2.6 MICROPILES

INTRODUCTION AND HISTORY

Background

Between 1993 and 1996, the Federal Highway Administration (FHWA) funded the single most significant and comprehensive review of global micropile practice so far conducted. This effort also underlined the desire of the FHWA to contribute to a contemporary French national research project's five year effort named FOREVER (Fondations REnforcées VERticalement) and designed to conduct a variety of integrated experimental programs relating to micropiles. The FHWA study featured the formation of an International Advisory Panel comprising specialists from North America and Europe. Foremost amongst the members was Prof. Fernando Lizzi, of Naples, Italy acknowledged as the "god father" of micropiles as defined herein.

No only did this group ensure that a comprehensive review of practice was conducted, but also they were able to resolve a number of fundamental issues regarding various aspects of the classification, design, construction and performance of micropiles. These issues had been a source of confusion and misunderstanding and had therefore restricted the use of micropiles in certain engineering circles.

This review therefore introduces certain novel concepts which the reader may find somewhat different from standard descriptions on micropiles, including those such as Welsh (1987), and Bruce (1994). However, this new approach has received international concurrence, and is also being incorporated in the FHWA's "Implementation Manual" currently being prepared for use by State Departments of Transportation (1997).

Scope

Micropiles are, generically, small-diameter, bored, grouted-in-place piles incorporating steel reinforcement. They have been used throughout the world for various purposes, and this has spawned a profusion of national and local names, including pali radice, micropali (Italian), pieux racines, pieux aiguilles, minipieux, micropieux (French), minipile, micropile, pin pile, root pile, needle pile (English), Verpresspfähle and Wurzelpfähle (German) and Estaca Raiz (Portuguese). All, however, refer to the "special type of small diameter bored pile" as discussed by Koreck (1978).

Such a pile can withstand axial and/or lateral loads, and may be considered as either one component in a composite soil/pile mass or as a small diameter substitute for a conventional pile, depending on the design concept. Inherent in their genesis and application is the precept that micropiles are installed with methods that cause minimal disturbance to structure, soil and environmental. This therefore excluded other related techniques from the FHWA study such as those that employ percussive or explosive energy (driven elements), ultra-high flushing and/or grouting pressures (jet piles) or large diameter drilling techniques that can easily cause lateral soil decompression (auger cast piles). In addition, the new developments being made with compaction grout piles have not yet been published, and so the details remain in the hands of the contractors involved.

Micropile construction techniques are amongst those used to install soil-nails - sub-horizontal in situ reinforcements used in excavation support and slope stabilization (Fig. 2.6-1). However, soil nailing was regarded in concept, design, and function to be beyond the scope of the report and had already been the subject of major federal (NCHRP 1987, FHWA 1994) and private studies (Juran and Elias 1990, and Bruce 1993).

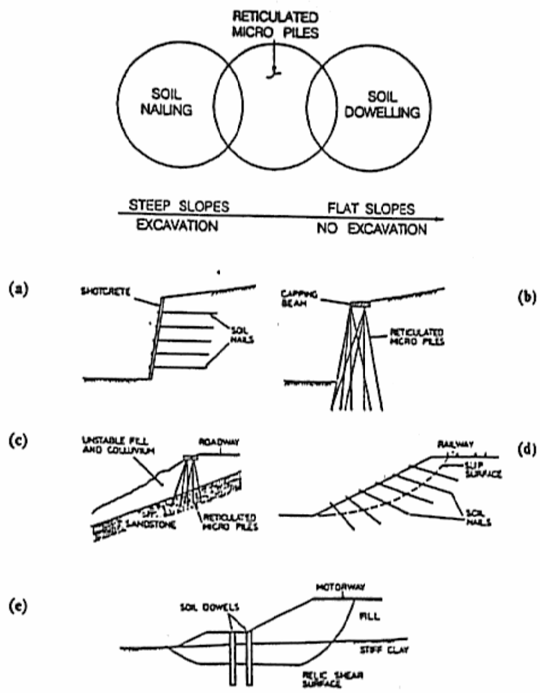

Fig. 2.6-1. Overlap of in situ reinforcement applications: (a) nails, and (b) micropiles, in excavations; (c) micropiles, and (d) nails, for general slope stabilization; and (e) dowels to stabilize residual slips in clay (Bruce and Jewell 1986, 1987).

Historical Note

The technology of micropiling was conceived in Italy in 1952 and introduced over two decades later into the United States (Bruce 1988, 1989). After a relatively slow start, the technology was widely applied by the late 1980's, especially in the eastern United Sates with an intensity mirroring that in Western Europe and South East Asia. Since that time, micropiling has spread both geographically and functionally within North America so that it is now equally common in California for seismic retrofits, and in the southern states and the Caribbean for slope stabilization. Overseas, renewed interest in the potential of micropiles in the aftermath of the Hanshin Earthquake in early 1995 has led to the formation of the Japanese Micropile Association in early 1997.

FUNDAMENTAL CONCEPTS

Characteristics and Definitions

Typical overviews of bearing pile types (e.g., by Fleming et al. 1985) begin by making the distinction between displacement and replacement types. Piles which are driven are termed *displacement* piles because their installation methods displace laterally the soils through which they are introduced. Conversely, piles that are formed by creating a borehole into which the pile is then cast or placed, are referred to as *replacement* piles because existing material, usually soil, is removed as part of the process. **Micropiles are a small-diameter subset of cast-in-place replacement piles.**

With conventional cast-in-place replacement piles, in which most, and occasionally all, the load is resisted by concrete as opposed to steel, small cross-sectional area is synonymous with low structural capacity. Micropiles, however, are distinguished by not having followed this pattern: innovative and vigorous drilling and grouting methods like those developed in related geotechnical practices such as ground anchoring, permit high grout/ground bond values to be generated along the micropiles's periphery. To exploit this potential benefit, therefore, high capacity steel elements, occupying up to 50 percent of the hole volume, can be used as the principal (or sole) load bearing element, with the surrounding grout serving only to transfer, by friction, the applied load between the soil and the steel. End-bearing is not relied upon, and in any event, is relatively insignificant given the pile geometries involved (Fig. 2.6-2). Early micropile diameters were around 100 mm (4 in), but with the development of more powerful drilling equipment, diameters of up to 300 mm(12 in) are now considered practical. Thus, micropiles are capable of sustaining surprisingly high loads (compressive loads of over 5000 kN (1120 kips) have been recorded), or conversely, can resist lower loads with minimal movement.

The development of highly specialized drilling equipment and methods also allows micropiles to be drilled through virtually every ground condition, natural and artificial, with minimal vibration, disturbance and noise, and at any angle below horizonal. Micropiles are therefore used widely for underpinning existing structures,

Fig. 2.6-3b. Here, the piles are not heavily reinforced since they are not individually and directly loaded: rather they circumscribe a zone of reinforced, composite, confined material that offers resistance with minimal movement. The piles are fully bonded over their entire length and so for this case to work, the soil over its entire profile must have some reasonable degree of competence. Lizzi's research (1982) has shown that a positive "network effect" is achieved in terms of load/movement performance, such is the effectiveness and efficiency of the reticulated pile/soil interaction in the composite mass.

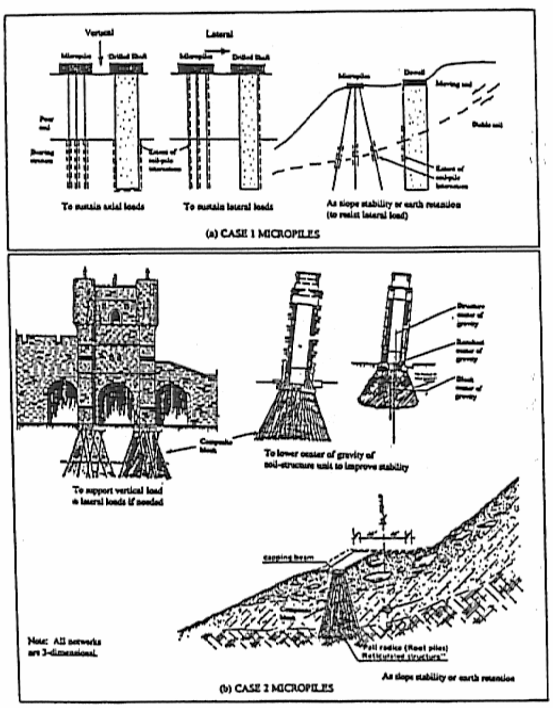

Fig. 2.6-3. Fundamental classification of micropiles based on their supposed interaction with the soil.

It is clear, therefore, that the basis of design for a CASE 2 network is radically different from a CASE 1 pile (or group of piles). Notwithstanding this difference, however, there will be occasions where there are applications transitional between these cases. For example, it may be possible to achieve a positive group effect in CASE 1 designs (although this attractive possibility is currently, conservatively, ignored for pile groups), while a CASE 2 slope stability structure may have to consider direct pile loading conditions (in bending or shear) across well defined slip planes. By recognizing these two basic design philosophies, even those transitional cases can be designed with appropriate engineering clarity and precision.

This classification also permits us to accept and rationalize the often contradictory opinions, made in the past about micropile fundamentals by their respective champions. For example, Lizzi (1982), whose intuitive focus is CASE 2 piles, was understandably an opponent of the practice of preloading high capacity micropiles, such as those described by Mascardi (1982) and Bruce (1992). These latter piles are now recognized as being of a different class of performance, in which complete pile/soil contact and interaction is not fundamental to their proper behavior. The advocates of these high capacity CASE 1 piles, in turn, now can appreciate the subtlety and potential of the CASE 2 philosophy.

Classification based on Method of Grouting. The successive steps in constructing micropiles are, simply:

- Drill;
- Place reinforcement; and
- Place and typically pressurize grout (usually involving extraction of temporary steel drill casing).

There is no question that the *drilling* method and technique will affect the magnitude of the grout/ground bond which can be mobilized. On the other hand, the act of placing the *reinforcement* cannot be expected to influence this bond development. Generally, however, international practice both in micropiles (e.g., French Norm DTU 13.2, 1992) and ground anchors (e.g., British Code BS 8081, 1989) confirms that the method of *grouting* is generally the most sensitive construction control over grout/ground bond development. The following classification of micropile type, **based primarily on the type and pressure of the grouting** is therefore adopted. It is shown schematically in Fig. 2.6-4.

- ***Type A:*** Grout is placed in the pile under gravity head only. Since the grout column is not pressurized, sand-cement "mortars", as well as neat cement grouts, may be used. The pile drill hole may have an underreamed base (largely to aid performance in tension), but this is now very rare and not encountered in any other micropile type.

- *Type B:* Neat cement grout is injected into the drilled hole as the temporary steel drill casing or auger is withdrawn. Pressures are typically in the range of 0.3 to 1 MPa (6 to 20 ksf), and are limited by the ability of the soil to maintain a grout tight "seal" around the casing during its withdrawal, and the need to avoid hydrofracture pressures and/or excessive grout consumptions.
- *Type C:* Neat cement grout is placed in the hole as for Type A. Between 15 and 25 minutes later, and so before hardening of this primary grout, similar grout is injected, once, via a preplaced sleeved grout pipe at a pressure of at least 1 MPa (20 ksf). This type of pile, referred to in France as IGU (Injection Globale et Unitaire), seems to be common practice only in that country.
- *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. In this case, however, a packer is used inside the sleeved pipe so that specific horizons can be treated, several times if necessary, at pressures of 2 to 8 MPa (40 to 160 ksf). This is referred to in France as IRS (Injection Répétitive et Sélective), and is common practice worldwide.

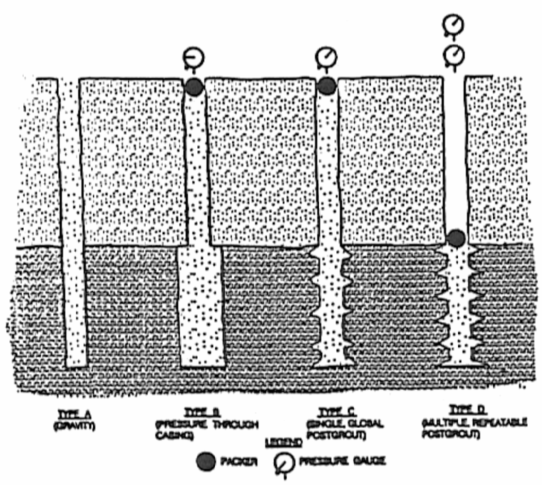

Fig. 2.6-4. Classification of micropile based on type of grouting.

Table 2.6-1 provides more details about this classification and also indicates the relationship between other proposed classifications and terminologies.

Combined Classification. Micropiles can therefore be allocated a classification number denoting the philosophy of behavior (CASE 1 or CASE 2), which relates fundamentally to the design approach, and a letter denoting the method of grouting (Type A, B, C, or D), which reflects the major constructional control over capacity.

For example, a repeatedly post-grouted micropile used for direct structural underpinning is referred to as Type 1D, whereas a gravity grouted micropile used as part of a stabilizing network is Type 2A.

Applications

Micropiles are used in two basic applications: as structural support and as in situ reinforcement (Fig. 2.6-5). For direct structural support, groups of micropiles are designed on the CASE 1 assumptions, namely that the piles accept directly the applied loads, and so act as substitutes for, or special versions of, more traditional pile types. Such designs often demand substantial individual pile capacities and so piles of construction Types A (in rock or stiff cohesives), and B and D (in most soils) are most commonly used.

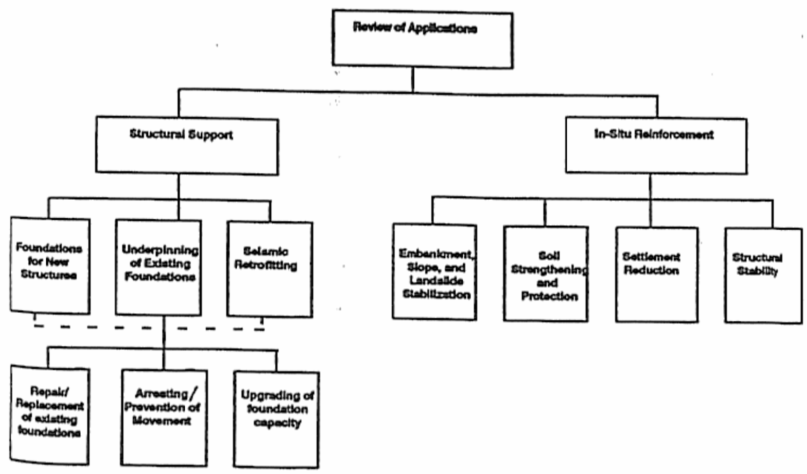

Fig. 2.6-5. Classification of micropile applications.

Table 2.6-1. Details of micropile classification based on type of grouting (continues).

MICROPILE TYPE AND GROUTING METHOD	SUBTYPE	DRILL CASING	REINFORCEMENT	GROUT	COMPARISON WITH OTHER TYPES OR CLASSIFICATIONS	NOTES
TYPE A Gravity grout only	A1	Temporary or unlined (open hole or auger)	None, monobar, cage or tube	Sand/cement mortar or neat cement grout, tremied to base of hole (or casing), no excess pressure applied	• Original "Root Piles" • GEWI Pile • French Types I or II	• Majority of Type A micropiles now used only when bond zone is in rock or stiff cohesives. • Includes underreamed piles, but very rare. • Unreinforced micropiles now not used (or allowed by codes).
	A2	Permanent, full length	Drill casing itself		• NCC Types S2 and R2	
	A3	Permanent, upper shaft only	Drill casing in upper shaft, bar(s) or tube in lower shaft (may extend full length)		• NCC Types S1 and S2	
TYPE B Pressure-grouted through the casing during withdrawal	B1	Temporary or fully extracted	Monobar(s) or tube (cages rare due to lower structural capacity)	Neat cement grout is first tremied into drill casing. Excess pressure (up to 1 MPa typically) is applied to additional grout injected during withdrawal of casing	• Later "Root Piles" • French Type I • Italian "Steel Pile" • GEWI Pile	• Sand/cement mortars are used very rarely, since these may cause problems during pressurization.
	B2	Permanent, full length	Drill casing itself		• NCC Types S2 and R2	
	B3	Permanent, upper shaft only	Drill casing in upper shaft, bar(s) or tube in lower shaft (may extend full length)		• NCC Types S1 and S2	

Table 2.6-1. Details of micropile classification based on type of grouting (continued).

MICROPILE TYPE AND GROUTING METHOD	SUBTYPE	DRILL CASING	REINFORCEMENT	GROUT	COMPARISON WITH OTHER TYPES OR CLASSIFICATIONS	NOTES
TYPE C Primary grout placed under gravity head, then one phase of secondary "global" pressure grouting	C1	Temporary or unlined (open hole or auger)	Monobar(s) or tube (cages rare due to lower structural capacity)	Neat cement grout is first tremied into hole (or casing). Between 15 to 25 minutes later, similar grout injected through tube (or reinforcing pipe) from head, once pressure is greater than 1 MPa.	• French Type III (Injection Globale et Unitaire)	• Appears to be used in France only. • Secondary grouting via a separate sleeved pipe or through the reinforcement tube equipped with sleeves.
	C2	Not possible	-		-	
	C3	Not conducted	-		-	
TYPE D Primary grout placed under gravity head, then one phase of secondary "global" pressure grouting	D1	Temporary or unlined (open hole or auger)	Monobar(s) or tube (cages rare due to lower structural capacity)	Neat cement grout is first tremied into hole (or casing). Some hours later, similar grout injected through sleeved pipe (or sleeved reinforcement) via packers, as many times as necessary to achieve bond.	• French Type IV (Injection Répétitive et Sélective) • Tubfix • IM Pile	• Typically, the classic tube à manchette is used with double packer. Alternatively, the steel tube can be equipped with sleeves or the DSI regrout tube (with return) can be used (Volume 3). • Secondary grouting via a separate sleeved pipe or through the reinforcement tube equipped with sleeves.
	D2	Not possible	-		-	
	D3	Permanent, upper shaft only	-		• NCC Type S1 • GEWI Pile	

For micropiles used as in situ reinforcement, the original CASE 2 network featured low capacity Type A piles. The research by Pearlman et al. (1992) on groups of piles, suggests that in certain conditions and arrangements, the piles themselves are principally, directly, and locally subjected to bending and shearing forces. This would, by definition, be a CASE 1 design approach. Such piles are usually highly reinforced and of Type A or B only.

Whereas CASE 1 and CASE 2 concepts alone or together can apply to slope stabilization and excavation support, generally only CASE 2 concepts apply to the other major applications of in situ reinforcement. Little commercial work has been done in these applications (with the exception of improving the structural stability of tall towers (Fig. 2.6-3b). However, the potential is real and the subject is being actively pursued in the "FOREVER" program in France. Table 2.6-2 summarizes the link between application, classification, design concept, and constructional method. It also provides an indication of how common each application appears to be world-wide.

Table 2.6-2. Relationship between micropile application, design concept, and construction type.

APPLICATION	STRUCTURAL SUPPORT	IN-SITU EARTH REINFORCEMENT			
Sub-applications	Underpinning of Existing Foundations New Foundations Seismic Retrofitting	Slope Stabilization and Excavation Support	Soil Strengthening	Settlement Reduction	Structural Stability
Design concept	CASE 1	CASE 1 and CASE 2 with transitions	CASE 2 with minor CASE 1	CASE 2	CASE 2
Construction type	Type A (bond zones in rock or stiff clays) Type B and D in soil (Type C only in France)	Type A (CASE 1 and 2) and Type B (CASE 1) in soil	Type A and B in soil	Type A in soil	Type A in soil
Estimate of relative application	Probably 95% of total world applications	0 to 5%	Less than 1%	None known to date	Less than 1%

DESIGN CONCEPTS

Volume 2 of the FHWA State of Practice Report (1996) deals with design. It is by far the largest and most complex volume of the five produced, and yet, in many ways, it is the least definitive, such is the current status of design methods in general. The approach adopted is as follows:

- ♦ Design of single micropiles [i.e., CASE 1, axial and lateral loads (101 pages)]
  - Geotechnical (external) design
  - Structural (internal) design
- ♦ Design of Groups of Micropiles (CASE 1) (73 pages)
  - Experimental Observations
  - Axial Loading (Load and Movement Calculation)
  - Lateral Loading (Load and Deflection Calculation)
  - Combined Loading
  - Cyclic Loading
  - Specific Methods for Foundation Underpinning, In Situ Soil Reinforcement, Slope Stabilization and Creeping Slopes
- ♦ Design of networks of micropiles (CASE 2) (7 pages)
  - Foundation Underpinning
  - Slope Stabilization

It is highly significant that the last section, dealing with CASE 2 structures, is extremely small in relation to the other two (CASE 1) sections. This reflects how little is actually known about CASE 2 design aspects, clearly highlights a major research need, and goes a long way towards explaining their infrequent use to date.

The static design methods for single CASE 1 piles draw from conventional bored pile theory, prestressed ground anchor practice, and of course from the more limited pool of micropile knowledge, per se. In competent soils, and in rock, the governing capacity calculation is the internal structural capacity of the pile itself, such is the great magnitude of grout/ground bond capacity which can be developed with contemporary drilling and grouting methods. This therefore focuses attention on the size, nature and yield strength of the reinforcement, assuming that the grout/steel bond [ultimate 1.5 MPa (30 ksf) for plain bar, 3 MPa (60 ksf) for deformed bar] is not critical, and that the contribution of the grout, *in compression*, is clearly defined (typical allowable design stresses of 0 to 40 percent U.C.S.). Allowable design stresses for steel range to 50%f_y. Movement calculations are driven by the same factors, plus the "effective" free length, i.e., that length below the head over which the pile reinforcement is actually being compressed. In this regard, the research of Bruce et al. (1992) has shown how this effective free length can be accurately calculated based on cyclic load-movement test data. Table 2.6-3 summarizes some geotechnical design guidelines.

Regarding groups, Lizzi (1982) showed, via laboratory tests, that for interpile spacings of 2 to 7 pile diameters, the axial load bearing capacity of the pile group was up to 30 percent greater than the sum of the individual piles in that group. Although this observation has been supported by numerous other researchers, no advantage appears to be taken of this "positive group effect" in contemporary micropile practice, although doubtless it does contribute to the surprisingly "stiff" response of micropile supported structures in practice.

Progressing to networks of piles, Lizzi (1978) showed an even greater positive group effect (Fig. 2.6-6). By reticulating the piles, the improvement over the same number of piles in a vertical group was 32 percent, while the positive group effect, relative to individual piles was 222 percent. These trends are being reevaluated by the FOREVER team in France, and early results appear totally consistent, allowing for variations in ground type and model geometry.

For pile groups and networks, therefore, it can be concluded that there is a certain degree of design rationale, backed by analytical and experimental studies. However, the extent of this rationale is small indeed compared to the great potential for its application, in underpinning and slope stabilization schemes, both seismic and static. Herein lies the principal challenge to micropile researchers over the next few years.

CONSTRUCTION

Fig. 2.6-7 illustrates the standard successive steps in the construction of a Type B micropile. As noted above, Type A piles are not subjected to excess pressure during Primary grouting while Types C and D are pressure grouted at some point after the Primary grouting is completed. Highlights of the successive steps are as follows.

Drilling

Where micropiles are to be installed through existing (reinforced) concrete or masonry footings, it is common to use high speed diamond drilling techniques to form an oversized hole, to permit the subsequent overburden drilling to commence. Diamond drilling typically provides a very smooth borehole wall and so, to enhance subsequent structure-pile load transfer, this interface is often "roughened up" using an appropriate tool. Alternatively, if it is environmentally and/or structurally permissible, a down-the-hole hammer can be used to penetrate these existing structures.

Thereafter, the technical and economic success of the job is largely dependent on the contractor's ability to drill through the overburden, any obstructions (natural and artificial), and into the bedrock if that is where the pile is to be founded. There are fundamentally six generic methods of drilling overburden, as summarized in Table 2.6-4, and the most appropriate method is selected with respect to the site, the subsurface conditions, and the type and size of the pile.

Table 2.6-3. Geotechnical Design Guidelines for Single Piles (continues).

Loading	AXIAL						LATERAL						SEISMIC
Purpose	Ultimate Load			Movement Control			Ultimate Load			Deflection Control			
Pile Type	A	B	C,D	A	B	C,D	A	B	C,D	A	B	C,D	
USA (Nicholson)	N/Ap (In rock)	Eq [17] LT		N/Ap	LT		N/Ap	LT		N/Ap	LT "p-y" LPILE		N/Av
AASHTO(1992) (Drilled Shafts) (Piles)	α meth - coh-TSA β meth - gran-ESA		N/Ap	Refer to ES,FE	N/Ap	N/Ap	Refer to AS	N/Ap	N/Ap	Refer to ES,FE "p-y" LPILE	N/Ap	N/Ap	N/Av
MBC (1988) * (Drilled Shafts) (Small Diameter Piles)	LT	N/Ap	N/Ap	LT	N/Ap	N/Ap	LT	N/Ap	N/Ap	LT	N/Ap	N/Ap	N/Av
API-RP-2A.1989 (Drilled Shaft Piles)	α meth-coh-TSA β meth- gran-ESA		N/Ap	N/Av	N/Av	N/Av	SEM	N/Ap	N/Ap	"p-y"	N/Ap	N/Ap	N/Av
CALTRANS (1994) (Drilled Shafts)	α meth- coh β meth- gran LT	N/Ap	N/Ap	"t-z" (Reese and O'Neill, 1987)	N/Ap	N/Ap	SEM	N/Ap	N/Ap	"p-y" different soils	N/Ap	N/Ap	ARS curves FE codes (GT Strudl or ADINA)
P.T.I, 1986 (Ground Anchors)	LT DC	LT DC	LT DC	LT	LT	LT	N/Ap	N/Ap	N/Ap	N/Ap	N/Ap	N/Ap	N/Av
GERMANY * (DIN 4128) (Small Diameter Injection Piles)	N/Ap	RV. (Ostermayer and Scheele, 1978) LT		N/Av	N/Av	N/Av	N/Av	N/Av	N/Av	N/Av	N/Av	N/Av	N/Av
FRANCE * (DTU- CCTG, 1993) (Micropiles)	DC (Pressurmeter, SPT)			"t-z"			"p-y" Pressuremeter			"p-y" Pressuremeter			N/Av

Table 2.6-3. Geotechnical Design Guidelines for Single Piles (continued).

Loading	AXIAL						LATERAL						SEISMIC
Purpose	Ultimate Load			Movement Control			Ultimate Load			Deflection Control			
Pile Type	A	B	C,D	A	B	C,D	A	B	C,D	A	B	C,D	
U.K (BS-8081, 1989)	α meth- β meth- SPT	Eq [18]. RV. (Ostermayer and Scheele,1978)		LT	LT	LT	N/Ap	N/Ap	N/Ap	N/Ap	N/Ap	N/Ap	N/Av
ITALY		(Lizzi, 1985)											N/Av

N/Ap = Non Applicable. TSA = Total Stress Analysis. Eq[17] = $f_s = p_v \tan\phi'$ A= Tremie Grouted Micropiles
N/Av = Non Available. ESA = Effective Stress Analysis LT = Load Testing. B = Low Pressure Grouted Micropiles

FE = Finite Element RV = Recommended Values DC = Design Charts C= Post-Grouted (French IGU)
ES = Elastic Solutions. * = Specific to Micropiles AS = Analytical Solutions D = Post Grouted (French IRS).
SEM = Semi-Empirical Model for Ultimate Load Values Used in "p-y" Curves.

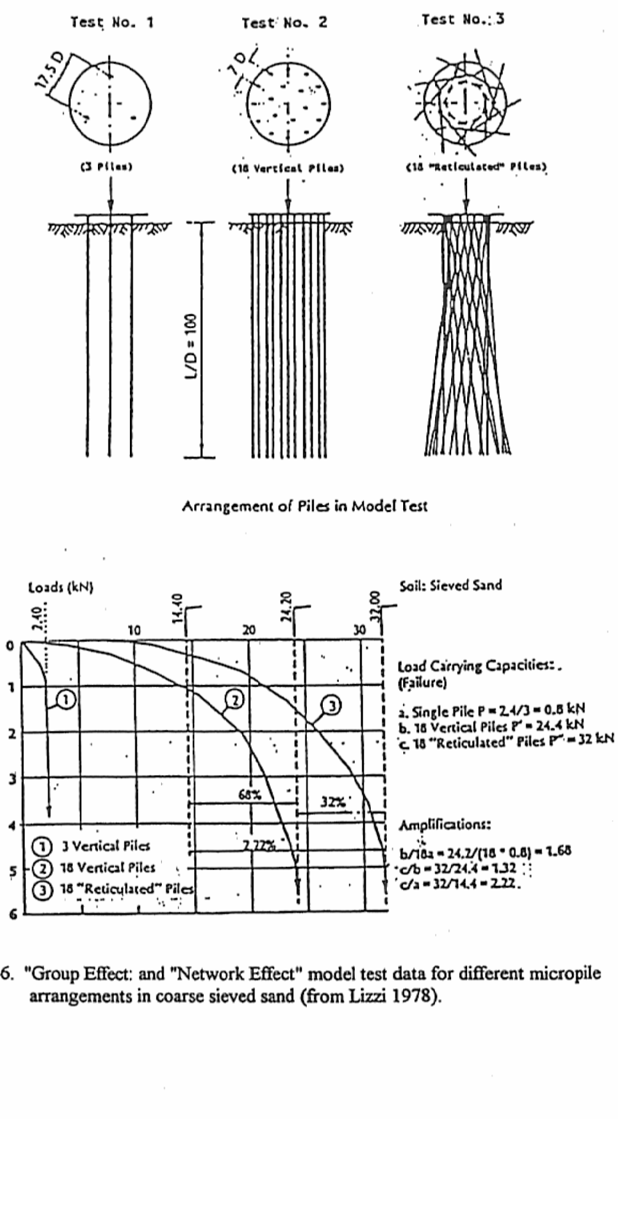

Fig. 2.6-6. "Group Effect: and "Network Effect" model test data for different micropile arrangements in coarse sieved sand (from Lizzi 1978).

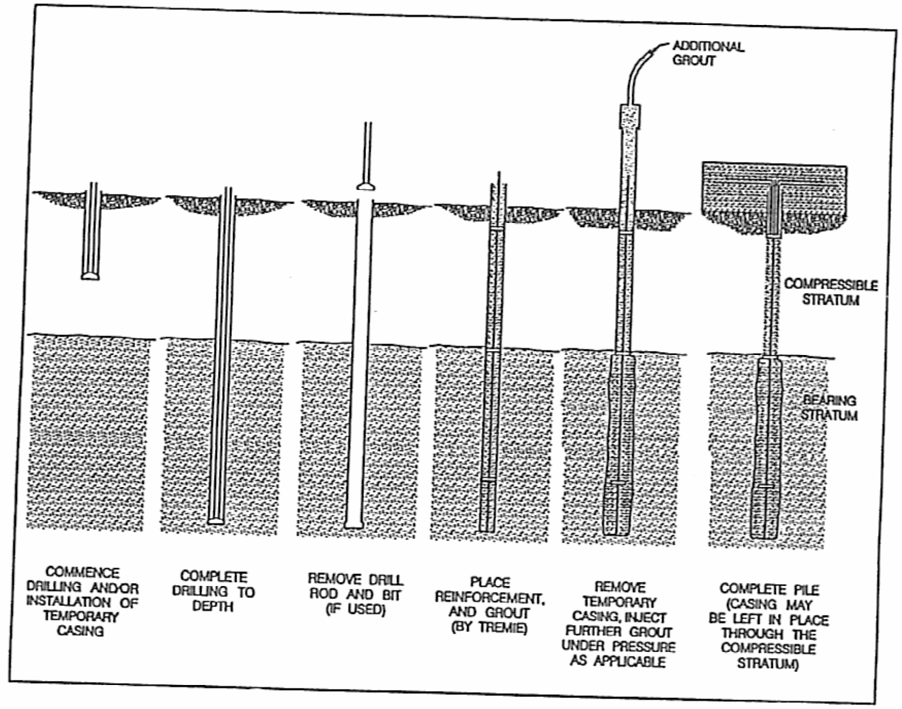

Fig. 2.6-7. Typical construction sequence for a Type A or B micropile.

Table 2.6-4. 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 duples	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.

Drilling rigs are typically diesel- or electro-hydraulically powered, and may be crawler or frame mounted. Special rigs have been developed for very restricted site conditions, and these rigs, although they may be relatively small in width and/or height, can provide considerable rotary power - essential for overburden drilling.

Drilling is most commonly conducted with water flush, although foam flush is frequently used in very difficult drilling conditions (Bruce et al. 1992). Air flush should only be permitted with extreme caution when drilling overburden in urban environments for fear of causing pneumatic fissuring of the ground and structural distress.

Reinforcement

Reinforcement commonly consists of one or more steel bars, Grade 60 or 150. Typical bar diameters range from 25 to 63 mm (1 to 2.5 in). Individual bar pieces are coupled together in lengths, which depending on the site circumstances, may vary from 1 to over 6 m (3 to 20 ft). Centralizers, usually plastic, should be located at 3 m intervals along each bar.

Alternatively the reinforcement can be in the form of a pipe section, with or without additional central reinforcement for whole or part of the length. Pipe sections - also used as the drill casing - are described in Table 2.6-5.

Table 2.6-5. Axial tension and compression loads fro API N-80 steel casing.

Casing OD	5-1/2	7	9-5/8
in/mm	139.7	177.8	244.5
Wall Thickness	0.361	0.498	0.472
in/mm	9.17	12.65	11.99
Steel Area	5.83	10.17	13.57
in^2/mm^2	3,760	6,563	8,756
Yield Load	466	814	1,086
kips/kN	2,075	3,619	4,829

Grouting

Grouts used in the Primary injection phase are stable, and have high 28-day unconfined compressive strengths - typically in excess of 25 MPa (500 ksf). In the United States, neat water/cement mixes of w/c = 0.40 to 0.50 are common, whereas in other countries, sand/cement mixes are more widely used, especially where grout takes into the surrounding formation (e.g., karstic limestone conditions) may be excessive. Special ground and/or ground water chemistries may require the use of special cements, but usually a Type I or II is sufficient - Type III if higher early strength is required. Additives are rarely necessary, although plasticizers are useful in very hot conditions or when pumping distances are substantial. Mixing is best conducted in high speed, high shear mixers.

Grout for Secondary operations - as in Type C and D piles - usually has a higher w/c ratio, to aid injection through the small-diameter pipework. It is reasoned in this case that excess mix water is forced out of the system during penetration into the ground, via the phenomenon of pressure filtration, so that the in situ grout likely has a composition closer to that of the Primary mix.

The Primary grouting of each micropile is always conducted as a continuous operation to ensure the structural continuity of the grouting and prevent "necking."

QA/QC AND TESTING

During Installation

Full details are to be maintained through all the construction processes to ensure the final quality of the product. Of particular importance is the recording of all relevant grout pressure-volume-depth-time data, since to a large extent, the grouting process is the major construction determinant of the grout/ground bond capacity. Certain contractors also favor testing the fluid grout (e.g., specific gravity, fluidity) prior to injection, to ensure that the injected grout meets the specifications, since samples for strength testing give only retrospective proof of the ability of the grout to reach the specified quality.

After Installation

For axially-loaded CASE 1 piles, load tests are conducted on a representative number of elements. It is common to use ASTM D 1143-81 (Compression) and ASTM D 3689-87 (Tension) [ASTM 1995], although the information yield from both can be greatly expanded by incrementally cycling the load, in the fashion of Performance Anchor testing (PTI 1996). As shown in Fig. 2.6-8, such testing permits the total pile movements to be partitioned into permanent and elastic components, so allowing fundamental investigations into load transfer mechanisms. CASE 1 piles subjected to lateral loading can be tested according to ASTM D 3966-81.

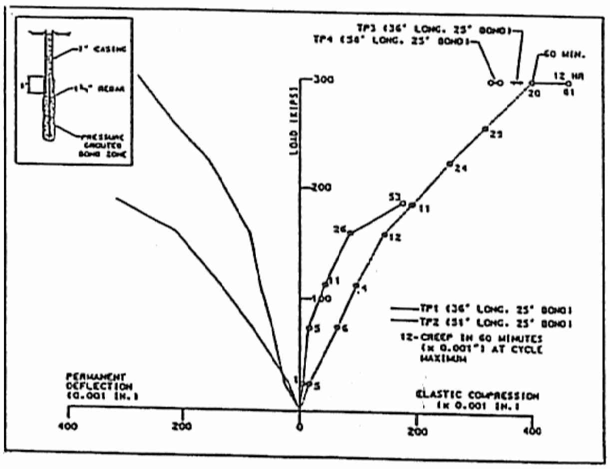

Fig. 2.6-8. Elastic/permanent movement performance of Test Pile 1 and Test Pile 2, Postal Square, Washington, D.C. (from Bruce 1992). 1 in = 25.4 mm, 1 kip = 4.48 kN.

There is no common, absolute set of acceptance criteria for CASE 1 axially-loaded piles, although many "solutions" based on geometric analyses of load-total movement curves have been proposed (Kulhawy et al. 1991). Optimally, the acceptance criteria are selected project by project, with respect to short-term movement, and creep performance. Analyzing pile load test data to meet these criteria is best conducted with the full insight afforded by cyclic loading programs.

CASE 2 piles, being part of a composite soil-pile mass are less meaningful to test individually. Rather the behavior of the whole composite structure is monitored, for example by inclinometers (in the case of a slope stabilization application) or movement gauges (in the case of structural stability or settlement reduction applications). Instrumentation of individual piles has been carried out (Palmerton 1984) but the data have typically proved difficult to analyze, given the lack of knowledge of the actual performance of such structures.

THE FUTURE

In the United States, as is the case worldwide, new geotechnical and structural challenges for both static and seismic retrofit are fostering the continuing growth of micropile technology. In particular, the demands of seismic engineering are provided

new impetus to the study and understanding of pile performance, in general, and pile networks especially.

Aided by the classification breakthrough made by FHWA (1996), researchers in the United States, France, and Japan are poised to close the gap that still exists between the level of analytical understanding, and the excellence of the construction, testing, and performance knowledge. One consequence will be a rapid growth in the application of CASE 2 structures, optimally and rigorously designed to ensure efficient and economic solutions especially for seismic applications.

The relative ease of global information retrieval and exchange systems, coupled with the momentum established by micropile researchers in the mid 1990s will ensure that developments in micropile technology will continue apace, and provide a fitting reflection of the foresight of their progenitor, Fernando Lizzi.

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