone upwards via the tube à manchette and using a double packer. By thorough washing of the tube after each phase of injection, desired locations could be reinjected, with controlled volumes of grout, several times to incrementally enhance grout-soil bond. This is reflected in the French term “Injection Répétitive et Sélective” (IRS) which was also referred to as a Type IV pile in French practice. Initial sleeve “rupture” pressures of up to 8 MPa were not uncommon, although subsequent “flow” pressures of far lower magnitude were typically recorded. Up to three phases of successive injection, spaced not less than 24 hours apart, and collectively injecting up to 50% of the initial Primary volume, were judged commercially and technically optimal.

The 1997 FHWA study found that there were several variants of the Type D pile method, although the “classic” tube à manchette with double packer was used on occasion in all the countries surveyed.

- In the U.S., the “Boston Method” was developed. This involved the installation with the micropile reinforcement of two relatively small diameter tubes à manchette with the sleeved sections covering the upper and lower part of the bond zone, respectively. By grouting from the top of the tube without the need for a packer, at least one sleeve could be relied upon to open in the upper and lower part of the bond zone, although the exact location and number could not be identified in advance. The upper tube would be grouted first, to conceptually provide vertical confinement of the post grout (water cement ratio = 0.50 to 0.75). This system proved surprisingly effective and economic and was also used widely for soil anchors.
- In Italy, systems were developed (e.g., “Tubfix”) where the steel pipe of the reinforcement was itself fitted with sleeves over the bond zone length, thus permitting repetitive post-grouting via a double packer, without the need to place a separate, smaller diameter tube à manchette.
- In Germany, “loop type” systems were popular ([Figure 7](#)). This featured a circulation line returning to the surface, thereby permitting the post-grout tube to be flushed out after each phase of post-grouting conducted from the surface. It was assumed that each phase of treatment opened a

different valve (or valves), so ensuring uniform treatment over the entire bond zone, without the need for a double packer. Maximum pressures of 3.5 MPa were reported. Primary grout mixes of water / cement ratios 0.36 to 0.44 were common, “depending on the type and quality of cement”.

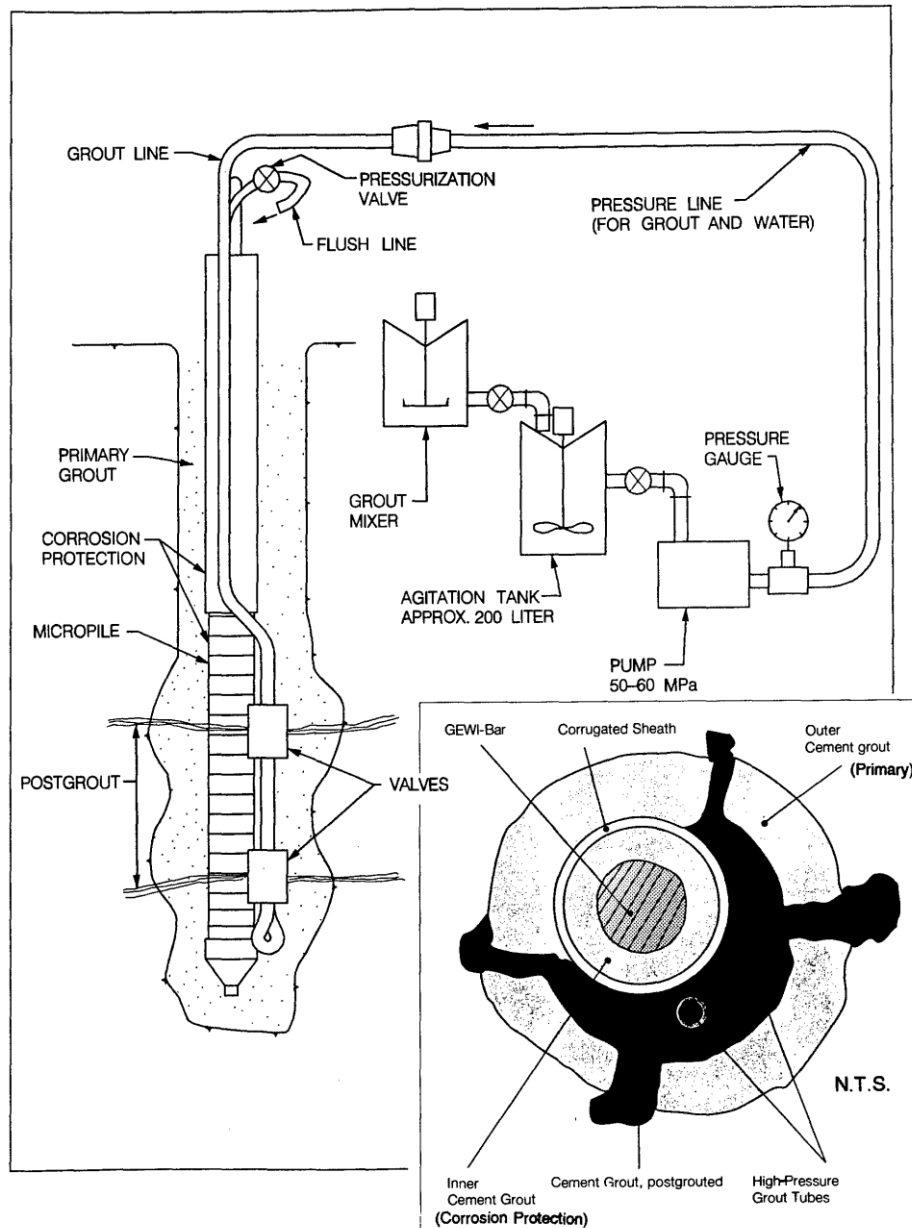


Figure 7. Elevation (schematic) and cross section of a “loop-type” post-grouting system (DSI, 1992). (Reproduced as Figure 15 in the FHWA Report, 1997).

As a general summary, the popularity of the various types of micropiles installed in the countries reviewed is shown in [Table 2](#).

Table 2. Summary of single micropile types used internationally, reproduced from Table 12 of the 1997 FHWA Report.

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

3. MAJOR CONSTRUCTION DEVELOPMENTS OVER THE LAST 30 YEARS

3.1. Drilling

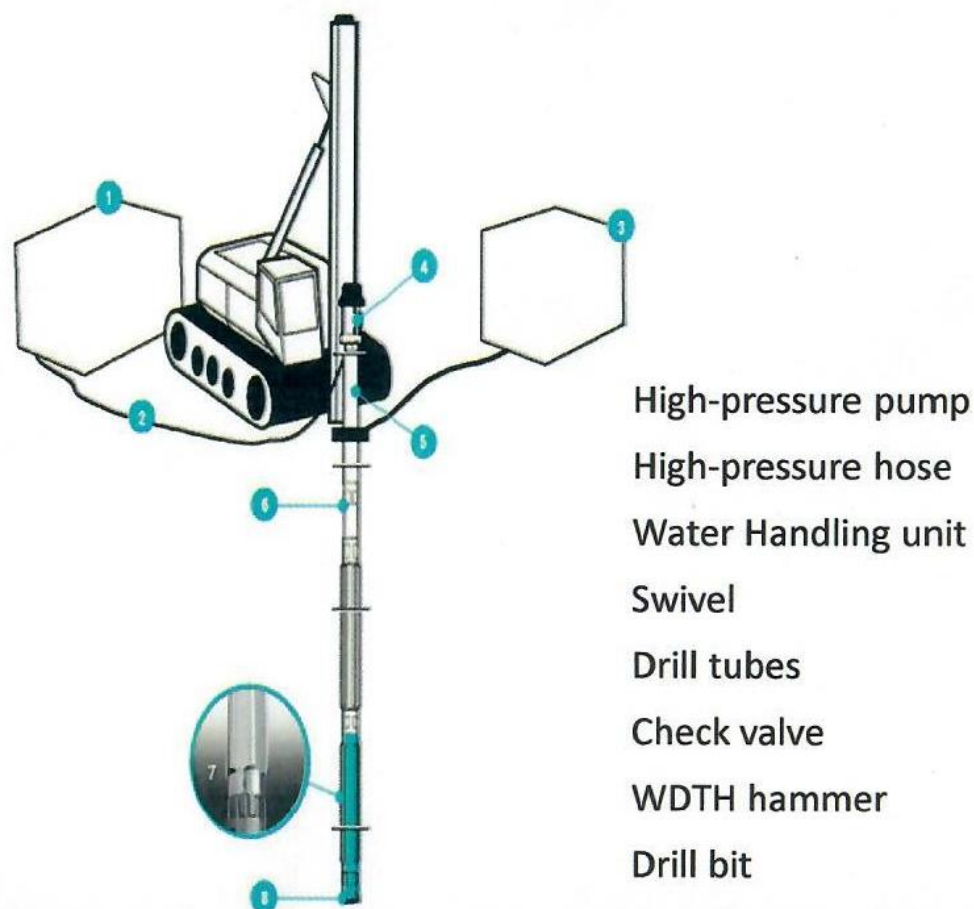
- As a general statement, developments in drilling equipment have focused on improving mechanical and hydraulic efficiency, increasing power (especially torque) and improving user-friendliness and operator safety while reducing operator numbers. In this regard, it is common to find automated rod and casing handling equipment (carousels or stacked magazine), with an increasing number of remotely operated drill rigs available on the market.
- Mirroring trends in the ground treatment industry, many projects specify requirements for automated “Monitoring While Drilling” (MWD) instrumentation. Such systems continuously record and display the Drilling Specific Energy, calculated from a number of drilling parameters, the most significant of which are drill torque, and instantaneous penetration rate. Properly calibrated against conventional site investigation data in the vicinity, MWD permits the ground conditions to be recorded for every hole, and is particularly useful for identifying the location and conditions of the targeted bond zone horizon. Such automated data are, of course, supplemented by the traditional manual logs maintained by the driller and / or the site inspection staff (Bruce, 2003).
- The use of rotary sonic overburden drilling rigs for dam remediation and environmental projects in particular has spread to many micropile applications, especially in very difficult / obstructed ground or when environmental considerations are paramount, since sonic drilling does not use a flushing medium)
- Water Powered Down-the-Hole Hammers have also become widely used for rock drilling, and resolve the problem of capturing expanding compressed air upon

exiting the drill collar. Data are provided in [Table 3](#), and a typical site arrangement is shown in [Figure 8](#).

Table 3. Water Powered Down-The-Hole-Hammer bit dimensions and operating parameters, (Courtesy of LKAB Wassara).

Hammer	Ø Drill Bit	Water Consumption	Max Operating Pressure
W50 (2')	60 mm (2 3/8')	80-130 l/min (20-35 USgpm)	170 bar (2500 psi)
W70 (3')	82 mm, 89 mm (3 1/4', 3 1/2')	130-260 l/min (35-70 USgpm)	180 bar (2600 psi)
W80 (3.5')	95 mm (3 3/4')	130-260 l/min (35-70 USgpm)	180 bar (2600 psi)
W100 (4')	115 mm, 120 mm (4 1/2', 4 3/4')	225-350 l/min (60-95 USgpm)	180 bar (2600 psi)
W120 (5')	130 mm, 140 mm (5 1/8', 5 1/2')	300-450 l/min (80-120 USgpm)	180 bar (2600 psi)
W150 (6')	165 mm (6 1/2')	350-500 l/min (95-130 USgpm)	150 bar (2200 psi)
W200 (8')	216, 254 mm (8 1/2', 10')	470-670 l/min (125-180 USgpm)	150 bar (2200 psi)

- The particular challenges of introducing casing through especially difficult and variable ground such as heavily karstified terrains, has led tooling manufacturers to develop highly specialized versions of the basic overburden drilling methods. For example, the Numa “Super-jaws” system features 2 to 4 “wings” on the drill bit that open by applying thrust from the drill head to the drill bit. These wings then cut a hole which is oversized relative to the diameter of the duplex casing, thereby allowing it to be smoothly advanced. This system is basically the direct descendent of the old Acker Casing Underreamer system of the 1970’s. However, if the bit suddenly encounters a soft horizon, the thrust on the bit is naturally lost, the wings therefore retract, and the casing (formerly supported on the extended wings) can literally fall off the bit and be lost down the hole. To combat this, Center Rock Inc. developed their “Roto Loc” system, similar to the “Super-jaws” but with the essential difference that a patented method of opening (and keeping open) the wings was invented, not relying on downwards pressure on the bit (Bruce and Lyon, 2017). The wings can then be folded back in when the casing has reached the final depth, so allowing the internal drill rod, hammer, and bit to be withdrawn. This system has many advantages, but it does require a high degree of operator skill, training and experience. Many different diameters are available, as listed in [Table 4](#).



Hammer	Ø Drill bit	Water consumption	Max op. pressure
W50 (2")	60mm, 64mm (2 3/8", 2 1/2")	80-130 l/min (20-35 USgpm)	170 bar (2500 psi)
W70 (3")	82mm, 89mm (3 1/4", 3 1/2")	130-260 l/min (35-70 USgpm)	180 bar (2600 psi)
W80 (3.5")	95mm , 102mm (3 3/4", 4")	130-260 l/min (35-70 USgpm)	180 bar (2600 psi)
W100 (4")	115mm, 120mm , 127mm (4 1/2", 4 3/4", 5")	225-350 l/min (60-95 USgpm)	180 bar (2600 psi)
W120 (5")	130mm, 140mm, 152mm (5 1/8", 5 1/2", 6")	300-450 l/min (80-120 USgpm)	180 bar (2600 psi)
W150 (6")	165mm , 178mm, 190mm, 203mm (6 1/2", 7", 7 1/2", 8")	350-500 l/min (95-130 USgpm)	150 bar (2200 psi)
W200 (8")	216mm, 254mm (8 1/2", 10")	470-670 l/min (125-180 USgpm)	150 bar (2200 psi)

Figure 8. Components of WDH equipment (Courtesy of LKAB Wassara).

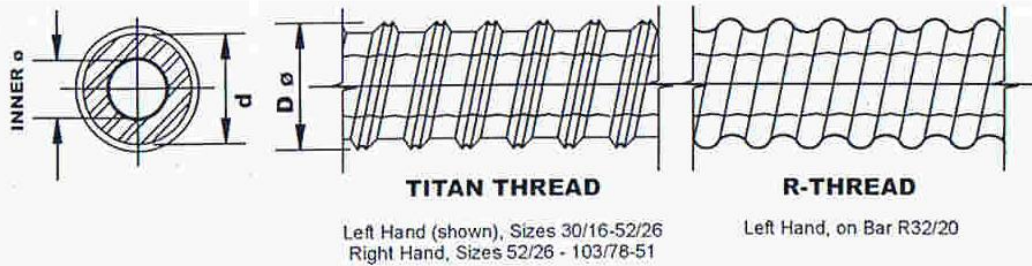
Table 4. Roto-Loc Diameters (Courtesy of Center Rock, Inc.)

SYSTEM	SHANK	WINGS	WING DIAMETER		SHOULDER DIAMETER	APPLICABLE CASING OD	WALL THICKNESS MAX	WEIGHT
			EXPANDED	RETRACTED				
			IN (MM)	IN (MM)	IN (MM)	IN (MM)	IN (MM)	LBS (KGS)
NO DRIVE SHOE SYSTEM								
ROC-0550RL	CR40, 340	2	5.86 (148.8)	4.38 (111.3)	N/A	5.500 (139.7)	.500 (12.7)	47 (22)
ROC-0600RL	CR40, 340	2	6.35 (161.3)	4.92 (125.0)	N/A	6.000 (152.4)	.500 (12.7)	58 (26)
ROC-0663RL	QL50	2	7.16 (181.9)	5.50 (139.7)	N/A	6.625 (168.3)	.500 (12.7)	71 (32)
ROC-0700RL	QL50, QL50HD, QL60, 360	2	7.75 (196.8)	5.94 (150.9)	N/A	7.000 (177.8)	.500 (12.7)	83 (38)
ROC-0763RL	QL60, QL60HD	2	8.34 (211.8)	6.52 (165.6)	N/A	7.625 (193.7)	.500 (12.7)	108 (49)
ROC-0863RL	QL60, QL60HD	4	9.35 (237.5)	7.50 (190.5)	N/A	8.625 (219.1)	.500 (12.7)	168 (76)
ROC-0963RL	QL80, 380	4	10.37 (263.4)	8.41 (213.6)	N/A	9.625 (244.5)	.545 (13.8)	231 (105)
ROC-1075RL	QL80	4	11.42 (290.1)	9.45 (240.0)	N/A	10.750 (273.1)	.595 (15.1)	250 (113)
ROC-1188RL	N100	4	12.54 (318.5)	10.17 (258.3)	N/A	11.875 (301.6)	.750 (19.1)	444 (201)
DRIVE SHOE SYSTEM								
ROC-0600RL	CR40, 340	2	6.35 (161.3)	4.92 (125.0)	5.27 (133.9)	6.000 (152.4)	.250 (6.4)	58 (26)
ROC-0663RL	QL50	2	7.16 (191.6)	5.50 (139.7)	6.25 (158.8)	6.625 (168.3)	.250 (6.4)	71 (32)
ROC-0963RLS	QL80, 380	4	10.37 (263.4)	7.92 (201.2)	8.75 (222.3)	9.625 (244.5)	.545 (13.8)	233 (106)
ROC-1075RLS	QL80	4	11.42 (290.1)	9.45 (240.0)	9.90 (251.5)	10.750 (273.1)	.350 (8.9)	306 (139)
ROC-1275RLS	N100	4	13.42 (340.9)	10.87 (276.1)	11.62 (295.2)	12.750 (323.9)	.625 (15.9)	521 (236)
ROC-1338RLS	N100	4	14.07 (357.4))	11.50 (292.1)	12.41 (315.2)	13.375 (339.7)	.750 (19.1)	500 (227)
ROC-1400RLS	N100	4	14.49 (368.0)	11.54 (293.1)	13.05 (331.5)	14.000 (355.6)	.750 (19.1)	510 (231)
ROC-1600RLS	QL120	4	16.88 (428.8)	14.07 (357.4)	15.13 (384.3)	16.000 (406.4)	.500 (12.7)	848 (385)
ROC-1800RLS	QL120	4	18.80 (477.5)	15.44 (392.2))	17.03 (432.6)	18.000 (457.2)	.845 (21.5)	998 (453)
ROC-2000RLS	QL200	4	20.87 (530.1)	17.63 (447.8)	19.06 (484.1)	20.000 (508.0)	.625 (15.9)	1353 (614)
ROC-2400RLS	QL200	4	24.88 (632.0)	21.20 (538.5)	22.81 (579.4)	24.000 (609.6)	.845 (21.5)	1868 (847)
ROC-3000RLS	CR260	4	30.87 (784.1)	26.90 (683.3)	28.80 (731.5)	30.000 (762.0)	1.000 (25.4)	4887 (2217)
ROC-3600RLS	CR260	6	36.90 (937.3)	33.88 (860.6)	34.82 (884.4)	36.000 (914.4)	1.000 (25.4)	5164 (2342)

3.2. **Grouting**

- The major development in grouting-related technologies is arguably the rise of the use of “self-drilling” bars (Type E). These hollow bars are percussed and rotated into the ground with a relatively high water-cement ratio grout used as the drill flush. Upon reaching target depth, additional grout of lower water-cement ratio is pumped, to displace upwards the “drilling” grout, which typically carries in suspension a considerable amount of drill debris. Typical bar sizes are shown in Table 5. In favorable soil conditions, (i.e., without obstructions or very dense / stiff horizons, instantaneous and industrial productivities can be very high – more than offsetting the cost of the special material used.
- Figure 9 shows the revised FHWA classification now including Type E piles.
- Mix designs remain largely based on simple, neat mixes of water-cement ratio around 0.45, occasionally with additives to improve pumpability and to retard set. The use of sand-cement grouts is now rare. In many countries, the use of Type 1L cement is, perforce, becoming common and definitive statements regarding its rheological and set properties (and how they compare to traditional types of cement) are awaited from trade and professional organizations.
- The use of automated systems for controlling and recording grout flow rates, volumes and pressures is becoming increasingly popular in many countries, although in others, there remains a reliance on “dipstick and gage” manual methods.
- Traditionally, the prime (and only) QA / QC tool has been the casting of cubes for 7, 14, and / or 28 day strength testing. This is retrospective and anachronistic: good practice is to routinely test the fluid properties of the grout before injection. Hence a Baroid Mud Balance provides a fast and accurate measurement of Specific Gravity and so confirmation that the correct water-cement ratio has been accurately batched. The use of a Marsh Flow Cone confirms the pumpability of the grout, and shows the change in grout rheology of a hydrating grout held in the storage tank approaching its acceptable “shelf life”. Prior to production works commencing, “classification testing” is conducted to confirm the relationship between mix design, stability and strength. These are not routine QA / QC tests during the production works. Guidance is provided in Chuaqui and Bruce (2003).
- The use of high speed, high shear mixers is universal, with a growing trend towards the use of semi-automatic mixer units. This is an important step towards assuring consistency of batching and hence the subsequent grout properties, fluid and set.

Table 5. Typical bar sizes (Courtesy of CTS / Titan)



Rod Size D Ø / INNER Ø	Area	Load Capacity					Outside Diameter		Weight
		Ultimate G.U.T.S.	Yield	Max. Test	Design ¹ Temp. Anch.	Design ² 60% G.U.T.S.	Effective d Ø	Nominal D Ø	
		in ² kips mm ²	kips kN	kips kN	kips kN	kips kN	in mm	in mm	
30/16	0.59	49.5	40.5	39.6	34.6	29.7	1.02	1.18	1.8
L.H. THREAD	382	220	180	176	154	132	26	30	2.7
30/14	0.61	58.5	49.5	46.8	40.9	35.1	1.03	1.18	1.9
L.H. THREAD	395	260	220	208.0	182	156	26	30	2.9
32/20	0.60	65.4	54.9	52.3	45.8	39.3	1.10	1.26	2.2
L.H. THREAD	389	291	244	233	204	175	28	32	3.2
30/11	0.69	72.0	58.5	58	50.4	43.2	1.03	1.18	2.2
L.H. THREAD	446	320	260	256	224	192	26	30	3.3
40/20	1.13	121.2	96.7	95.6	84.8	72.7	1.42	1.57	3.8
L.H. THREAD	726	539	430	425	377	323	36	40	5.6
40/16	1.36	148.4	118.1	116.9	103.9	89.0	1.42	1.57	4.7
L.H. THREAD	879	660	525	520	462	396	36	40	7.0
52/26	2.07	208.9	164.2	160.1	146.2	125.3	1.92	2.05	6.7
L.H. THREAD	1337	929	730	712	650	557	49	52	10.0
73/56	2.19	268.5	176.5	174.3	173.2	161.1	2.76	2.87	7.5
R.H. THREAD	1414	1194	785	775	770	716	70	73	11.1
73/53	2.53	260.9	218.1	208.7	182.6	156.5	2.76	2.87	8.3
R.H. THREAD	1631	1160	970	928	812	696	70	73	12.3
73/45	3.50	366.5	265.3	263.1	256.6	219.9	2.76	2.87	12.0
R.H. THREAD	2260	1630	1180	1170	1141	978	70	73	17.8
73/35	4.20	445.2	304.7	302.5	301.3	267.1	2.76	2.87	14.2
R.H. THREAD	2710	1980	1355	1345	1340	1188	70	73	21.2
103/78	4.88	513.2	404.8	400.3	359.2	307.9	3.94	4.06	16.7
R.H. THREAD	3146	2282	1800	1780	1597	1369	100	103	24.9
103/51	8.53	778.1	618.4	616.1	544.6	466.8	3.94	4.06	29.2
R.H. THREAD	5501	3460	2750	2740	2422	2076	100	103	43.4
127/111*	4.65	539.7	407.0	404.8	377.8	323.8	3.94	4.06	15.8
R.H. THREAD	3000	2400	1810	1800	1680	1440	100	103	23.5
130/60	14.79	1785.5	1180.6	1173.8	1169.3	1071.3	4.92	5.12	50.4
R.H. THREAD	9540	7940	5250	5220	5200	4764	125	130	75.0

¹ Temporary Anchors ; ² Permanent Anchors

Imperial values converted from metric values; October 2006

Note:
Subject to
change without
notice.

Shear Force
Allowable shear
force is deter-
mined by the
formula:

$$Q_{allow} = \frac{Yield * A}{1.75 * \sqrt{3}}$$

A: (Steel) Area

Certified to
ISO 9001

TITAN 127/111:
Allowable bend-
ing moment =
23.9 kNm
(737.5 lbsft)

- **Type A:** Gravity
- **Type B:** Pressure grouting through casing
- **Type C:** Single, global post grout
- **Type D:** Multiple, repeatable post grout
- **Type E:** Injection bore bars

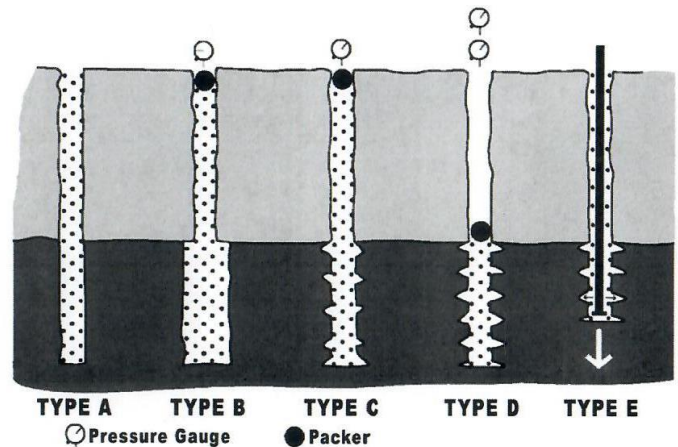


Figure 9. Revised Micropile Classification (FHWA, 2005).

4. FINAL REMARKS

The active promotion of micropile technology throughout the world by professional and trade associates, including ISM and several outstanding individual members, has led to several major trends becoming apparent, including:

- a rapid expansion of the market size,
- a marked improvement in QA / QC and verification methodologies,
- the use of similar drilling and grouting equipment and systems worldwide, reflecting the efforts of several excellent manufacturers, and collaborative contractors,
- continuous development of construction equipment to enhance productivity and reliability,
- a rise in the use of automated equipment (in some cases, even remotely operated to improve personnel safety), and
- ongoing developments in materials (e.g., the growing use of Type 1L cement and mix designs).
- the laudable and increasing awareness in the profession of environmental and sustainability goals, especially in the development of new techniques and materials.

Although these are common threads, there is still evidence of national and regional “flavors”, reflected in the growing number of codes of practices in various countries. However, these differences tend to be relatively minor, and are certainly not as dramatically diverse as the variety of approaches detailed in the 1997 FHWA Report.

Amidst all this “leveling up” throughout the world, there is still opportunity for innovation, as arguably most clearly illustrated by the rapid rise in the use of “self-drilling” micropiles (Type E in the FHWA classification). This growth is a source of inspiration for all practitioners involved in the construction of micropiles, driven to gain competitive advantage.

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Dedicated to the Vision of
Dr. Fernando Lizzi
(1914-2003)
The Godfather of Micropiles

