Drilled and Grouted Micropiles:

State-of-Practice Review

Volume I: Background, Classifications, Cost

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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 first in a series. The other volumes in the series are:

FHWA-RD-96-017 Volume II: Design
FHWA-RD-96-018 Volume III: Construction, QA/QC, and Testing
FHWA-RD-96-019 Volume IV: Case Histories

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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)
Dywidag Systems International (Germany)
Fondedile SpA (Italy)
Rodio (Italy)
Soletanche (France)
Bachy (France)
Franki (South Africa, via Dr. Ross Parry-Davies)
Insond (Austria)
University of Pittsburgh (U.S.A.)
Cornell University (U.S.A.)
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. INTRODUCTION

BACKGROUND

The technology of micropiling was conceived in Italy in the early 1950's and was introduced more than 2 decades later into the United States. Since the mid-1980's in particular, there has been a rapid growth in the use of this technology, mainly as foundation support elements in static and seismic applications and as in situ reinforcement for slope and excavation stabilization.

Many of these uses are directly related to transportation projects. Therefore, in 1993, the Federal Highway Administration (FHWA) decided to fund this study into the technology of micropiling. This decision largely reflected the industry's growing awareness of the potential of micropiling as a means of resolving difficult foundation and slope stability problems. However, it also underlined the desire of the FHWA to be a cooperative partner to their French colleagues who, in 1993, had commenced a new 5-year national project named "FOREVER" (Fondations Renforcées Verticalement). The FOREVER project was organized under the aegis of the Institute for Applied Research and Experimentation in Civil Engineering (IREX) and is under the technical direction of Professor François Schlosser of the National Civil Engineering School (ENPC) and Dr. Roger Frank of the Center for Education and Research in Soil and Rock Mechanics (CERMES). It is also supported by the National Public Works Federation (FNTP), the Center for Studies and Research on Construction and Public Works (CEBTP), the system of Civil Engineering Laboratories (LPC), and a collection of other research and testing bureaus, businesses, contractors, and owners. (Acronyms in parentheses reflect the French names of the organizations.) FOREVER includes studies, numerical modeling, laboratory testing (centrifuge), and full-scale field testing. Its chief objective is to promote the use of micropiles in all fields: deep foundations of new buildings and structures, stabilization of slopes and embankments, "consolidation" of existing foundations, retaining walls, reduction of embankment settlement, and shallow foundations.

The major tasks of the FHWA study, as set in the "State-of-Practice Request for Proposals," were defined as follows:

<u>Task A. State-of-Practice Determination</u>

A.1. Comprehensive review and detailed analysis of the available research and development results, laboratory and field testing data, and site observations and monitored case studies to establish a comprehensive engineering knowledge base.

A.2. Critical assessment of the available analytical models and design methods for single piles, groups of piles, and networks of reticulated piles under both axial and lateral loading conditions and applications.

A.3. Comparisons and analyses of currently used construction specifications and quality assurance procedures for each installation technique.

Task B. Research Needs Assessment

B.1. Evaluation of the limitations and uncertainties in the current state of practice, and development of relevant research programs to effectively address the identified engineering research needs.

Task C. Research Coordination with Foreign Programs

C.1. Recognizing the major role of European specialists, it was decided to involve them as fully as possible, via the creation of an International Advisory Board (for written and conference contributions). The Principal Investigators would undertake a series of visits to other specialists in Europe.

Tasks D and E

Referred to as the Draft Final and Final Report processes, respectively.

The contract was awarded to Nicholson Construction Company, with Dr. Donald Bruce and Professor Ilan Juran of the Polytechnic University of Brooklyn (New York) being nominated as the two Principal Investigators. The International Advisory Board initially consisted of Dr. Fernando Lizzi (Italy), Professor Schlosser (France), Professor Stuart Littlejohn (United Kingdom), and Dr. Thomas Herbst (Germany), although it was later supplemented by other specialists, including Mike Turner (United Kingdom), Professor Fred Kulhawy (Cornell University, New York), James Mason and Ray Zelinski (California Department of Transportation [Caltrans]), Professor Reidar Bjorhovde (University of Pittsburgh, Pennsylvania), and Bob Lukas (Ground Engineering, United States). This group provided a blend of contractor, consultant, academic, and client that mirrored the team assembled in France.

The proposed task and progress schedule is shown as figure 1, while the as-built schedule, reflecting the various changes and extensions that occurred during the program, is shown as figure 2.

As explained in the subsequent sections of this chapter, the subject of the study can be referred to generically as small-diameter drilled and grouted piles. They have been used throughout the world for various purposes, and this has spawned a profusion of local names, including: pali radice, micropali (Italian); pieux racines, pieux aiguilles, minipieux, micropieux (French); minipile, micropile, pin pile, root pile, needle pile (English); Verpresspfähle, Kleinbohrpfähle, Wurzelpfähle (German); and Estaca Raiz (Portuguese). All, however, refer to a "special type of small-diameter bored pile" (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 (figures 3 and 4). Inherent in their genesis and application is the precept that micropiles are installed with methods that cause minimal disturbance to structure, soil, or environment. This, therefore, excludes other related techniques from this particular study, such as those that employ percussive or explosive energy (driven elements), ultra-high flushing and/or grouting pressure (jet nails), or large-diameter drilling techniques that may cause lateral soil decompression (auger-cast piles).

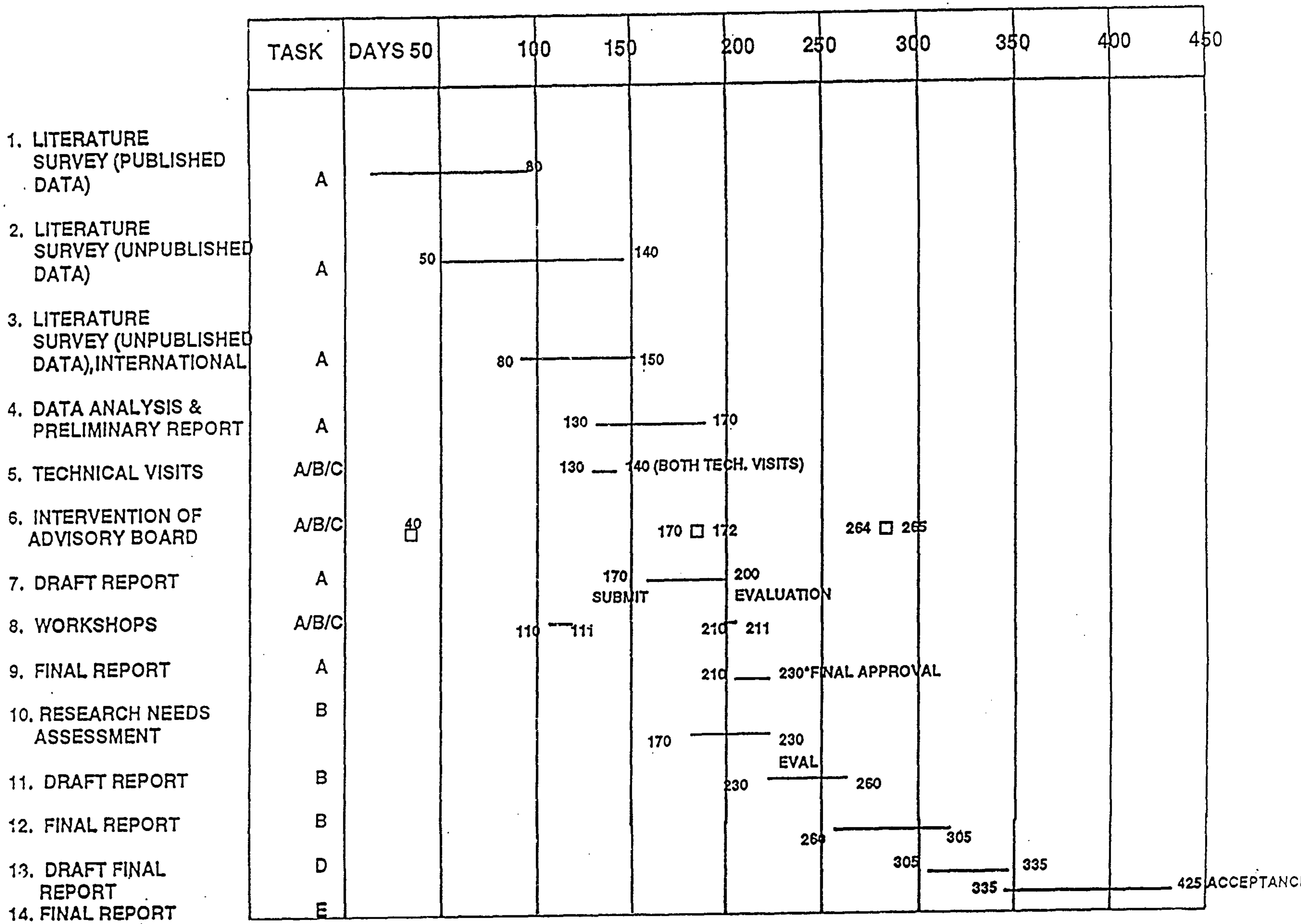


Figure 1. Proposed task and progress schedule.

	TASK	1993 NOV	1993 DEC	1994 JAN	FEB	MAR	APRIL	MAY	JUNE	JULY	AUG	SEP	OCT	NOV	DEC
1. LITERATURE SURVEY (PUBLISHED DATA)	A														
PLANNED%		38	76	100											
ACTUAL %		25	50	75	85	95	100								
2. LITERATURE SURVEY (UNPUBLISHED DATA)	A														
PLANNED %			12	47	78	100									
ACTUAL %		10	20	35	50	70	100								
3. LITERATURE SURVEY (UNPUBLISHED DATA) INTERNATIONAL	A														
PLANNED %			0	17	57	100									
ACTUAL %		2	10	20	40	50	100								
4. DATA ANALYSIS & PRELIMINARY REPORT	A														
PLANNED %						53	100								
ACTUAL %		5	10	20	30	34	50	70	80	90	100				
5. TECHNICAL VISITS	A/B/C														
PLANNED %						100									
ACTUAL %					30	49				75			100		
6. INTERVENTION OF ADVISORY BOARD	A/B/C														
PLANNED %			100				100			100					
ACTUAL %				100			100			0					
7. DRAFT REPORT	A														
PLANNED %							37	100							
ACTUAL %						10	20	40	50	70	80	90	95	95	95
8. WORKSHOPS	A/B/C														
PLANNED %					100	0		100					0		
ACTUAL %					0	100		0					100		
9. FINAL REPORT	A														
PLANNED %								10	100						
ACTUAL %									0	0	0	0	0	0	0
10. RESEARCH NEEDS ASSESSMENT	B														
PLANNED%							18	70	100						
ACTUAL %						5	15	30	40	50	60	70	80	90	95
11. DRAFT REPORT	B														
PLANNED %									40	100					
ACTUAL %											0	0	10	20	20
12. FINAL REPORT	B														
PLANNED %										30	100				
ACTUAL %												0	0	0	0
13. DRAFT FINAL REPORT	D														
PLANNED %												100			
ACTUAL %												0	0	0	0
14. FINAL REPORT	E												33	67	100
													0	0	0

Figure 2. Actual (as-built) task and progress schedule.

		1995								1996			
	TASK	JAN	FEB	MAR	APRIL	MAY	JUNE	JULY		JAN	FEB		
1. LITERATURE SURVEY (PUBLISHED DATA)	A								EVALUATION + FINAL APPROVAL				
PLANNED%													
ACTUAL %													
2. LITERATURE SURVEY (UNPUBLISHED DATA)	A												
PLANNED %													
ACTUAL %													
3. LITERATURE SURVEY (UNPUBLISHED DATA) INTERNATIONAL	A												
PLANNED %													
ACTUAL %													
4. DATA ANALYSIS & PRELIMINARY REPORT	A												
PLANNED %													
ACTUAL %													
5. TECHNICAL VISITS	A/B/C												
PLANNED %													
ACTUAL %													
6. INTERVENTION OF ADVISORY BOARD	A/B/C		0										
PLANNED %													
ACTUAL %			100										
7. DRAFT REPORT	A												
PLANNED %													
ACTUAL %		100	120										
8. WORKSHOPS	A/B/C			0									
PLANNED %													
ACTUAL %				100									
9. FINAL REPORT	A												
PLANNED %													
ACTUAL %		10	10	100									
10. RESEARCH NEEDS ASSESSMENT	B												
PLANNED%													
ACTUAL %		95	100										
11. DRAFT REPORT	B												
PLANNED %													
ACTUAL %		20	40	100									
12. FINAL REPORT	B												
PLANNED %													
ACTUAL %		0	0	50	95	100							
13. DRAFT FINAL REPORT	D												
PLANNED %													
ACTUAL %		0		30	85	95		100					
14. FINAL REPORT	E	0		0	0	0		0			100		

Figure 2. Actual (as-built) task and progress schedule (continued).

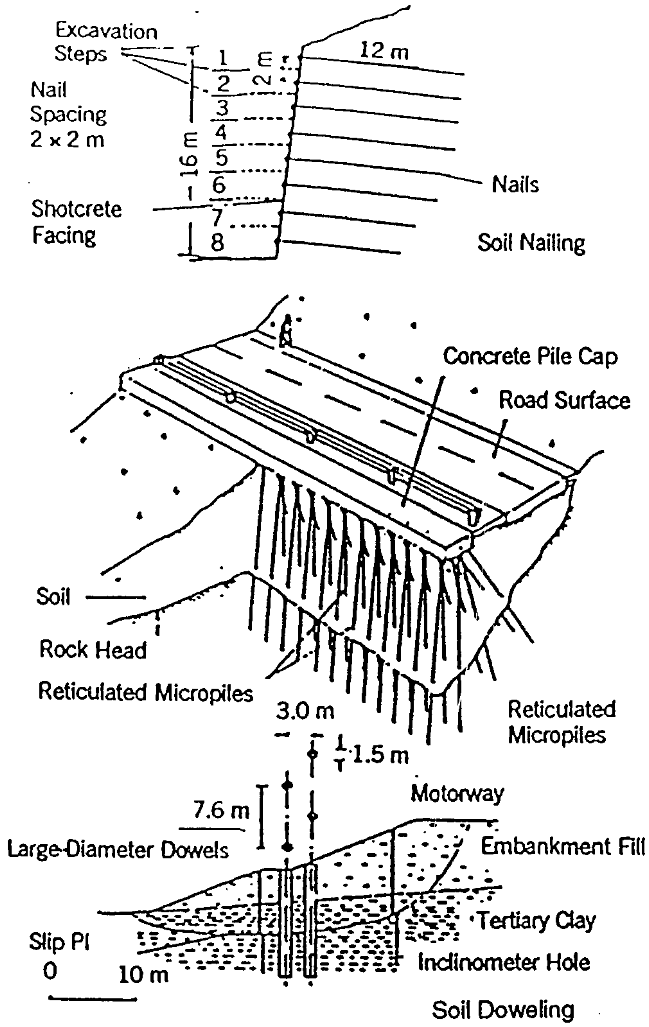


Figure 3. The family of in situ soil reinforcement techniques (Bruce and Jewell, 1986-1987).

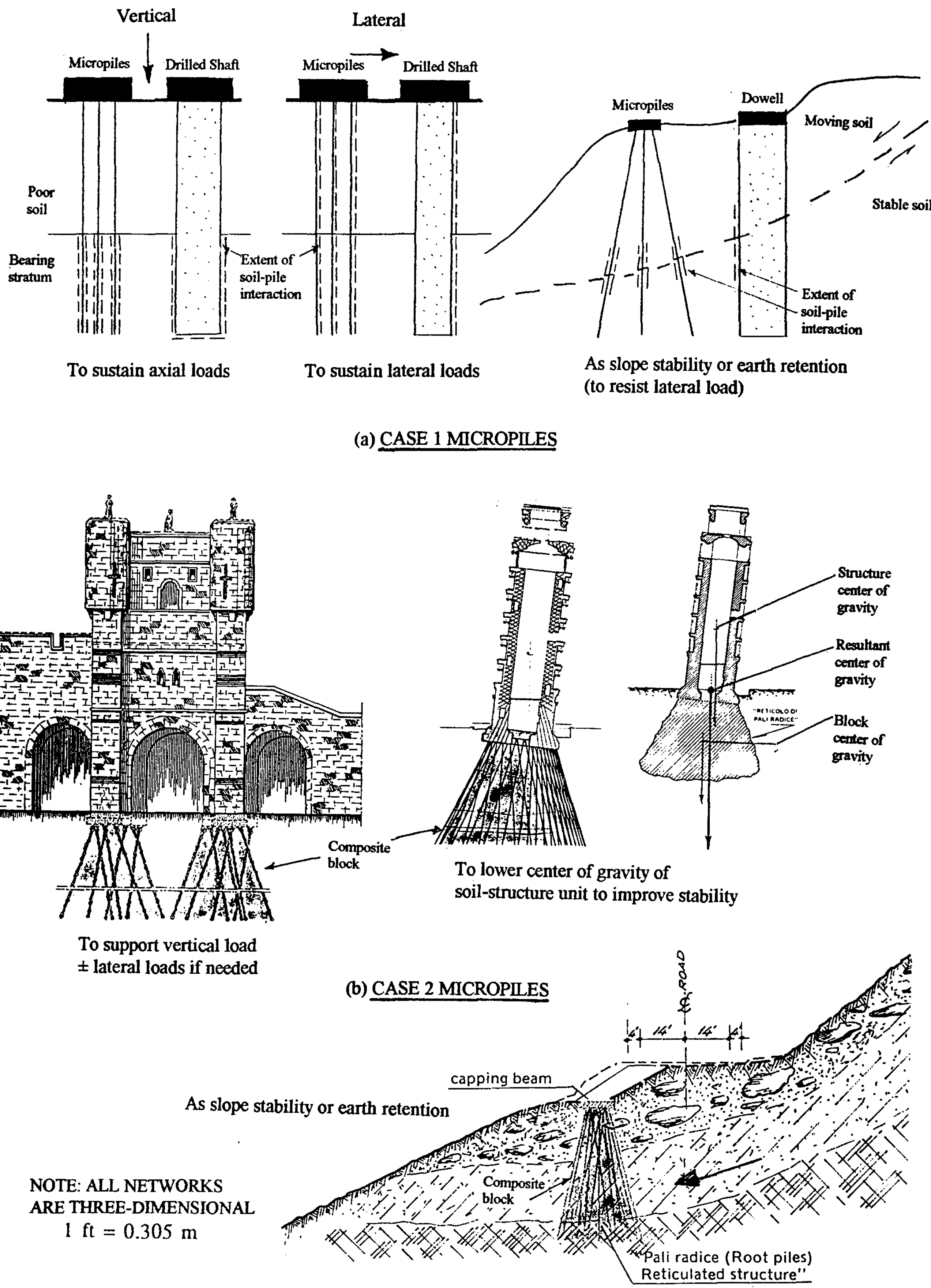


Figure 4. Fundamental classification of micropiles based on their supposed interaction with the soil.

Likewise, micropile construction techniques are among those used to install soil nails, sub-horizontal in situ reinforcements used in excavation support and slope stabilization (figure 3). However, soil nailing is regarded in concept, design, and function to be outside the scope of this report; in fact, it has already been the subject of major Federal (NCHRP, 1987; FHWA, 1994) and private studies (Juran and Elias, 1990; Bruce, 1993).

SCOPE AND INTENT OF THE REPORT

This report constitutes a state-of-practice survey based on a critical assessment of data published worldwide on micropiles. As well as being a comprehensive introduction to the technology, the report is designed to assist the FHWA in establishing, at a future date, reliable design guidelines, construction specifications, and quality control procedures for the wide spectrum of micropile applications. It is also intended that the report will highlight future research needs. It must be emphasized, however, that it is not in the scope of this study to produce a design manual or a set of practice recommendations.

The characteristics, classification, and historical background of micropiles are described in chapter 1 of volume I, followed by a structured review of their applications in chapter 2. Chapters 3 and 4 of this volume deal with feasibility, costs, and contracting practices. Volume II examines design, analysis, and performance issues as related to single piles, groups of piles, and networks of piles. Volume III discusses construction, quality assurance and control, as related to specifications, and reviews the range of pile tests that can be conducted. Volume IV provides details of significant case histories from around the world, but with an emphasis on domestic examples. The assessment of research needs and a proposal as to how they could be satisfied are provided in a different volume.

The report describes the use of micropiles for both structural support and in situ reinforcement. Both static and seismic loading conditions are examined.

DEFINITIONS AND CHARACTERISTICS

The generic classification of piling methods and systems proposed by Fleming et al. (1985) is shown in figure 5. Piles that 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, the 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, such as those developed in related geotechnical practices as ground anchoring, permit high soil/grout bond values to be generated along the micropile's periphery. To exploit this potential benefit, 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. Early

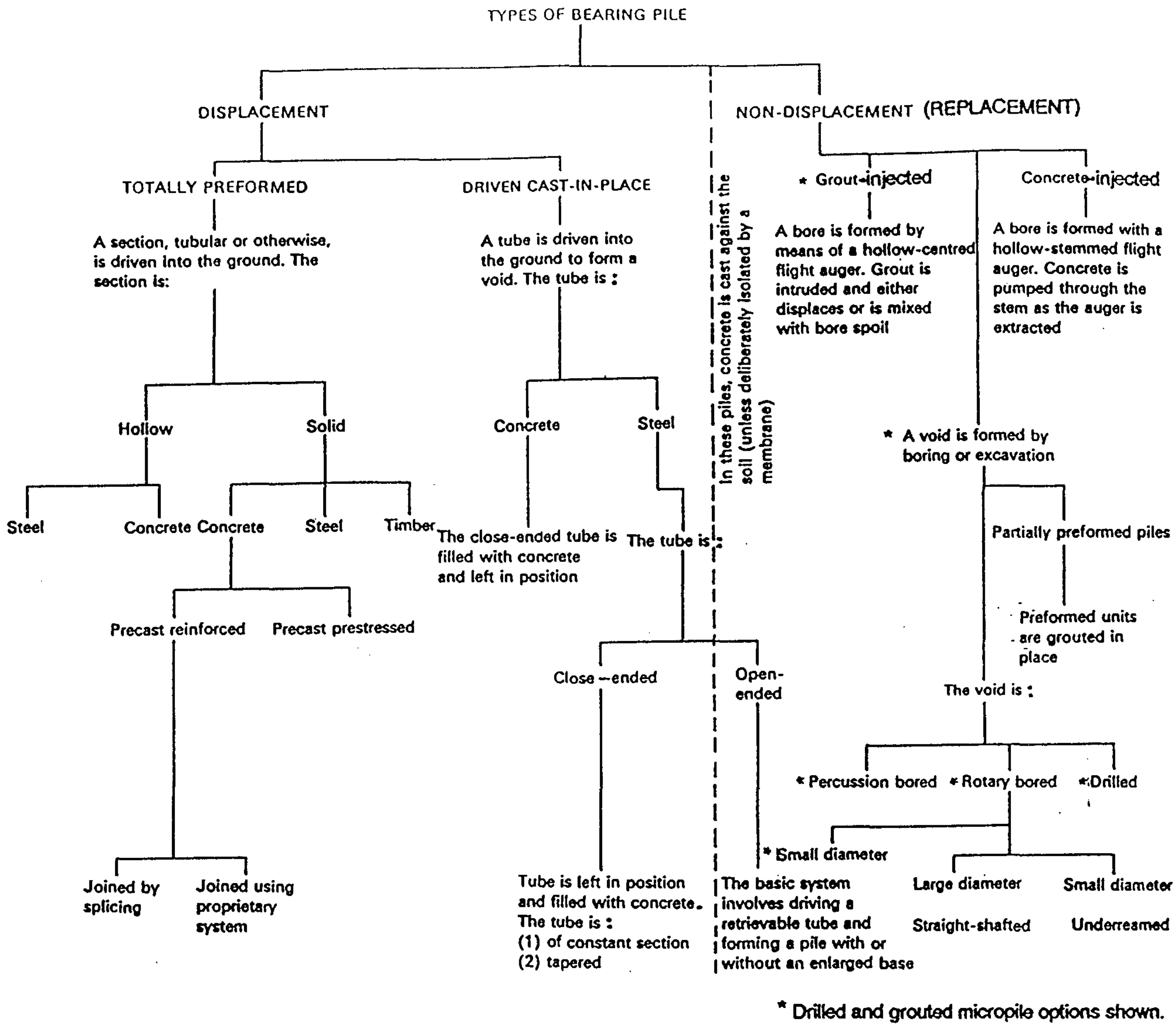


Figure 5. Classification of bearing pile types based on construction and current practice. [Note: Small diameter taken to be $\leq$ 600 mm (24 in); large diameter taken to be > 600 mm (24 in). Fleming et al. (1985), Weltman and Little (1977).]

micropile diameters were around 100 mm, but with the development of more powerful drilling equipment (figure 6), diameters of up to 300 mm are now considered practical. Thus, micropiles are capable of sustaining surprisingly high loads (compressive loads of more than 5000 kN have been recorded), or conversely, they 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 horizontal (figure 7). Micropiles are, therefore, used widely for underpinning existing structures, and the equipment can be further adapted to operate in locations with low headroom and severely restricted access (figure 8).

Figure 6. Example of contemporary diesel-hydraulic or electro-hydraulic track rig used for micropiling.

All of these observations of the traditionally recognized characteristics of micropiles lead to a fuller definition of a micropile: "a small-diameter (less than 300 mm[less than 250 mm in France]), replacement, drilled pile composed of placed or injected grout, and having some form of steel reinforcement to resist a high proportion of the design load." This load is mainly (and initially) accepted by the steel and transferred via the grout to the surrounding rock or soil using high values of interfacial friction with minimal end-bearing component, as is the case for ground anchors (FHWA, 1984) and soil nails (DFI, 1988). They are constructed using the type of equipment used for ground anchor and grouting projects, although micropiles often must be installed in low-headroom and/or difficult-access locations. They must be capable of causing minimal damage to structure or foundation material during installation and must be environmentally responsive. The majority of micropiles are between 100 and 250 mm in diameter, 20 to 30 m in length, and 300 to 1000 kN in compressive or tensile service load, although far greater depths and loads are not uncommon in the United States. Figure 9 shows that the ratio of the circumference to the cross-section area is extremely high, thus, the perimetric area governs the load transfer mechanism, namely skin friction, as opposed to end bearing, as discussed in Volume II.

Figure 7. Diesel-hydraulic track rig drilling inclined micropiles.

Figure 8. Electro-hydraulic track rig with short mast, used for micropiling in low-headroom conditions. [Note: Tracks withdraw inwards to permit rig movement through narrow openings.]

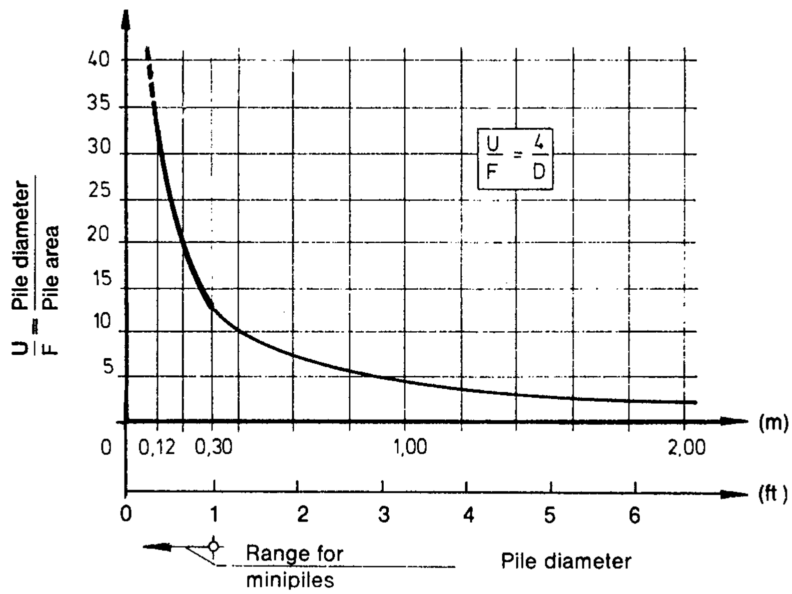


Figure 9. Ratio of pile circumference to pile cross section (DSI, 1992).

CLASSIFICATION

It is common to find micropiles sub-classified according to diameter, constructional process, or nature of the reinforcement. However, given the definition of a micropile provided in Definitions and Characteristics, above, this report concludes that a new and rigorous classification should be adopted based on two criteria:

- Philosophy of behavior.
- Method of grouting.

The former criterion dictates the basis of the overall design concept, and the latter is the principal determinant of pile/ground bond capacity.

Classification Based on Philosophy of Behavior

As detailed and illustrated in subsequent chapters, micropiles are usually designed to transfer structural loads to more competent or stable strata. Therefore, they act as "substitutes" or alternatives for other conventional pile systems (figure 4a). For axially loaded piles, the pile/ground interaction is in the form of side shear and so it is restricted to that zone of ground immediately surrounding the pile. For micropiles used as in situ reinforcements for slope stabilization, recent research by Pearlman et al. (1992) suggests that pile/ground interaction occurs only relatively close to the slide plane, although above this level, the pile group may also provide a certain degree of continuity to the pile/ground composite structure. In both cases, however, the pile (and more correctly the reinforcement) directly resists the applied loads. This is equally true for cases when individual piles or groups of piles are used. In this context, a group is defined as a tight collection of piles, each of which is subjected to direct loading. Depending on prevailing codes, the individual pile design capacity may have to be reduced to conform to conventional "reduction ratio" concepts usually associated with driven piles, although this restriction is never enforced for micropiles given their mode of construction, which tends to improve, not damage, the inter-pile soil (volume II).

When axially loaded piles of this type are designed to transfer their load only within a remote founding stratum, pile head movements will occur during loading in proportion to the length and composition of the pile shaft between the structure and the founding stratum. In this instance, the pile can be preloaded to ensure that the structure can be supported without further movement or settlement being necessary. Also, if suitably competent ground conditions exist all the way down from below the structure, then the pile can be fully bonded over its entire length and thus movements under load will be smaller than in the previous case (figure 4a).

In this report, such directly loaded piles, whether for axial or lateral loading conditions, are referred to as CASE 1 elements. They comprise virtually all North American applications to date and at least 90 percent of all known international applications.

On the other hand, we may distinguish the small group of CASE 2 structures. Historical Development of Micropiles describes how Dr. Lizzi introduced the concept of micropiling when he patented the "root pile" (palo radice) in 1952. The name alone evoked the concept of support and stabilization by locking onto a three-dimensional network of reticulated piles, similar to the root network of a tree. This concept involves the creation of a laterally confined soil/pile composite structure that can work for underpinning, stabilization, and earth retention, as illustrated in figure 4b. The piles are not heavily reinforced since they are not individually and directly loaded. 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. Research (volume II) has suggested that a positive "network effect" is achieved in terms of load/movement performance, such as the effectiveness and efficiency of the reticulated pile/soil interaction producing the composite mass.

It is clear, therefore, that the basis of the design for a CASE 2 structure is radically different from a CASE 1 pile (or group of piles). Notwithstanding this difference, however, there will be occasions where there are transitional applications 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.

The 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 focus is CASE 2 piles, was understandably an opponent of the technique of preloading high-capacity micropiles, such as those described by Mascardi (1982) and Bruce (1992). Now that these latter piles are recognized as being of a different class of performance, in which complete pile/soil contact and interaction are not fundamental to their proper behavior, the issue is correctly and honorably resolved.

Classification Based on Method of Grouting

Chapter 3 details the various steps in constructing micropiles. They are:

- Drill.
- Place reinforcement.
- Place grout (usually involving extraction of temporary drill casing).

There is no question that drilling method and technique will affect the scale of the grout/ground bond that can be mobilized. On the other hand, the act of placing reinforcement should not influence this bond development. Overall, however, international practices in both micropiles (e.g., French Norm DTU 13.2, 1992) and ground anchors (e.g., British Code BS 8081, 1989) confirm 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 figure 10.

- 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, are used. The pile drill hole may have an underreamed base (to aid performance in tension), but this is now very rare and not encountered in any other pile type.

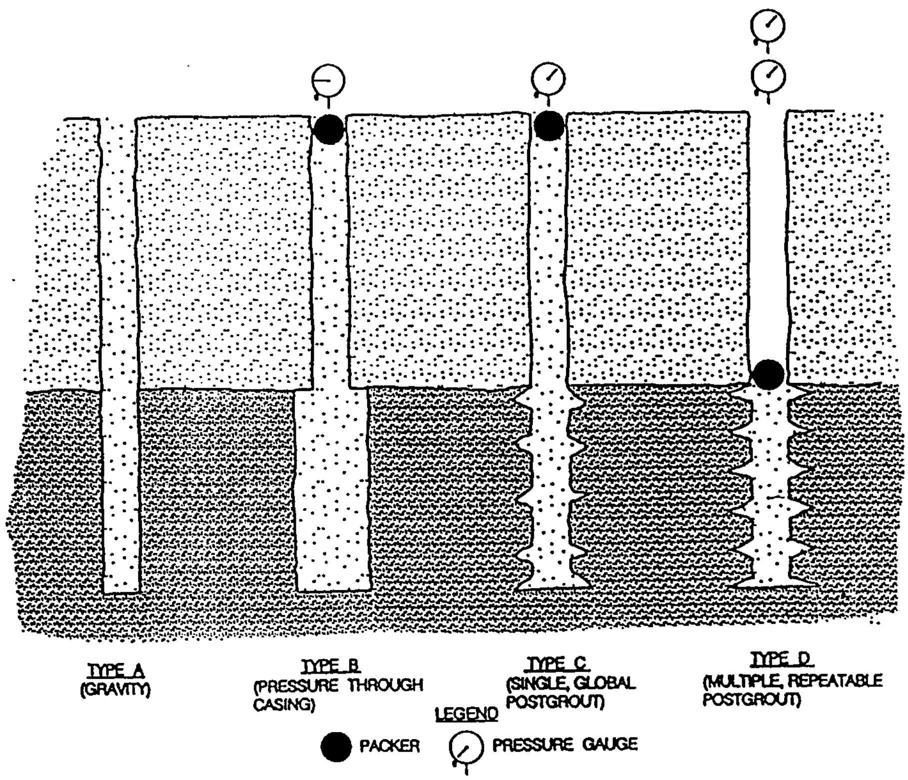


Figure 10. Classification of micropile type based on type of grouting.

- Type B: Neat cement grout is injected into the drilled hole as the temporary steel drill casing is withdrawn. Pressures are typically in the range of 0.3 to 1 MPa. They are limited by the ability of the soil to maintain a tight grout "seal" around the casing during its withdrawal and the need to avoid hydrofracture pressures and/or excessive grout consumption.

- Type C: Neat cement grout is placed in the hole, as done for Type A. Between 15 to 25 minutes later, 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. This type of pile, referred to in France as IGU (Injection Globale et Unitaire), is common practice only in that country.

- Type D: Neat cement grout is placed in the hole, as done 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. This is referred to in France as IRS (Injection Répétitive et Sélective), and it is a common practice worldwide.

Table 1 provides more details about this classification and it also indicates the relationship between other proposed classifications and terminologies. The relationship to pile capacity is explored in volume II, while volume III provides full descriptions of each aspect of these types.

Combined Classification

Micropiles referred to in this report are allocated a classification number denoting the philosophy of behavior (CASE 1 or CASE 2) that relates fundamentally to the "design" approach, and a letter denoting the method of grouting (Types A, B, C, or D) that 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. At this point, it may be interesting to note that the piles in each of the many case histories referred to in this report comfortably fall into this combined classification and, by so doing, have verified its selection.

HISTORICAL DEVELOPMENT OF MICROPILES

The concept of micropiles dates back to Italy in the early 1950's when innovative and reliable methods of underpinning historic buildings and monuments were being sought in that war-damaged country (Lizzi, 1982). Specifically, a system was needed that could accept structural loads with minimal movements and could be installed in confined working areas and in various soil types. In addition, it was essential that the construction method impose minimal adverse effects on the structure being underpinned or on adjacent structures.

In response to this need, the Italian specialty contractor Fondedile, under the technical direction of Dr. Lizzi, developed the "palo radice" (root pile), a small-diameter, drilled, cast-in-place, lightly reinforced grouted pile. These piles were ideally suited for underpinning applications. Their small diameters of around 100 mm permitted construction with small-sized rotary drill rigs that could be operated under restricted-access conditions and could drill through existing structures and subsoils with minimal disturbance. In addition, the injection of the grout, consisting of coarse sand, cement, and water, promoted high frictional bond with the surrounding soil. Such piles were tested to loads of more than 400 kN, and no grout/soil interfacial failures were recorded, although at the time, the anticipated load that was calculated conventionally for such a pile was only about 100 kN. The typical arrangement of pali radice as used for underpinning is shown in figure 11, which is extracted from Fondedile's first patent application of March 11, 1952.

That year also saw the first *application* of root piles, for the underpinning of the A. Angiulli school in Naples, Italy. The piles were 13 m long, 100 mm in diameter, and centrally reinforced by a 12-mm-diameter bar. The excellent load-holding performance of the first test pile (figure 12) in the volcanic ashes and sands at this site drew widespread professional attention, as did the publication of similar data from numerous sites thereafter. It is easy to concur with Dr. Lizzi's later assessment (1982) that "the introduction of 'Pali Radice' gave rise to a complete change in the field of underpinning."

The acquisition and publication of such test information, essential from a business development viewpoint, were facilitated by the relatively low cost of direct full-scale load tests in the field, and were driven by the innovative spirit of research into new applications. In contrast, most contemporary construction regulations (for example,

Table 1. Details of micropile classification based on type of grouting.

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 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	

in France and Germany) for cast-in-place bored piles permitted diameters in excess of 300 mm, only vertical installations, and very little opportunity for alternatives or innovation.

With the growing acceptance of the technique in international circles, the use of root piles spread quickly throughout Europe. For example, Fondedile introduced root piles into the United Kingdom in 1962, mainly to underpin historic structures threatened by decades of neglect. By 1965, similar systems had been used in West Germany in association with underground urban transportation schemes and, in the same decade, root piles were used during construction of parts of the Milan, Italy, subway. During this project, the Milan Subway Authority introduced the term "micropali" (micropiles) in reference to root piles, because the term "root piles" was proprietary. It is noteworthy that at the time of the introduction of micropiles to West German practice, the West German Code (DIN4014) for Bored Piles limited the capacity of a *400-mm-diameter pile* to the range of 300 to 370 kN, compared to demonstrated micropile capacities in excess of 1000 kN for 120- to 250-mm-diameter elements.

While the great majority of these applications were direct underpinning (CASE 1), the demands of urban engineering had encouraged the appearance of the CASE 2 "reticulated pali radice" (figure 13), the first full-scale tests of which were carried out in 1957. Such structures were then applied for slope stabilization, reinforcement of quay walls, protection of buried structures, and other soil and structure support and reinforcement needs, as described more fully in chapter 2.

Elsewhere, other contractors had begun to develop their own proprietary micropiles, such as the GEWI pile (first used at the Hoechst facility in Frankfurt, West Germany, by Dywidag in 1971), the Rodio Tubfix Micropile (first tested in Switzerland in 1962), and the pieu aiguille (Soletanche, 1974). These techniques were quickly exported overseas by branches or licensees of the original European contractors.

Refinements continue to be made by European contractors, driven by the need to provide highly engineered, high-quality solutions to progressively difficult construction challenges, in an extremely competitive economic climate. One example is the growing use of Type D piles, in which bond enhancements by high-pressure grouting are favored over bond area enlargement by enlarging the pile perimeter. Indeed, in France, very lightly reinforced or unreinforced Type A piles (equivalent to the original root pile) are no longer used for economic reasons. CASE 2 applications appear to be considered only in Italy and Japan, although increasing awareness of their potential is clear, especially in France.

Fondedile introduced micropiles into North America in 1973 by executing a number of CASE 1 underpinning jobs, principally in the New York and Boston areas. The first example of a reticulated CASE 2 structure was in 1975 for a structure to support and stabilize the abutment and pier foundation of a bridge along I-55 in Jackson, MS (figure 14). In November 1977, a similar structure was completed in the Mendocino National Forest, CA (figure 15) to stabilize a landslide area along Forest Highway 7. Both projects were instrumented by the U.S. Army Corps of Engineers, under contract with the Research Division of the FHWA (Palmerton, 1984).

By the mid-1970's, American specialty contractors who were previously engaged in drilling, grouting, and anchoring work began to develop their own variants, such as the Nicholson Pin Pile (Bruce, 1988), a Type A, B, or D highly reinforced pile. All

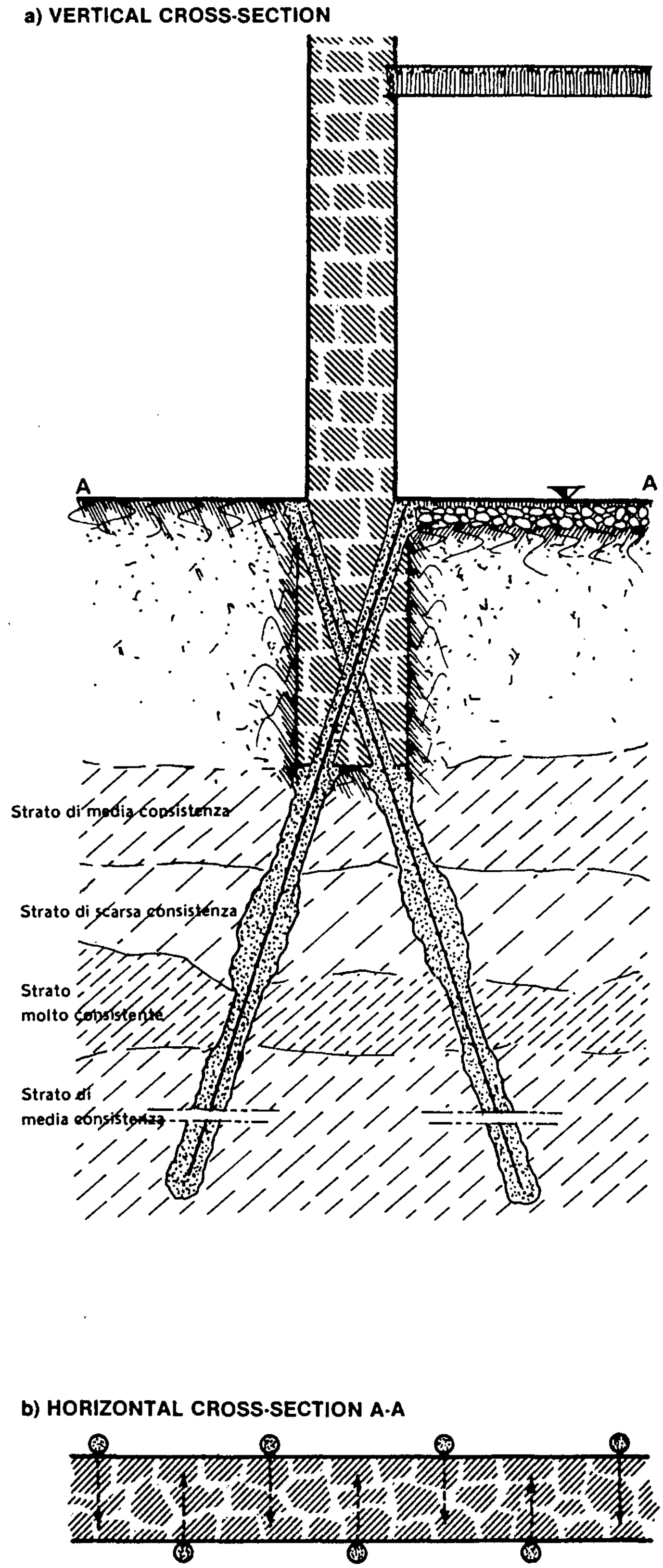
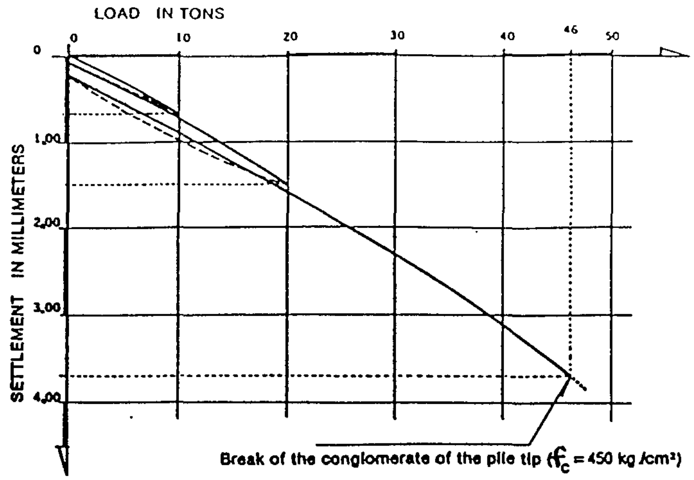


Figure 11. Classic arrangement of underpinning of a masonry wall using "pali radice" (from patent no. 497736, March 1952).



1 metric ton = 1.10 tons (short)

Figure 12. Load-movement data from the first root pile test, A. Angiulli school, Naples, Italy, 1952 (Lizzi, 1982).

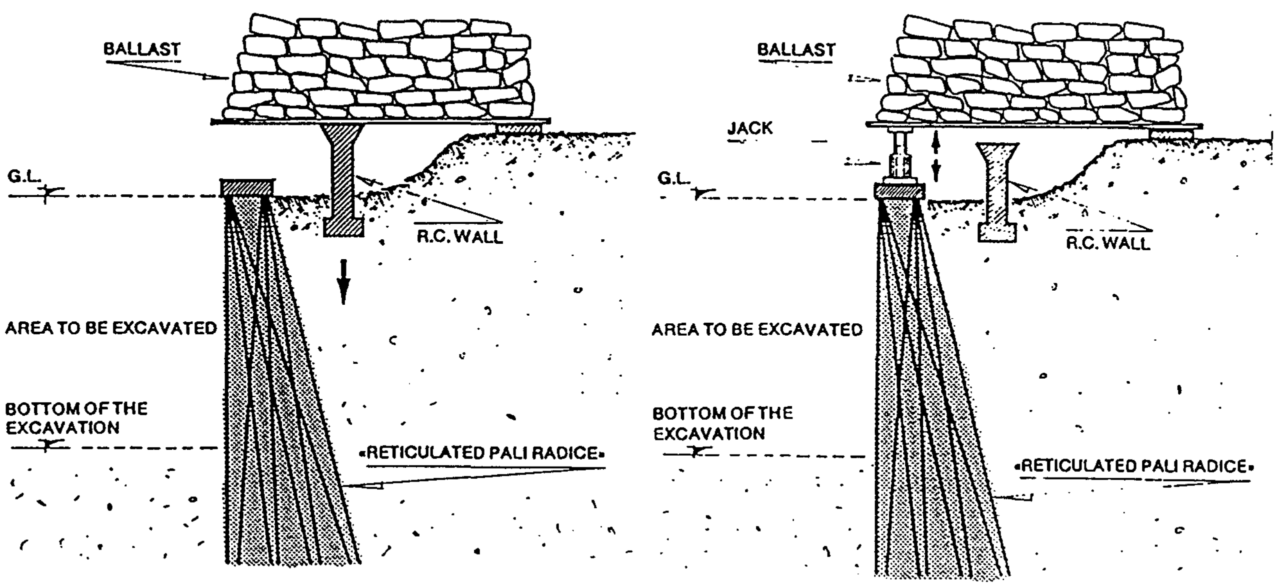


Figure 13. Early load tests on reticulated micropile structures: first phase - load being applied behind wall; second phase - load directly on the wall, Milan Subway, 1957 (Lizzi, 1982).

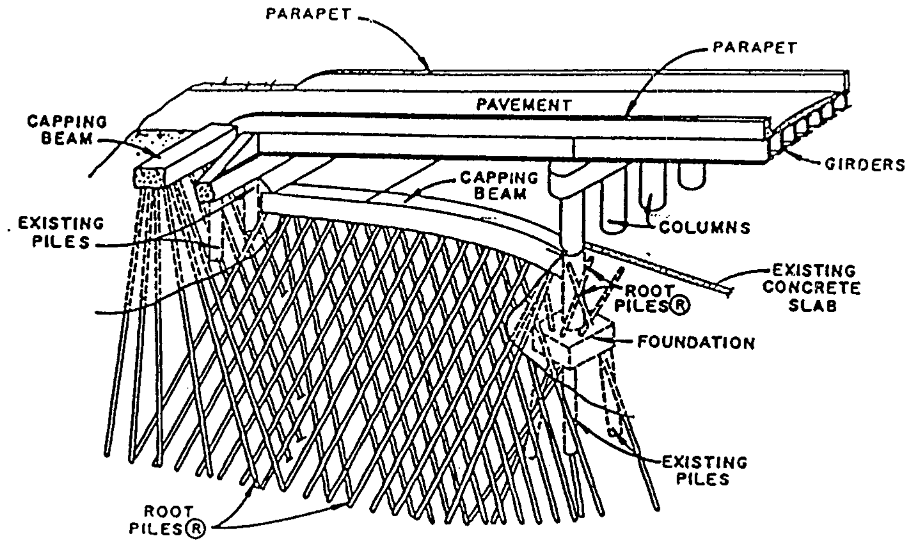
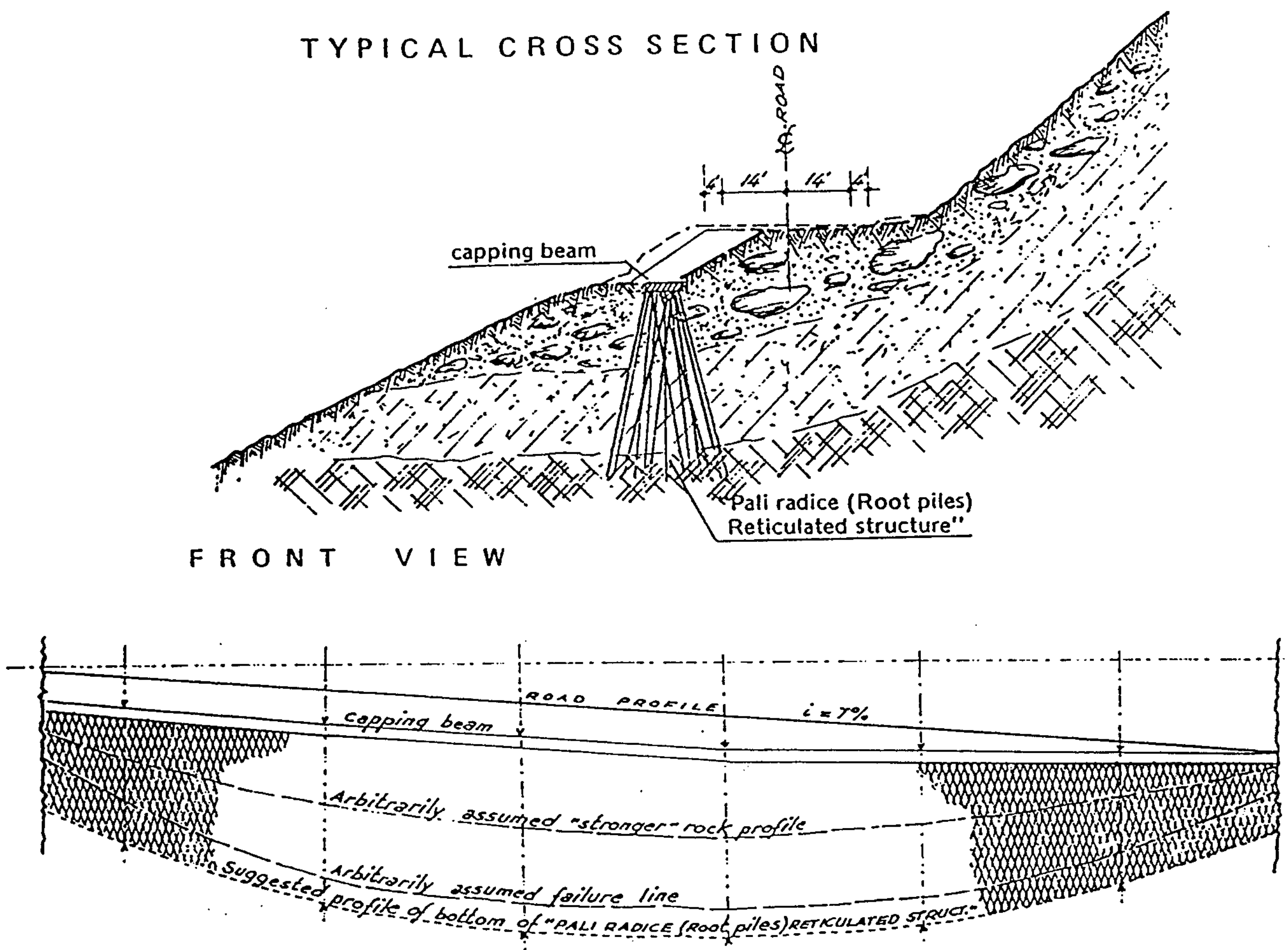


Figure 14. Reticulated micropile structure for abutment and pier support, Jackson, Mississippi (Lizzi, 1978).



1 ft = 0.305 m

Figure 15. Reticulated micropile structure for slope stabilization, Mendocino Pass, California (Lizzi, 1978).

applications were CASE 1. The skepticism of a traditional East Coast piling market, however, did not encourage rapid application of the technique, and indeed, by 1984, Fondedile decided to close their American venture for commercial reasons.

Ironically, the period from 1987 onwards then saw rapid growth as the pressure from innovative contractors, the weight of successful case histories, and the newly realized needs of consultants and owners working in old urban environments finally overcame the concerns of the traditionalists (Bruce, 1988a). As an illustration, Bruce (1994) listed 25 case histories of micropile projects completed by his company, in the United States, between the years 1978 and 1988. However, an additional 20 projects were completed in the subsequent 2 years. All of these applications were in the older urban areas of the East Coast or the "Rust Belt," or for industrial facilities in the Southeast. Since then, the number of applications conducted by a wide range of specialty contractors – for underpinning alone – has continued its exponential advance, with much activity now centering on seismic retrofit applications on the West Coast.

There has also been a significant, if less dramatic, growth in the use of micropile structures for slope stabilization, especially in the rural areas of the Appalachian Mountains, where more conventional solutions using large-scale equipment may not be feasible. These systems have been designed conceptually as CASE 1, although the legacy of the original Fondedile CASE 2 concept continues to influence aspects of design, as detailed in volume II. Further details of U.S. applications are provided in volume IV.

This contrast between micropile growth in Europe and North America in many ways reflected the situation in piling in general. In Europe, in the immediate post-war years, there was a shortage of steel, but an abundance of cheap (although often highly mechanically qualified) labor. Cast-in-place concrete piling, therefore, became popular, and in the absence of rigorous analytical expertise, industry leadership was in the hands of specialty geotechnical contractors. Designs relied heavily on the results of prior load test programs, while innovations were driven by the particular challenges posed by war damage and new urban infrastructure projects.

In North America, materials, especially steel, were generally cheaper and more readily available, although labor costs were significantly higher. Furthermore, there was no need for reconstruction programs, and the major capital works were typically outside the cities rather than inside them. This set of circumstances, therefore, favored the growth of the low-technology, prescriptive specification driven-pile market rather than encouraging the formation of specialty design/build companies.

Today the situation is similar throughout the world in terms of construction costs and technical demands. There are strong technical and academic centers, and responsive and supportive public administrations. Specialty construction companies have been founded, or have been exported, globally. These factors all have helped the growth of micropiles.

Regarding micropiles in other countries, in Canada, there has been little demand and very few applications. The activity level in Mexico is currently similar, although the problem there is less a shortage of applications rather than a shortage of funding. As in most other fields of specialty engineering, the potential in Mexico is considerable. Applications in South America have likewise been restricted by financial difficulties, although some large CASE 2 installations have been made, such as for a landslide prevention scheme along the Santos-São Paulo Highway in Brazil in 1977 (Lizzi, 1978).

According to Heinz (1994), "micropiling is alive and well in South Africa," based on technologies imported by European specialty contractors working with local partners. Micropiles are regarded as a "specialized tool from the geotechnical toolbox for solving particular problems" - an observation now common internationally. Applications vary from 300-mm-diameter high-capacity CASE 1 piles to 75-mm-diameter "simple" CASE 2 soil reinforcement, and piles of Types A, B, and D have been installed in a wide variety of soil and rock conditions.

OVERVIEW

In summary, it appears that micropiles are being used throughout the world and are regarded as a reputable construction tool of exceptional value and potential. While the underpinning of Europe's historic structures continues apace (e.g., Gouvenot, 1975; Herbst, 1982; Attwood, 1987; Doornbos, 1987), especially in cities being impacted by new underground construction, the newer and expanding cities of the Far East (Bruce and Yeung, 1983; Schlosser, 1994; Mitchell, 1985), South Africa, and South America are blossoming markets as sophisticated construction continues in soft soils, with high water tables in highly congested and populous areas, often under the threat of major seismicity. The market in the United States appears to be a combination of both, given the ongoing upgrading and refurbishment of industrial, transportation, and commercial structures, especially on the coastal belts. On a somewhat cautionary note, the FOREVER team sees two major research challenges to be met that could promote micropile growth: (1) there are no specific or general recommendations regarding long-term corrosion, and (2) "the tools for the design, dimensioning, and calculation of micropiles do not sufficiently take into account the effects of groups or networks (of piles) under a wide variety of stresses (static, cyclic, and seismic loads). Throughout their history, commercial and technological innovations have almost always preceded studies of fundamental performance and the development of design methods."

CHAPTER 2. REVIEW OF APPLICATIONS

INTRODUCTION

Micropiles are used in two basic applications: as structural support and as in situ soil reinforcement (figure 16). For direct structural support, groups of micropiles are designed based 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 Type A (gravity grouted, bond in rock), Type B (pressure grouted through the head), and Type D (post-grouted) are most commonly used.

For micropiles used as in situ reinforcement, the original reticulated network concept of Lizzi (CASE 2) featured low-capacity piles circumscribing and internally reinforcing a composite pile/soil gravity structure. Type A piles (gravity grouted, fully bonded in soil) typify these designs. Recent 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 typically are highly reinforced and of Type A or Type 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 [figure 4]). However, the potential is real and the subject is being actively pursued in the FOREVER program.

Table 2 summarizes the link between application, classification, design concept, and constructional method. It also provides an indication of how common each application is on a worldwide basis.

REVIEW OF APPLICATIONS

Further details of many of the projects introduced in this section are provided in volume IV, which observes the same organization as below. As shown in figure 16, there are two basic applications, namely structural support and in situ reinforcement.

Structural Support

Underpinning of Existing Foundations

Micropiles were originally devised for underpinning in the restoration of weakened, historic buildings, and were primarily installed as CASE 1 pile groups. Lizzi (1982) suggested that micropiles were an ideal underpinning tool for the following reasons:

- Their execution did not "...introduce, even temporarily, any undue weakness or overstress in the structure as well as the soil."
- They responded immediately to additional structural movement.

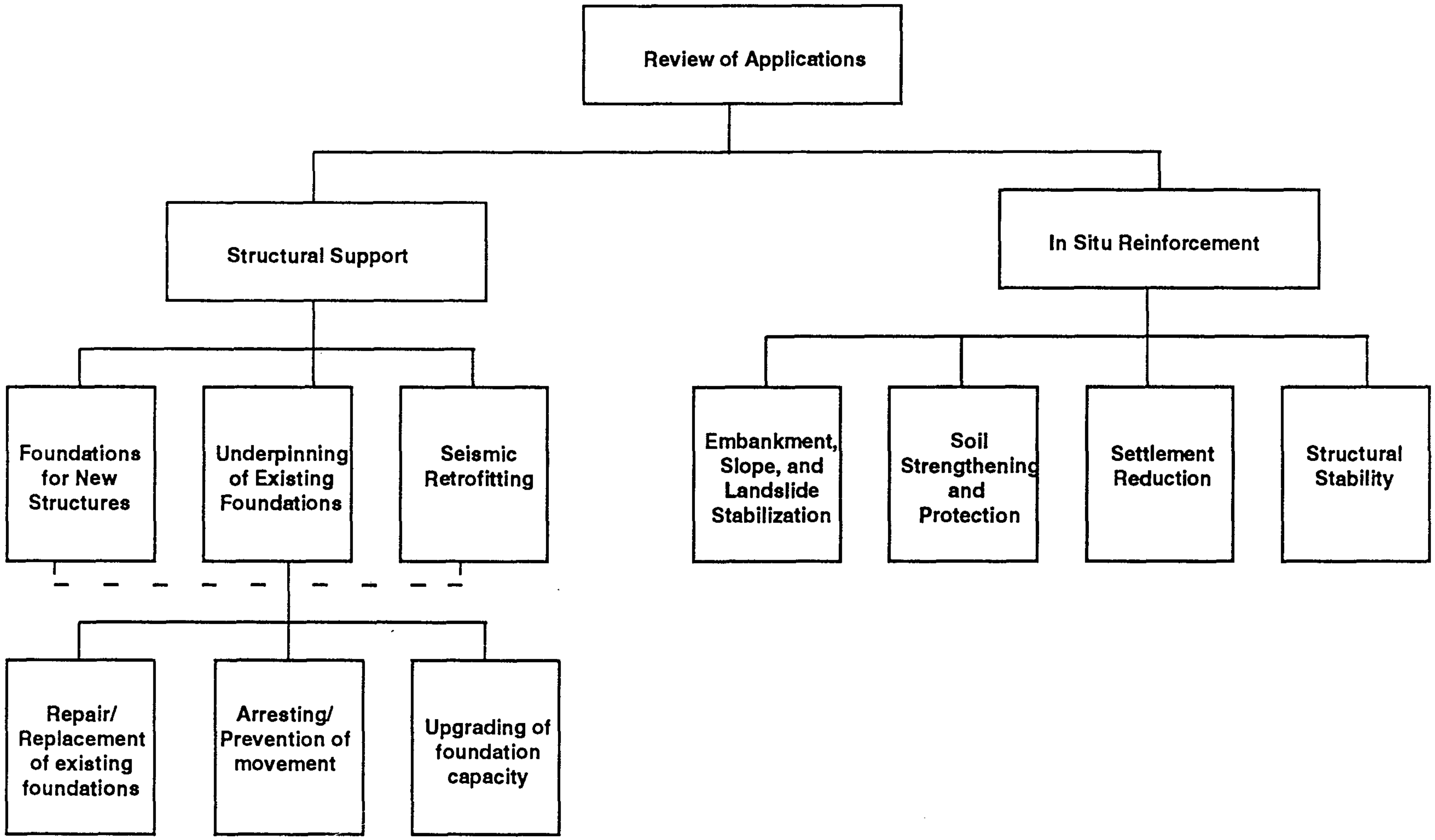


Figure 16. Classification of micropile applications.

- They enabled assessment of a new Factor of Safety.

Lizzi has also alluded to the possibility of using CASE 2 networks as indirect structural underpinning. However, it would seem that this application is, more precisely, in the realm of improving structural stability through creating and attaching an underlying pile/ground composite base, and, therefore, is introduced in the section on Structural Stability and is described in volume II.

Micropiles have been used for underpinning:

- To prevent or arrest structural settlements.
- To repair or replace deteriorating or inadequate foundations.
- To upgrade the load-bearing capacity of an existing foundation to permit, for example, the extension or raising of a structure.

Arresting or preventing structural movements

Structural movements can be caused by a variety of factors, including:

- <u>Poor ground beneath the existing foundation</u>. Munkfakh and Soliman (1987) describe the use of micropiles in the underpinning of a transit vehicle repair facility at Coney Island, New York, that had experienced major differential floor subsidence due to uneven settlements of an underlying soft soil layer. New heavy equipment was expected to accelerate these differential settlements. Micropiles were an effective solution because they could be easily and economically constructed to deeper soil strata of more competent bearing materials. In this case, micropiles were employed based on their proven performance; compatibility with existing soil; relatively low cost compared with other solutions; and, most importantly, because they could be installed while the facility remained in full operation (figure 17).

Another common application is to stabilize moving retaining walls (e.g., Attwood, 1987 and Lizzi, 1971). Since micropiles can be installed through, and bonded within, existing structures, they can provide a direct connection with a competent underlying horizon without the need for pile caps, while at the same time reinforcing the structure internally. Also, construction can be executed without compromising the existing stability of the wall.

- <u>Dewatering activities or groundwater table fluctuations</u>. Micropiles can be founded in deeper horizons where the effects of groundwater table lowering will not be felt. Therefore, nearer surface settlements will not cause structural movements, although a certain degree of separation between structure and subsoil surface may occur.

- <u>Adjacent deep excavations or tunneling activities</u>. Structural movements may be caused by adjacent deep excavations or underlying bored tunneling. In each example, the structure so threatened can be directly underpinned by CASE 1 micropiles, as described above. In addition, however, this movement potential can be eliminated by CASE 2 networks, as illustrated in Soil Strengthening and Protection.

Table 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) Types B and D in soil (Type C only in France)	Type A (CASE 1 and CASE 2) and Type B (CASE 1) in soil	Types 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%

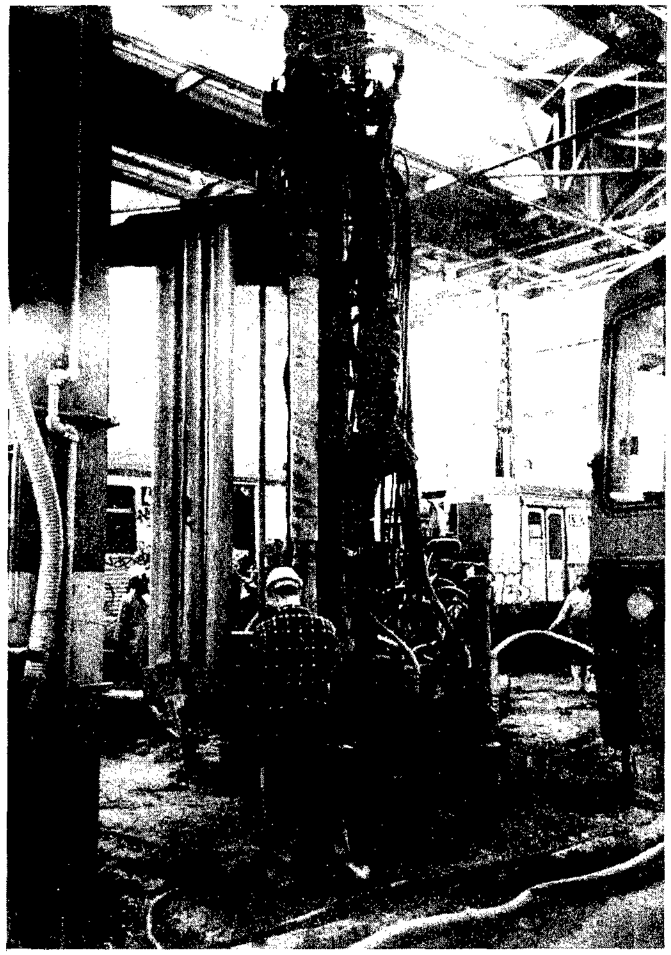
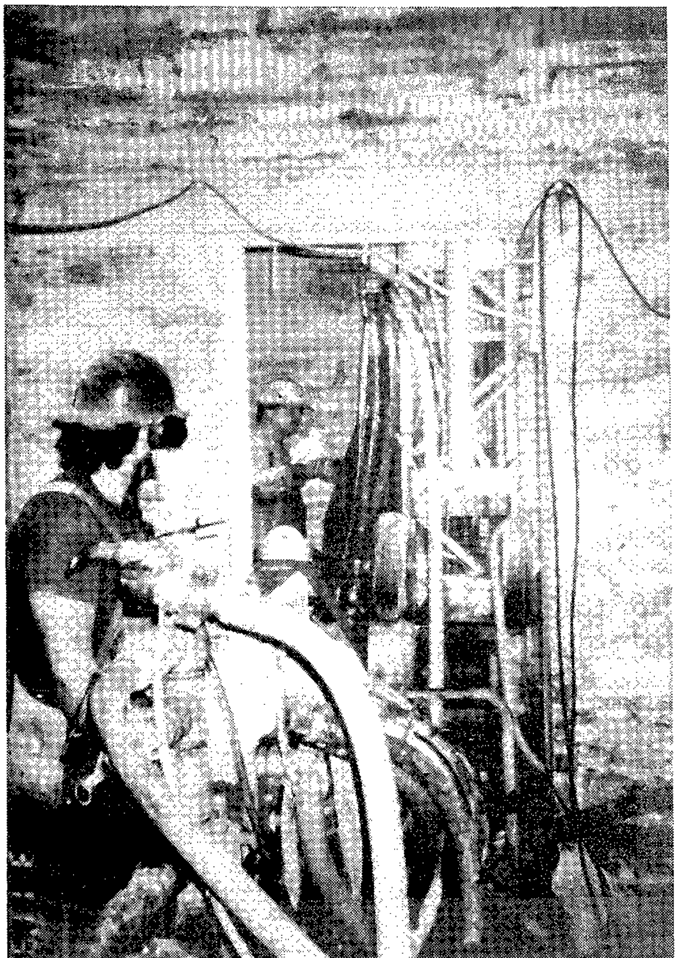


Figure 17. Installation of micropiles in a fully operational transit repair facility, Coney Island, New York.

Repair and/or replacement of existing foundations

Micropiles have been used to repair or replace existing foundations that have deteriorated or have otherwise proven inadequate due to improper design or construction, unforeseen loadings, or other factors, including fluctuating water tables. Groneck et al. (1993) describe the use of Type 1B micropiles to replace deteriorated timber piling at a grain export facility on the Columbia River in Washington State. It was constructed in the 1930's under a fast schedule, which did not allow time for the original timber piles to be treated with a preservative. Subsequently, severe deterioration of the piles occurred in the region above the water table, leading to differential settlement to an extent that threatened to shut down operations. Of the several underpinning options considered, only micropiles could economically accommodate construction in the tight-access and environmental conditions, in close proximity to existing piles, without interrupting facility operations, and without further threatening the stability of the structure (figure 18).

(a)



(b)

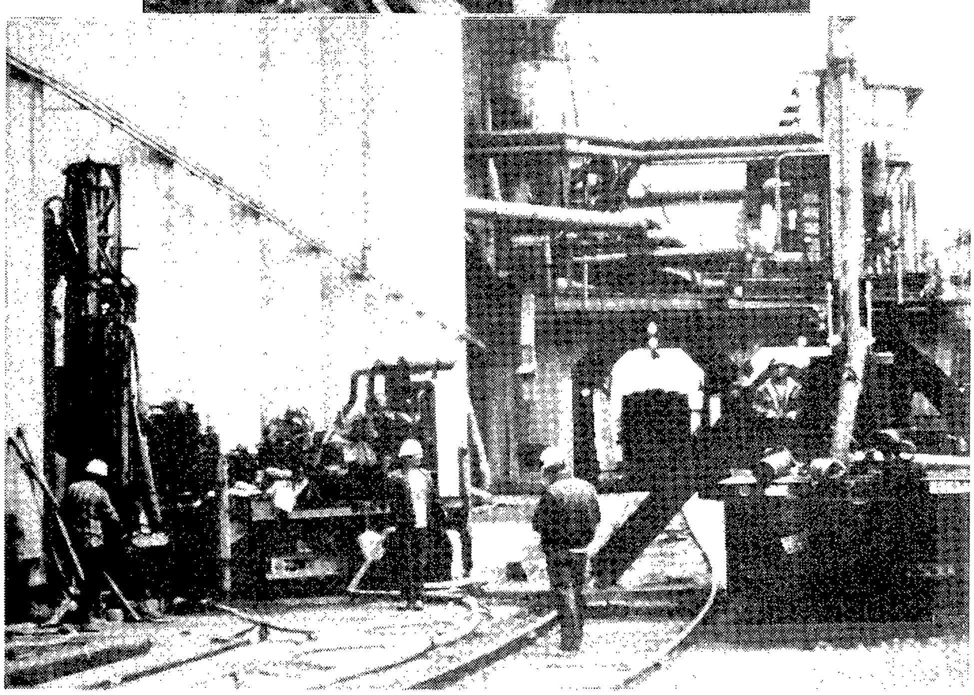


Figure 18. Installation of micropiles: (a) inside and (b) outside an operational grain facility, Vancouver, Washington.

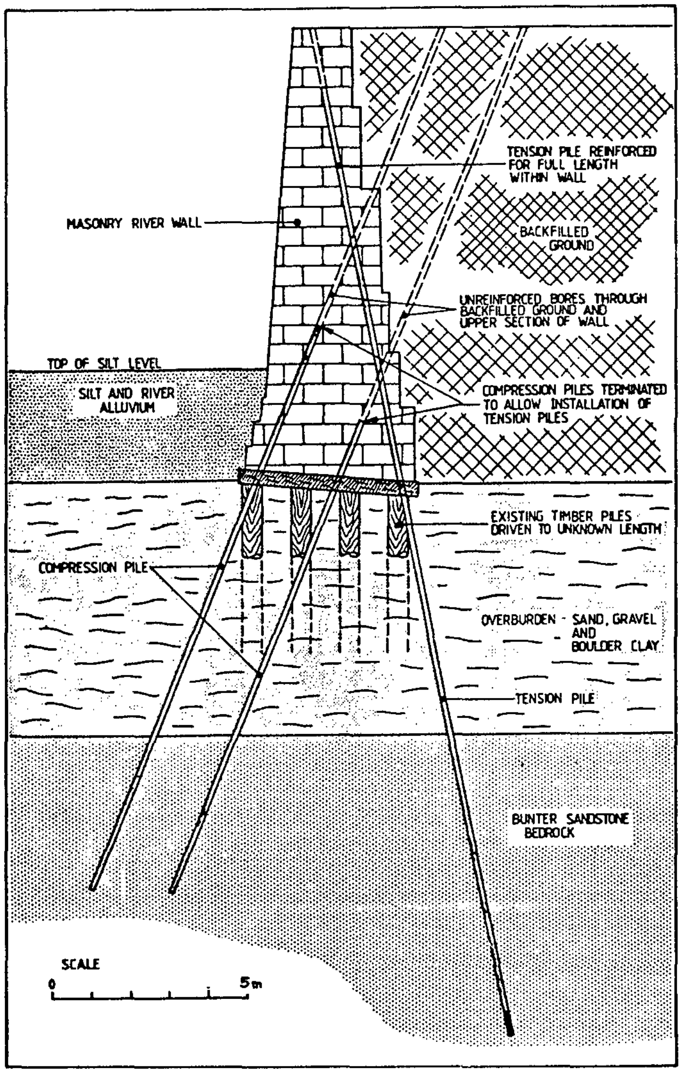


Figure 19. Typical cross section of reinforced and underpinned retaining wall at Albert Docks, Liverpool, United Kingdom (Turner and Wilson, 1990).

Turner and Wilson (1990) describe an application at Albert Docks in Liverpool, United Kingdom. By careful design of their orientation, the Type 1B micropiles were capable of withstanding both tension and compression loads (figure 19). They penetrated through the existing river wall and the underlying alluvium and boulder clay to be bonded into the underlying bedrock.

Bruce et al. (1990) describe the underpinning with micropiles of the Bascule piers of a 60-year-old bridge in Maryland, originally founded on wooden piles. River scour had exposed these piles in several places, leaving them vulnerable to attack and thus in need of replacement. Micropiles were employed for several reasons: their construction was possible both through the structure and the scour zone; their design and performance could be verified easily and economically through a test program; and through preloading, they could guard the bridge against further settlement during the transfer of structural load to these new foundation elements.

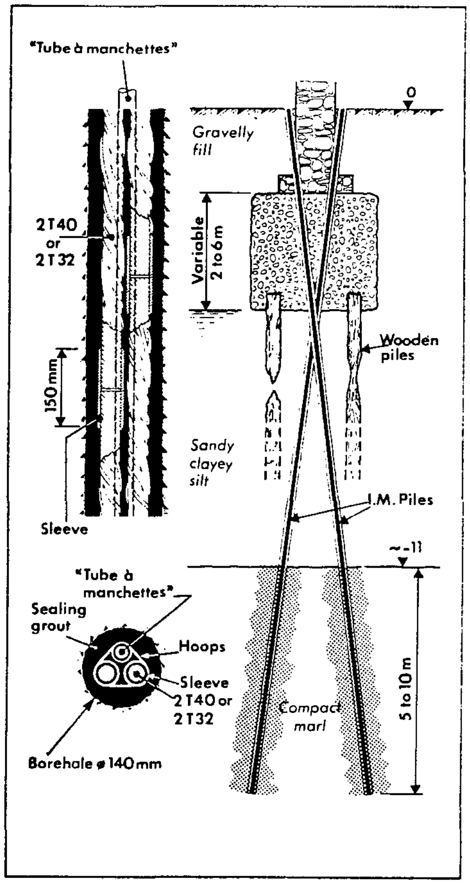


Figure 20. Typical cross section and details, Marseille Law Courts, France (Soletanche, 1986).

The Soletanche Company published a short case history of a similar application for the Marseille Law Courts Building (1986). The massive structure was more than 100 years old and was founded on concrete footings 2 to 6 m thick, bearing on wooden piles driven into sandy, clayey silt. A drop in groundwater level had caused deterioration and even destruction of the wooden piles and consolidation of the silt. Type 1D micropiles were selected to transfer the structural loads to a compact marl layer, about 11 m below the footings (figure 20). Each of the 605 piles was installed to a maximum depth of 10 m into the marl and was rated at a 320- to 500-kN service load. The work was completed in 20 months under difficult working conditions, operating from cellars, corridors, and offices in small areas at one time, so as not to interfere with activities in the building.

Upgrading of load capacity

The load capacity of a foundation system may have to be upgraded if the structure it supports is to accept additional load. For example, an existing bridge or elevated highway structure may have to be widened to accommodate increases in traffic volume, or additional stories may be desired on a building. Vibration-induced loadings may be anticipated from machinery newly placed within a structure.

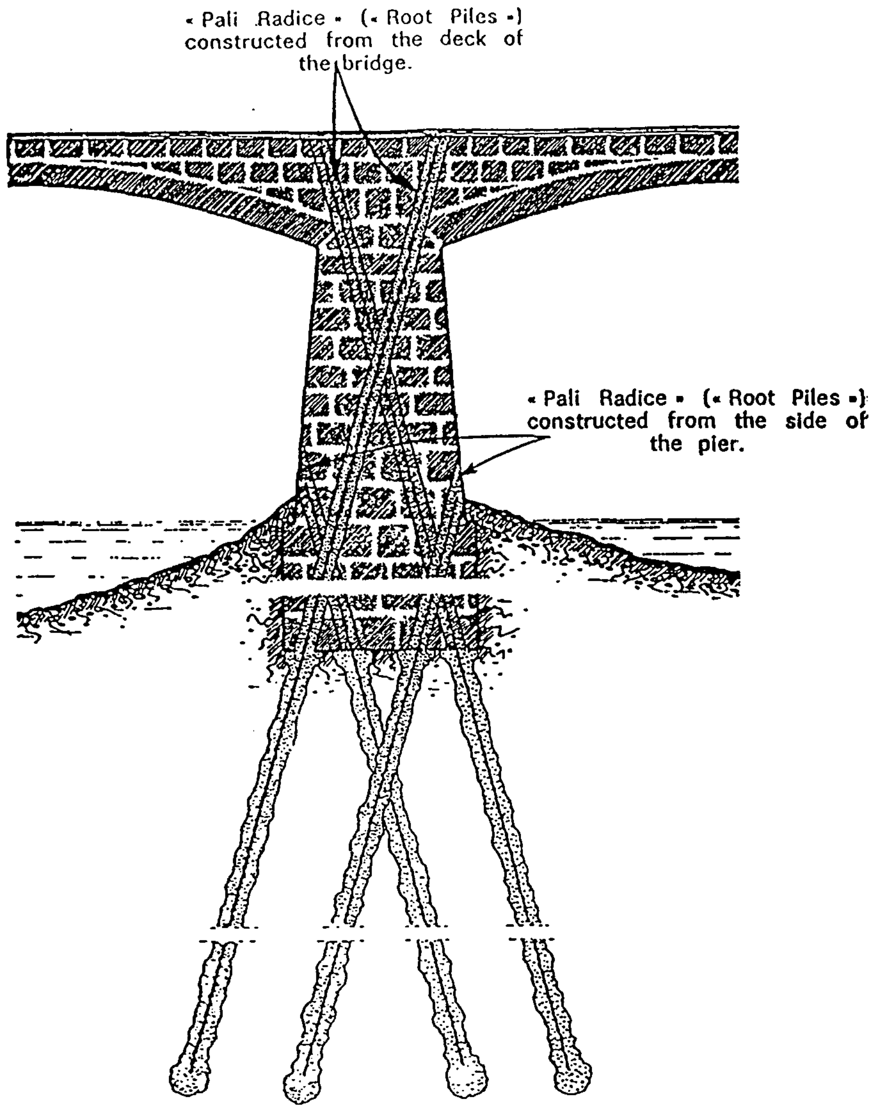


Figure 21. Underpinning of a bridge pier in Italy by root piles (Lizzi, 1971).

In most of these cases, however, access to existing foundations is restricted, with installation often required from within basements or beneath bridges or other elevated structures. Also, if the structure is located in an urban area, access may be further limited and additional restrictions may be imposed on permissible vibrations and noise levels during construction.

Micropiles are well suited to these conditions. For example, Lizzi (1971) describes the use of Type 1A micropiles to increase the load-bearing capacity of a bridge pier in Italy to accommodate widening of the bridge deck (figure 21). Micropiles were employed in this case because their construction would not disturb, vibrate, or shake the existing structure, and also because they could accommodate associated restricted access. In Lizzi's opinion, the most important example of this type is the earlier underpinning of the Ponte Vecchio in Florence from 1962 to 1966. This bridge had survived the rigors of war only to be found succumbing to traffic vibrations. Micropiles were installed through the existing footings and the bridge successfully withstood a major flood in 1966, which proved to be very destructive to other structures in the valley.

The redevelopment and refurbishment of a commercial building on Boylston Street in Boston, Massachusetts, involved the addition of two stories and a mechanical penthouse (Bruce, 1988a). To accommodate the resultant increase in load, Type 1B micropiles were constructed through enlarged pile caps around the original timber piles (figure 22). Piling had to be executed from within the basement of the structure, in headroom as low as 2.4 m.

Other notable and recent examples in the United States include an extension to the Presbyterian University Hospital, Pittsburgh, Pennsylvania, and the rehabilitation of the Old Post Office Building in Washington, D.C. (Bruce, 1992) (volume IV).

Foundations for New Structures

The numerous advantages that micropiles can offer when being installed under existing structures may not always be needed when selecting a piling system for new construction. However, when difficult ground conditions are foreseen; limitations are placed on noise, vibration, and spoil handling; or access is severely restricted, then micropiles have again proved to be a cost-effective option. Construction duration and technical performance are less common, although occasionally controlling factors, and the following circumstances — singly or together — can be regarded as the most compelling.

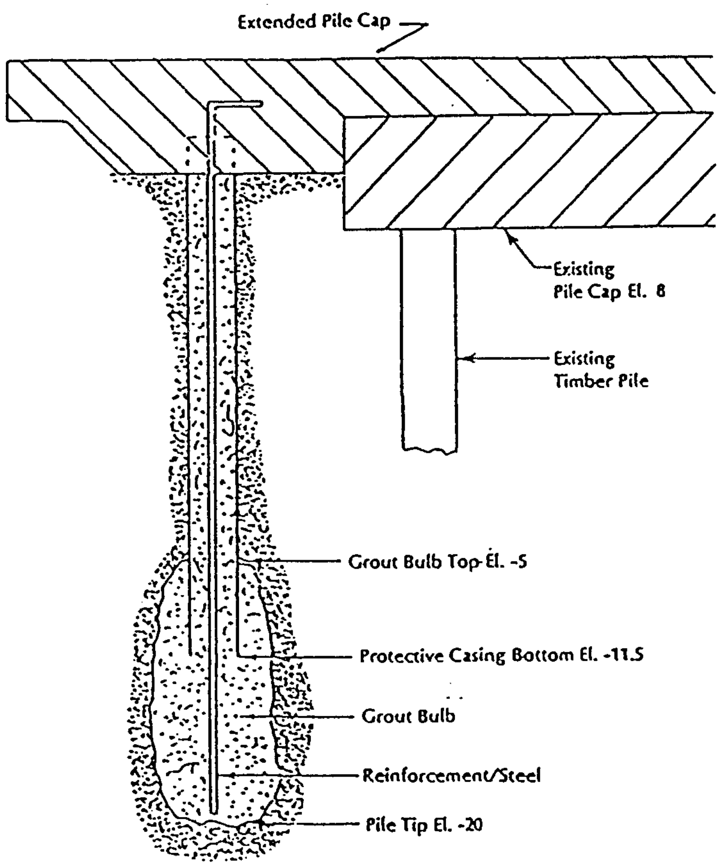


Figure 22. General arrangement of micropiles, Boylston Street, Boston, Massachusetts (Bruce, 1988a).

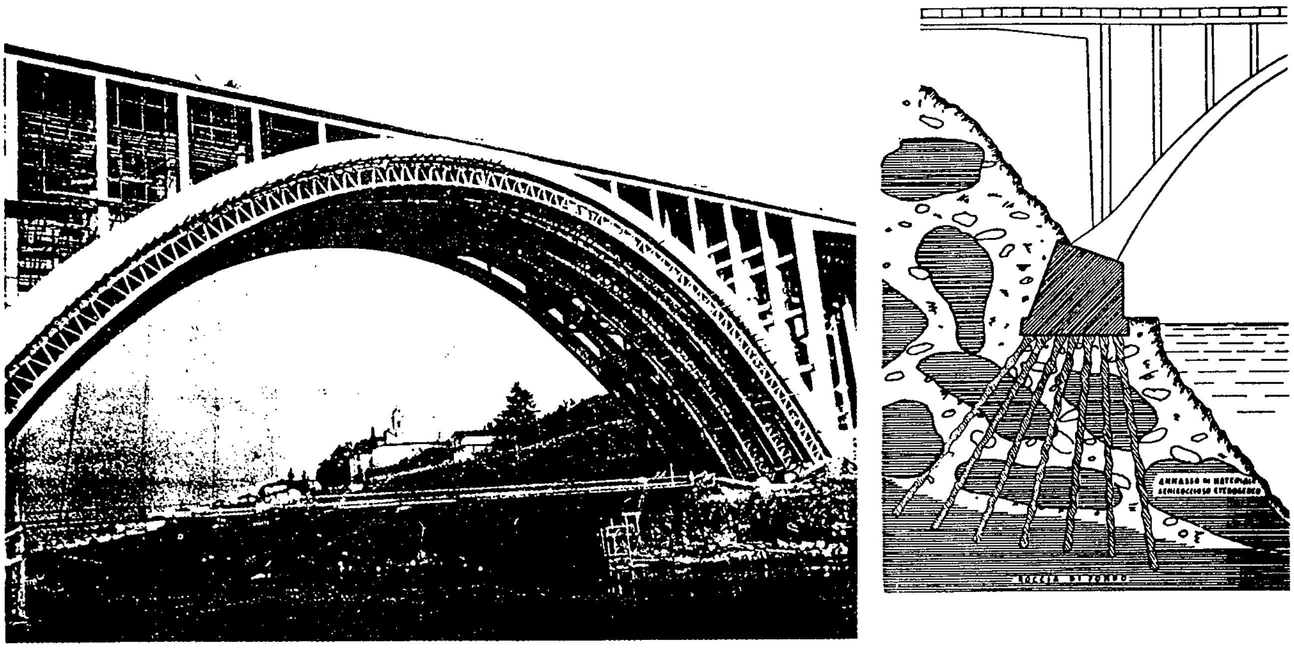


Figure 23. Widening of the thruway between Milan and Bergamo, Italy, over the Adda River (Lizzi, 1971).

(1) <u>Restrictive Site or Access Conditions</u>

The relatively small drilling and grouting equipment utilized for micropile construction enable micropiles to be constructed in certain restrictive situations where other pile techniques cannot be implemented. For instance, micropiles can provide deep foundations for new structures that are close to existing ones, and they are often used as foundations for new bridge abutments or piers (for example, at the Brooklyn-Queens Expressway), when power lines or other overhead obstructions limit installation headroom, or when steep slopes prevent the operation of conventional pile driving and drilled shaft equipment (Pearlman and Wolosick, 1992). Figure 23 illustrates the latter situation, encountered during widening of a thruway spanning the Adda River in Italy. Micropiles were installed from the steep abutment slope. In addition, the specialized drilling equipment could reliably penetrate the bouldery overburden.

(2) <u>Difficult Geological Conditions</u>

Micropiles have been used as new foundations in cases where geological conditions are particularly difficult, variable, or unpredictable, and conventional driven or bored piles would be exceptionally difficult or costly to install. Examples of such conditions include soils with random boulders; fills with buried services or old building materials; and variable geological conditions, such as where hard layers alternate with weak ones. Karstic limestone terrains are often addressed in this way, and examples abound throughout the world of using micropiles as opposed to large-diameter bored piles.

A particularly interesting example was described by Fenoux (1976). Micropiles were used for the foundations of the "Le Fermentor" building in Monte Carlo, Monaco, which was built on highly variable ground (figure 24) with a great many obstructions, including old foundations. No conventional piling system could have been economically mobilized or used in the timeframe available.

New bridge construction is a common application since abutments are often located in areas of inadequate bearing materials, such as alluvial deposits, which dictate the need for deep foundation alternatives. Bruce (1988, 1989) describes the construction of a new highway crossing over the Delaware River in New Jersey, for which driven piles were originally specified to be founded on solid rock. During excavation for one of the piers, however, only random rock thicknesses (karstic limestone) were found, and the actual bedrock surface was highly irregular. A Type 1A micropile alternative was chosen over large-diameter drilled shafts mostly due to economics, but also because of the technical advantage of the micropiles transferring load by skin friction as opposed to end bearing, eliminating the risk of failure by piles punching through into soft clay interbeds in the karstic limestone. The decision to use micropiles was further supported when comparing actual micropile depths to anticipated drilled shaft depths (figure 25). The large-diameter elements would have been too short to develop adequate end bearing, since poor or voided rock was found consistently below the predesigned caisson tip levels.

(3) Environmentally Sensitive Areas

Bruce (1988, 1989) describes the use of Type 1B micropiles as foundations for the support poles of an aviary-type structure, constructed in an environmentally sensitive garden setting where preservation of the existing flora and fauna was paramount. Conventional piling and spread footings were unacceptable because of the nature of the site and their potential for disturbing the existing flora. Micropiles could accommodate construction in this sensitive environmental setting and were also economically advantageous since their installation utilized the same equipment being used for adjacent ground anchors. The swampy underfoot conditions were also not conducive to the use of large equipment, and the required loads could not be met by the capacities of contemporary mechanical or helical anchors.

Micropiles are also particularly useful in supporting tower structures, such as electrical transmission towers, since they can withstand both compressive and tensile loadings with equal facility. Weltman (1981) confirms that micropiles are used as tension elements below buoyant structures, although there are usually several other options in this instance.

Bruce and Yeung (1983) describe the use of Type 1A micropiles to support a new extension of the Hong Kong Country Club. The nature of the site and the presence of an adjacent wildlife park ruled out the use of large equipment and severely restricted noise and waste emission.

Micropiles are also becoming a preferred option to underpin structures in old urban and industrial areas that are underlain by contaminated soil. The small-diameter drilling and easily controllable flush effluent clearly reduces the degree of potential surface contamination. Recent examples of this advantage include the work done for a petrochemical facility in Mobile, Alabama (Bruce et al., 1992), and the piles installed under the Brooklyn-Queens Expressway in New York City (Bruce and Gemme, 1992), and applications are being considered for the Interstate 880 Replacement Project in Oakland, California.

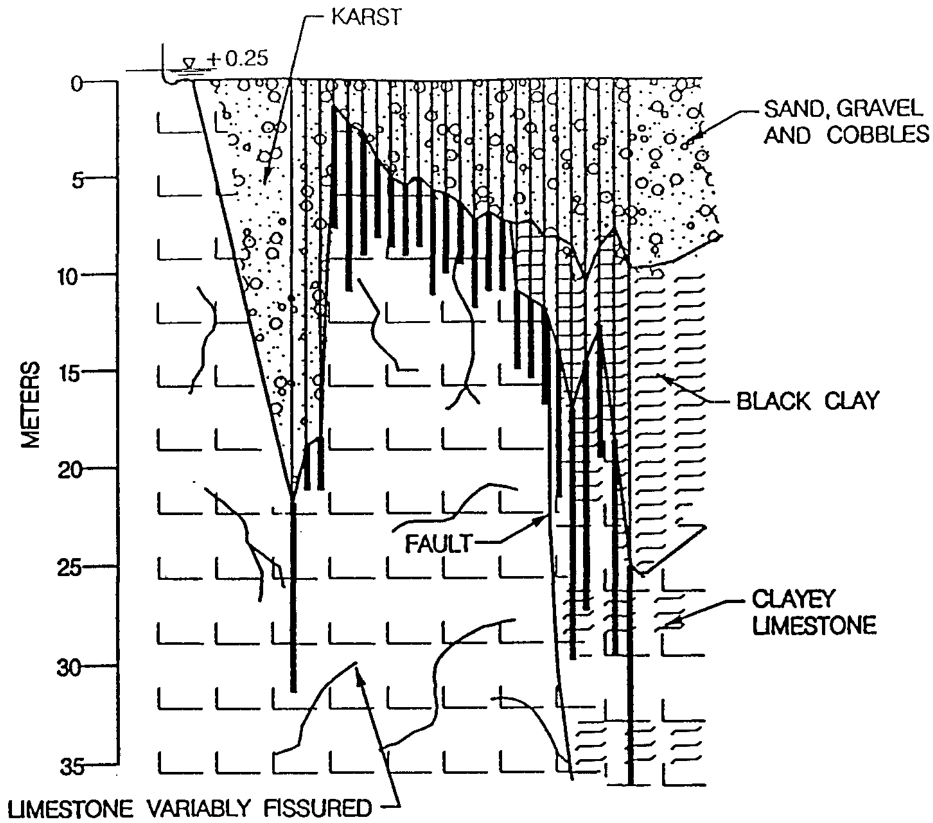


Figure 24. Cross section of a new foundation on "needle piles" adapted to locally variable conditions, "Le Fermentor" Building, Monte Carlo, Monaco (Fenoux, 1976).

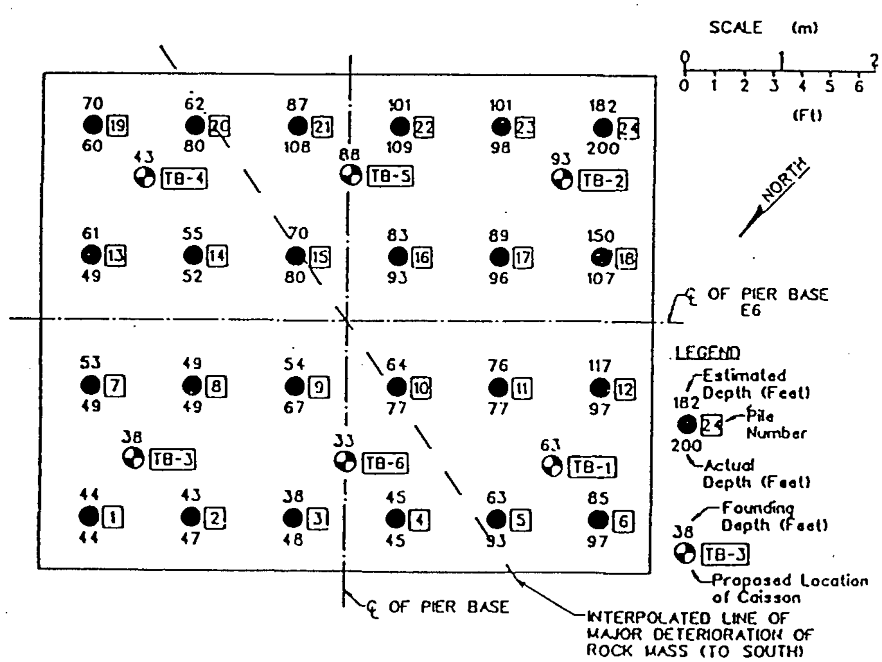


Figure 25. Actual micropile lengths and foreseen drilled shaft depths, Pier 36, I-78 Bridge, Warren County, New Jersey (Bruce, 1988, 1989).

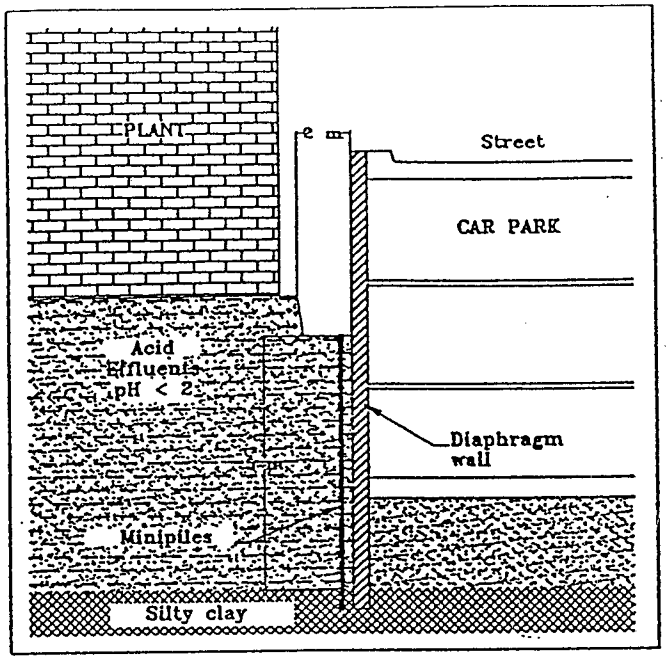


Figure 26. Protection of a diaphragm wall with secant minipile screen utilizing anti-acid grout (Bachy, 1992).

A rather unusual case history was reported by the French company, Bachy (1992). A concrete diaphragm wall had been installed to form an underground carpark in Barcelona, Spain (figure 26). Studies of the concrete revealed it was chemically and physically deteriorating due to extremely aggressive groundwater (chlorides, sulphates, and pH values as low as 1.7) originating from an adjacent metallurgical plant. An in situ screen was therefore required to isolate the concrete wall from the souce of pollution, and, primarily as a result of very restricted surface access, a Type 1A micropile wall was designed. Special anti-acid grouts were designed by the contractor. The 200-mm-diameter piles were overlapped (secant piles) in the deeper, high-acidity regions, and were made contiguous in areas of medium acidity. Investigations carried out after completion of the screen detected no trace of acid on the diaphragm wall, and so confirmed the effectiveness of the micropile screen wall.

Seismic Retrofitting

Micropiles are increasingly being used in the seismic retrofit of highway structures such as bridge piers. Often, footing capacities must be upgraded to accommodate higher design overturning moments, and micropiles provide a practical alternative to traditional techniques (compression piling and tiedown anchors) since they derive their load-holding capacity through frictional bond. They therefore perform comparably in tension and in compression, optimizing the required number of elements. In addition, micropiles can surmount many of the typical constraints associated with such upgrades, most of which are undertaken in urban areas:

- Noise- and vibration-level limitations.
- Installation in low-headroom conditions (such as beneath bridge decks) affecting drilling and reinforcement placing activities.

- Difficult drilling conditions due to ground obstructions, high water tables, utilities, or strata of alternating hardness.
- Reduction of excessive pile lengths to meet higher tensile capacity requirements.
- Limited right-of-way access.
- Inability to extend existing footings.
- Contaminated soils.

As an example, Pearlman et al. (1993) describe the use of micropiles for the strengthening of existing footings during the seismic retrofit of the Caltrans North Connector Overcrossing (I-110). Originally, the design prescribed the use of 610-mm-diameter Cast-In-Drilled-Hole (CIDH) concrete piles. However, concrete obstructions and water-bearing sand layers encountered during drilling, together with low overhead clearance, prevented the use of this pile type. An alternative program utilizing Type 1B micropiles was successfully implemented to complete the retrofit at considerable savings to the owner (figure 27). This application of micropiles is extremely important, an observation underlined by the extensive testing of such piles in the major FHWA/Caltrans test-pile program conducted in 1992 and 1993 in San Francisco, California, and their subsequent acceptance as an approved Caltrans foundation option (Caltrans, 1993) on several sole-source contracts.

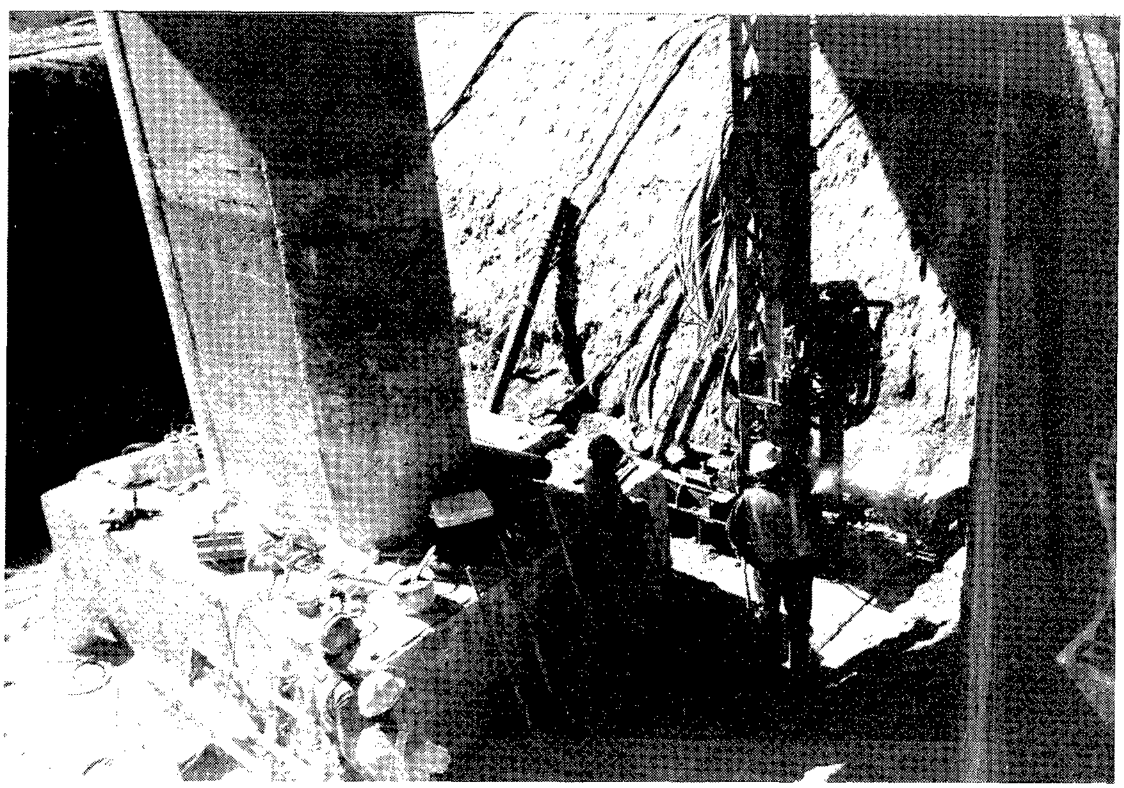


Figure 27. Drilling of micropiles using a detached drill mast, for seismic retrofit in footing excavation, I-110, Los Angeles, California.

It should be noted that all piles installed so far in U.S. seismic retrofit programs have been CASE 1, and nearly all were vertical. Although it is suspected that great potential exists for a CASE 2 approach to seismic retrofit, this cannot yet be verified, given the current state of knowledge. Internationally, no systematic data exist on the performance of micropile-supported structures during seismic events. However, Lizzi contributed a "statistical datum" in the form of his observation that each of the multitude of delicate structures founded on micropiles have "remained practically unaffected by the several severe earthquakes" recently recorded in Italy (Lizzi, 1994).

In Situ Reinforcement

Lizzi (1982a) originally advanced the "knot effect" concept, in which the micropiles, appropriately spaced in a reticulated three-dimensional network, encompass and reinforce the soil, while at the same time they are supported by the soil. For slope stabilization applications, he advocated that these reticulated micropile structures create a reinforced soil/pile "gravity retaining wall," where the soil supplies the essential resisting force (the gravity), and the piles supply additional resistance to the tensile and shear forces acting on this wall. For such structures, the individual piles are engaged as friction piles securing the pile/soil composite mass in the upper section (CASE 2), and as structural elements subjected to shear and bending in the lower part (CASE 1). The overall aim of this structure is to provide a stable block of reinforced soil to act as a coherent retaining structure, holding back the soil behind it, while providing resistance to shear across the failure plane.

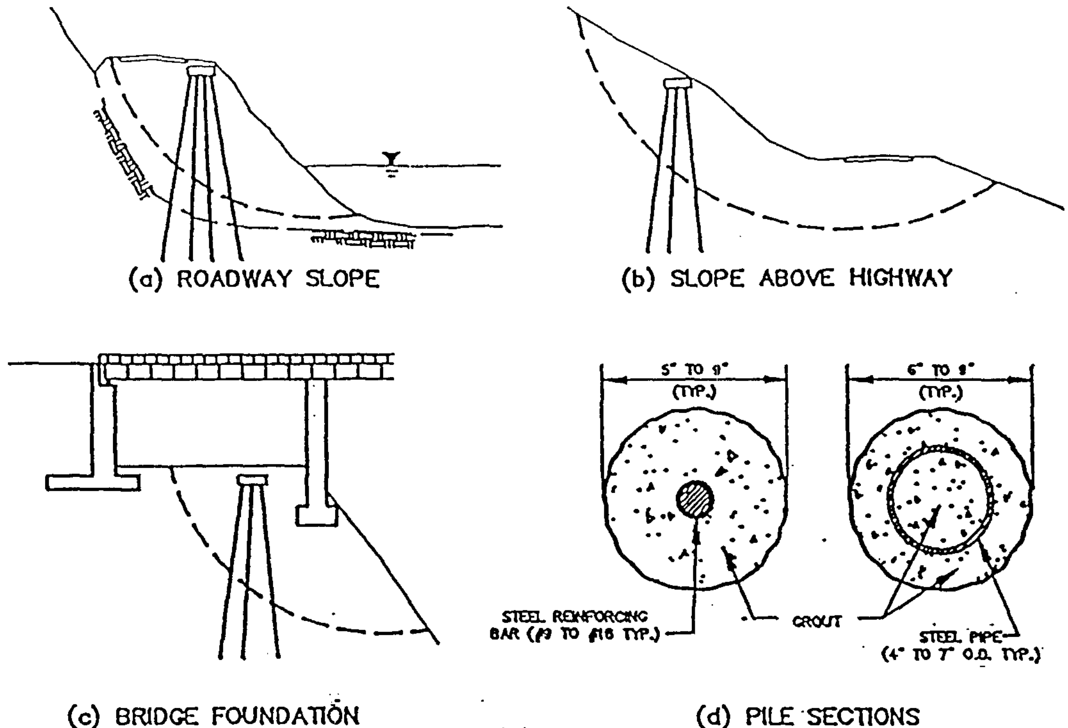
In contrast, Pearlman et al. (1992) present evidence that similar structures, but comprising groups of inclined micropiles, do not necessarily behave as gravity walls, but rather the micropiles serve only to connect the moving zone (above the slip surface) to the stable zone (below the slip surface). Through their bending and shear capacities alone, they therefore increase sliding resistance along the slip plane. Palmerton (1984) also presents data indicating similar behavior. These piles would, therefore, be purely CASE 1 in design approach.

For the other three major applications of in situ reinforcement shown in figure 16, the design concept appears to be CASE 2, although certain older case histories indicate that some soil-strengthening and retention schemes have featured at least a minor component of direct axial loading (CASE 1).

In all these examples, it must be restated that the in situ reinforcement features micropiles installed in steeply dipping networks and groups. This is in contrast to the in situ reinforcement effect provided by subhorizontal groups of soil nails, as illustrated in figure 3.

Embankment, Slope, and Landslide Stabilization

Typical configurations and applications of inclined micropile walls for the stabilization of slopes, embankments, and landslides are illustrated in figure 28. Lizzi (1978) confirms certain differences in design concept for different types of soil masses, as shown in figure 29. For the case of bouldery, stiff, or dense formations, the shear resistance of the piles across the supposed sliding surface is important (CASE 1). Conversely, in the loose soil condition where



1 in = 25.4 mm

Figure 28. Typical configurations and applications for inclined micropile (Type A) walls (Pearlman et al., 1992).

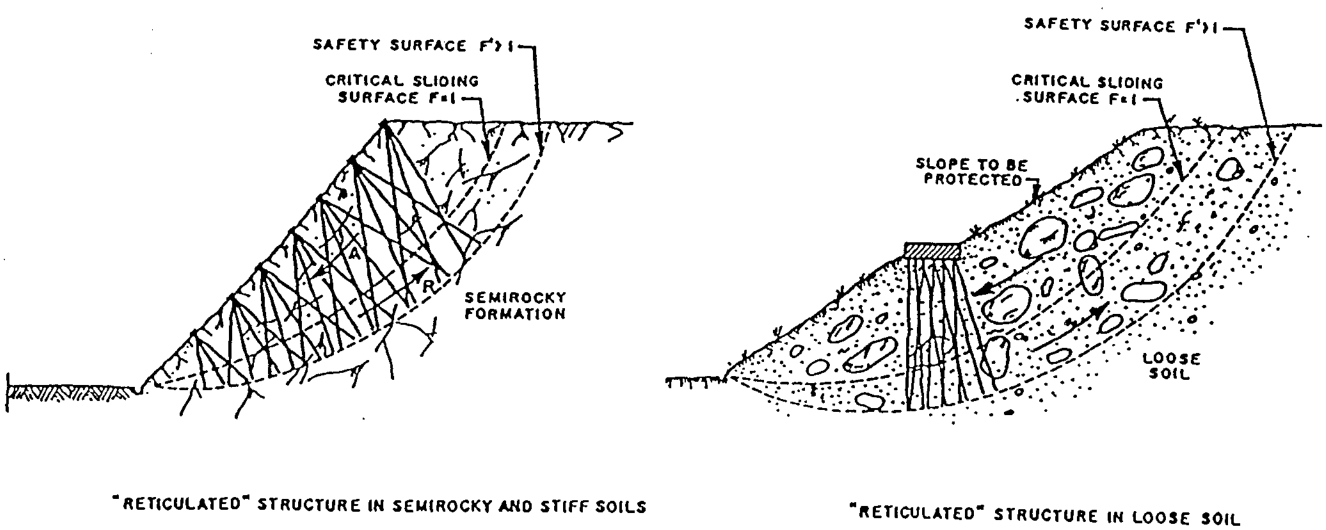


Figure 29. Typical patterns for reticulated micropiles in two different slope stability applications (Lizzi, 1978).

the idea is to construct a gravity wall, the essential element is the soil, and so the individual capacity of the piles is not critical (CASE 2). Similar types of installations for slope or embankment stabilization are termed RPM (Root Pile Method) in Japan. Reticulated micropiles have also been used to reinforce the downstream portion of an embankment dam (Lizzi, 1982a).

Regardless of their design concepts, micropile groups and networks are employed where conventional retaining systems cannot be constructed due to geological, site/access, or cost constraints. Particular advantages include the ability to provide stabilization without the need to excavate and the relatively low impact of the installation procedure on the existing stability of the slope.

Soil Strengthening and Protection

Reticulated micropile networks have been used on occasion to retain or support soils during nearby or adjacent tunneling or deep excavation activities.

Figure 30 illustrates the application as a retaining wall, protecting the foundation of an existing building during the excavation of a cut-and-cover section. Figure 31 shows a similar, more recent application -- support of excavation in a road-widening project. In both cases, surface access was severely restricted, and major restraints had been placed on noise and waste emission levels. Each example was essentially a CASE 2 design, but the latter structure also had a CASE 1 function as it was, over part of its length, subjected directly to compressive structural loads by a new bridge deck bearing on its capping beam.

A slightly different application is shown in figure 32. The foundation soils (Zone B) of an existing building were protected against any loss of ground (in Zone A) arising from the adjacent underground construction of new subway tunnels. Construction was carried out from within a small service tunnel and traffic was not disrupted. This example is purely a CASE 2 structure, in contrast to that in figure 33 where the micropile network provided both a zone of reinforced soil (to limit soil movement) and direct structural underpinning (CASE 1) for the existing building.

In the example shown in figure 34, a newly constructed tunnel was being structurally damaged by irregular point loads exerted by a very variable, often loose, flyche-type sediment. Conventional prestressed rock bolts were not feasible due to the absence of an appropriate bonding horizon. Instead, a network of 10- to 12-m-long CASE 2 micropiles were installed, creating an in situ arch of reinforced soil that accepted the loads and distributed them evenly to the lining.

An extremely significant application of essentially vertical piles for ground reinforcement was provided by Blondeau et al. (1987) and is described in detail in volume IV. The soil under a heavy structure involved in the construction of a new nuclear power plant in South Korea was found to be a weathered and faulted zone with static and dynamic properties far inferior to the adjacent sound rock masses (figure 35). These CASE 2 piles were installed to improve the mass properties of the faulted zone and to render it much closer in both static and dynamic performance, as a foundation mass, to the intact rock.

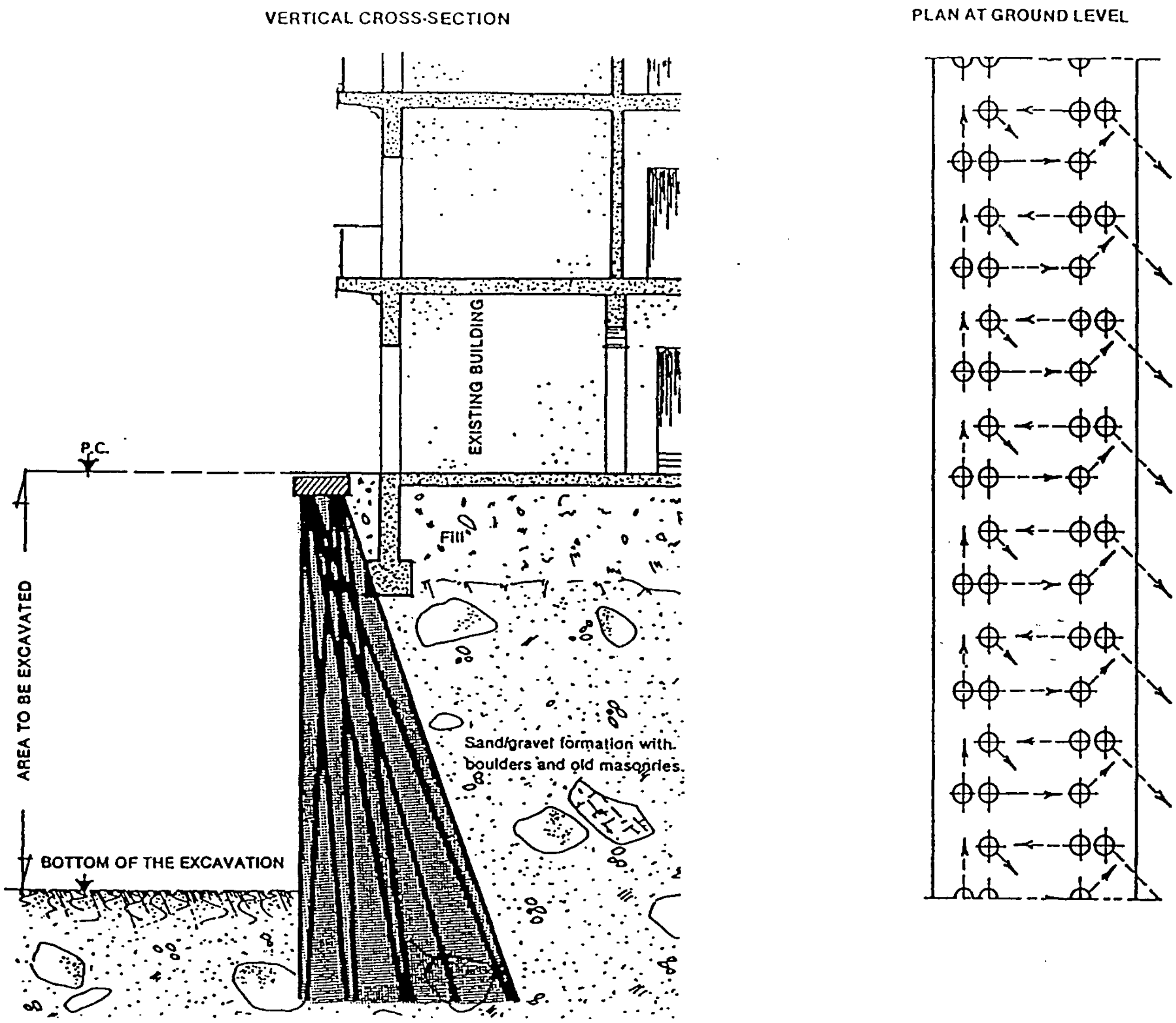
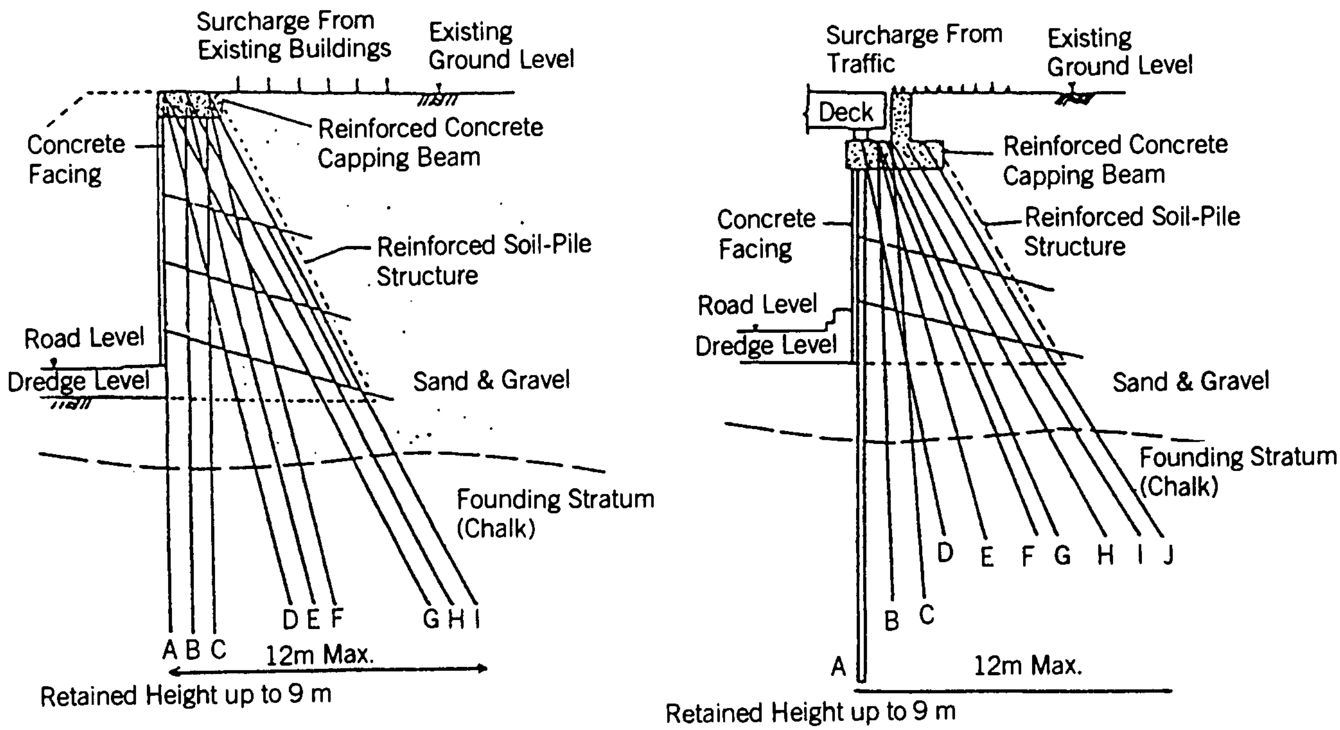


Figure 30. Typical micropile configuration to protect a structure from the effects of an adjacent subway excavation in Milan, Italy (Lizzi, 1982).



Typical Range of Pile Loadings (kN)

	Max	Min
Row A	165	90
B	153	65
C	142	40
D	134	23
E	118	-10
F	103	-43
G	87	-77
H	72	-110
I	56	-144

	Max	Min
Row A	366	311
B	125	57
C	114	33
D	105	17
E	99	0
F	84	-33
G	77	-49
H	62	-81
I	51	-106
J	40	-130

Figure 31. Reticulated micropile wall used as support of excavation, Dartford, United Kingdom (Attwood, 1987).

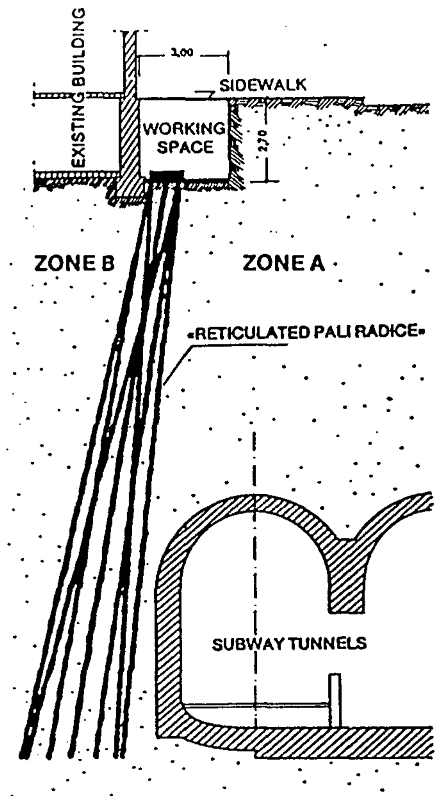


Figure 32. Typical micropile scheme for protection of a structure during tunneling operations, Paris, France (Lizzi, 1982).

Settlement Reduction

A few examples of CASE 2 projects using vertical small-diameter driven piles have been recorded. Figure 36 shows an American experiment by Korfiatis using wooden piles to reduce the settlement potential of an embankment, while in England a system called Bridge Approach Support Piling (BASP) was proposed to provide improved foundation conditions for access ramps. Weltman (1981) and FOREVER (1992) have suggested that such applications could be executed with micropiles, although cost-effectiveness would be a major issue. No full-scale example has yet been reported. However, Plumelle (1984, 1985) has reported on several tests conducted at an experimental site near Paris, and this initiative may well result in future commercial applications (figure 37).

Structural Stability

Lizzi and Carnevale (1981) demonstrated how reticulated root piles were used to improve the stability of a tall and slender tower in Mosul, Iraq (figure 38). This CASE 2 network defines a bulb of reinforced soil, attached to the structure, whose weight significantly lowers the center of gravity of the whole structure-bulb system. In this approach, the piles do not have to extend to great depths, or even reach a particularly competent bearing horizon. It is merely sufficient that the block of reinforced soil has adequate weight. For the example shown in figure 38, the micropiles were part of a complete and integrated soil and structure retrofit.

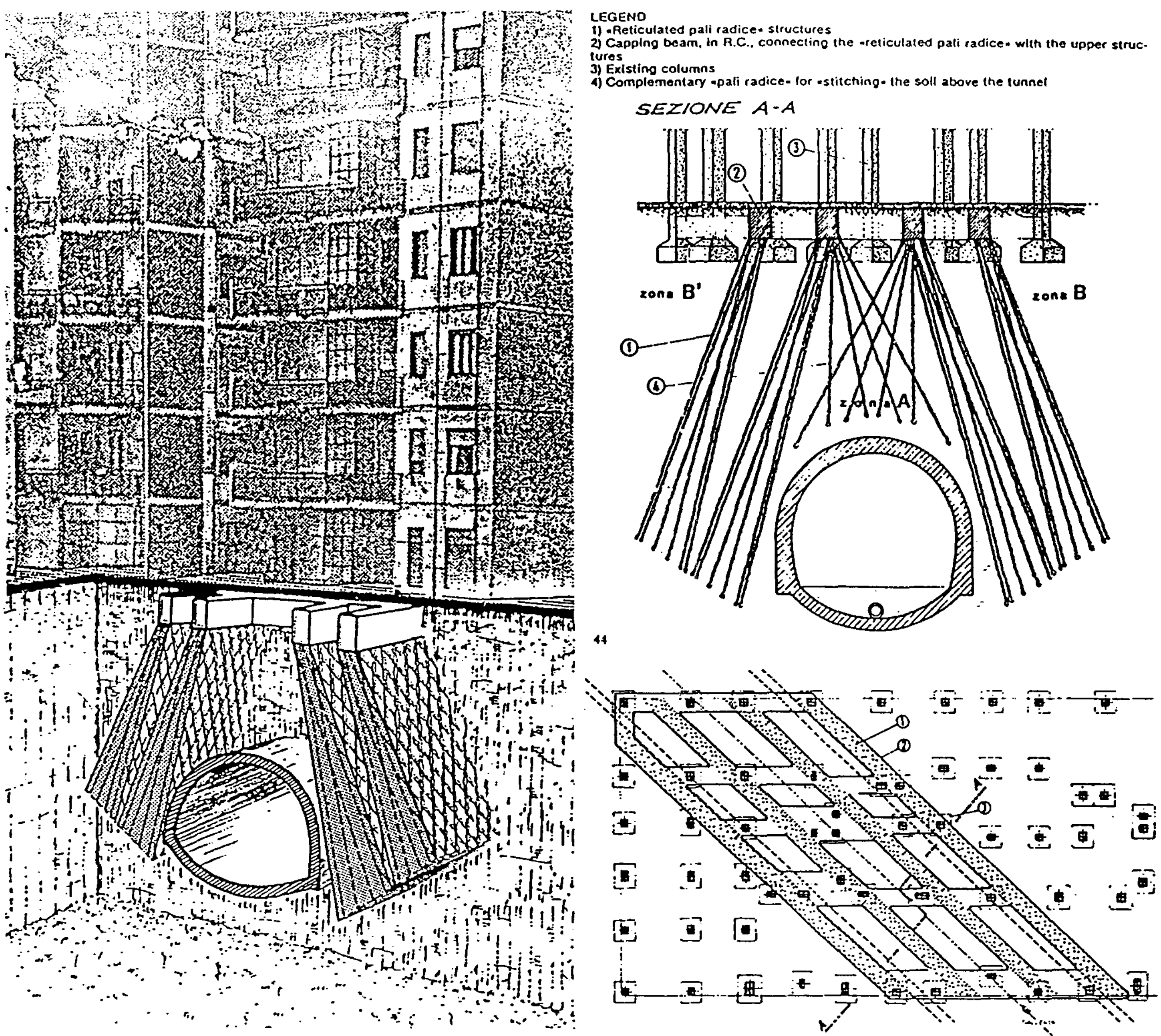


Figure 33. Reticulated micropile underpinning of a building during construction of a railway tunnel in Salerno, Italy (Lizzi, 1982).

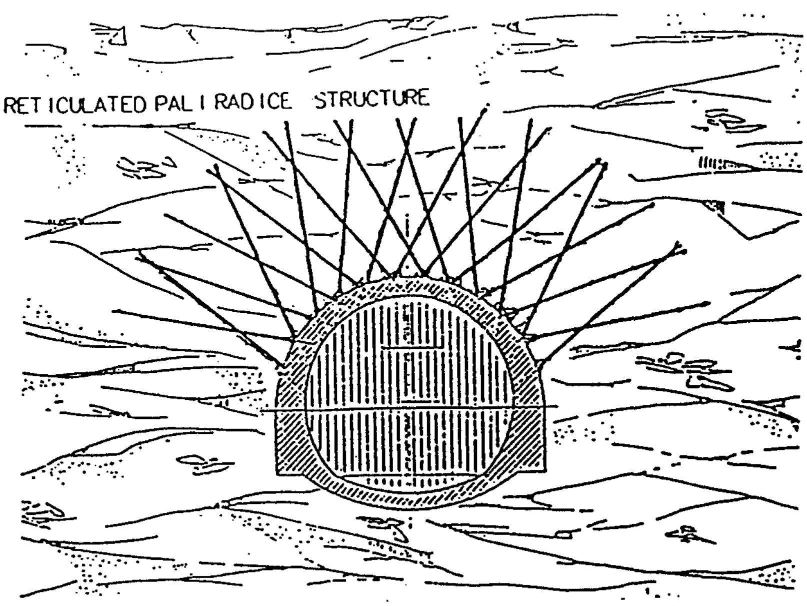


Figure 34. Scheme of reinforcement of an existing tunnel (Lizzi, 1994). [Note: Reinforcements are inclined in three dimensions.]

This whole concept has great potential in the seismic retrofit of structures in that the dynamic response of a structure/network system will be substantially different from that of the structure alone. The lack of experimental demonstration and appropriate design methodologies has to date prevented this solution from being adopted more widely.

FACTORS INFLUENCING THE CHOICE OF MICROPILES

Physical Constraints

Micropiles can be installed with conventional, small-scale drilling and grouting equipment (volume III). The maneuverability and compactness of such equipment can permit piles to be installed in confined, awkward, or otherwise physically constraining work spaces. For this reason, micropiles have proved to be an ideal choice when project requirements dictate installation in areas with low overhead clearance, or in close proximity to existing walls, columns, footings, or other structures. Drilling and placing of reinforcement can proceed in as little as 2 m of headroom and as close as 200 mm to existing walls. These features also can allow installation from within existing, fully functional facilities with minimal disruptions to normal operations. The portable or modular nature of the equipment also allows installation on steep slopes, in limited right-of-way areas, and in other locations that do not permit conventional piling rigs to be mobilized. The small diameters further facilitate pile installation in areas known to have buried services or other utilities.

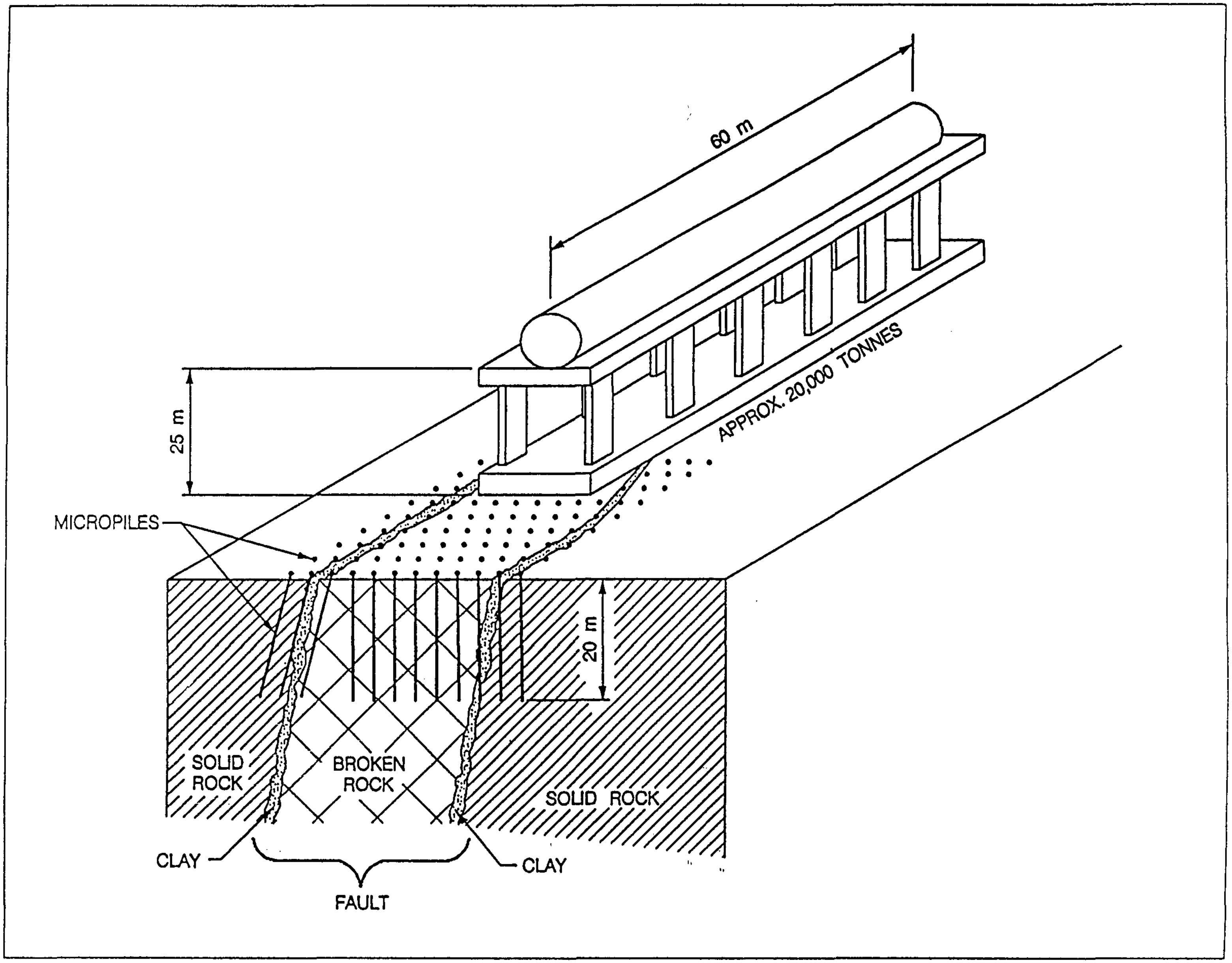


Figure 35. Micropile foundations at Uljin Nuclear Power Plant, South Korea (Blondeau et al., 1987).

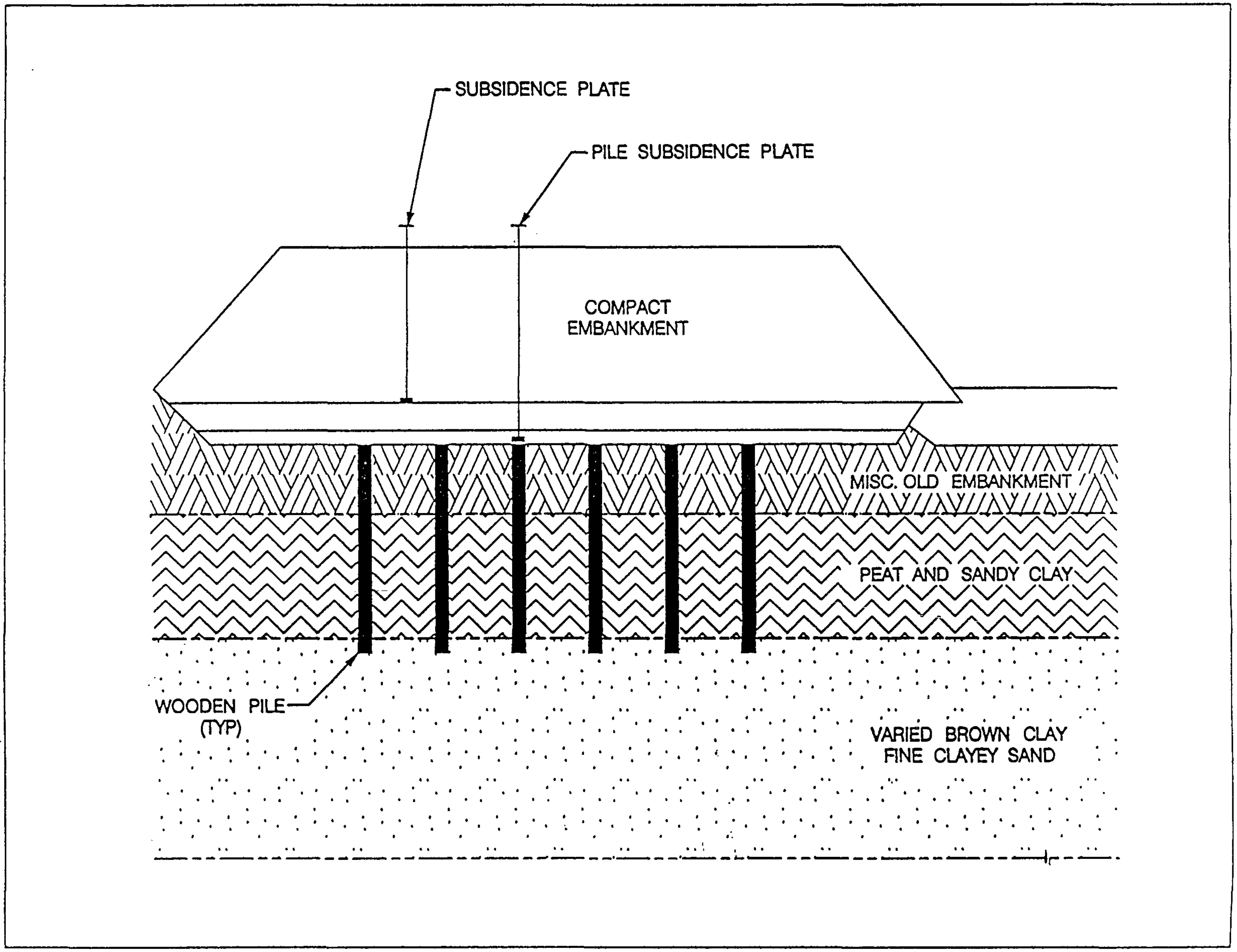


Figure 36. Experiment with an embankment on soil reinforced with small-section driven wooden piles to reduce settlement (Korfiatis, 1984).

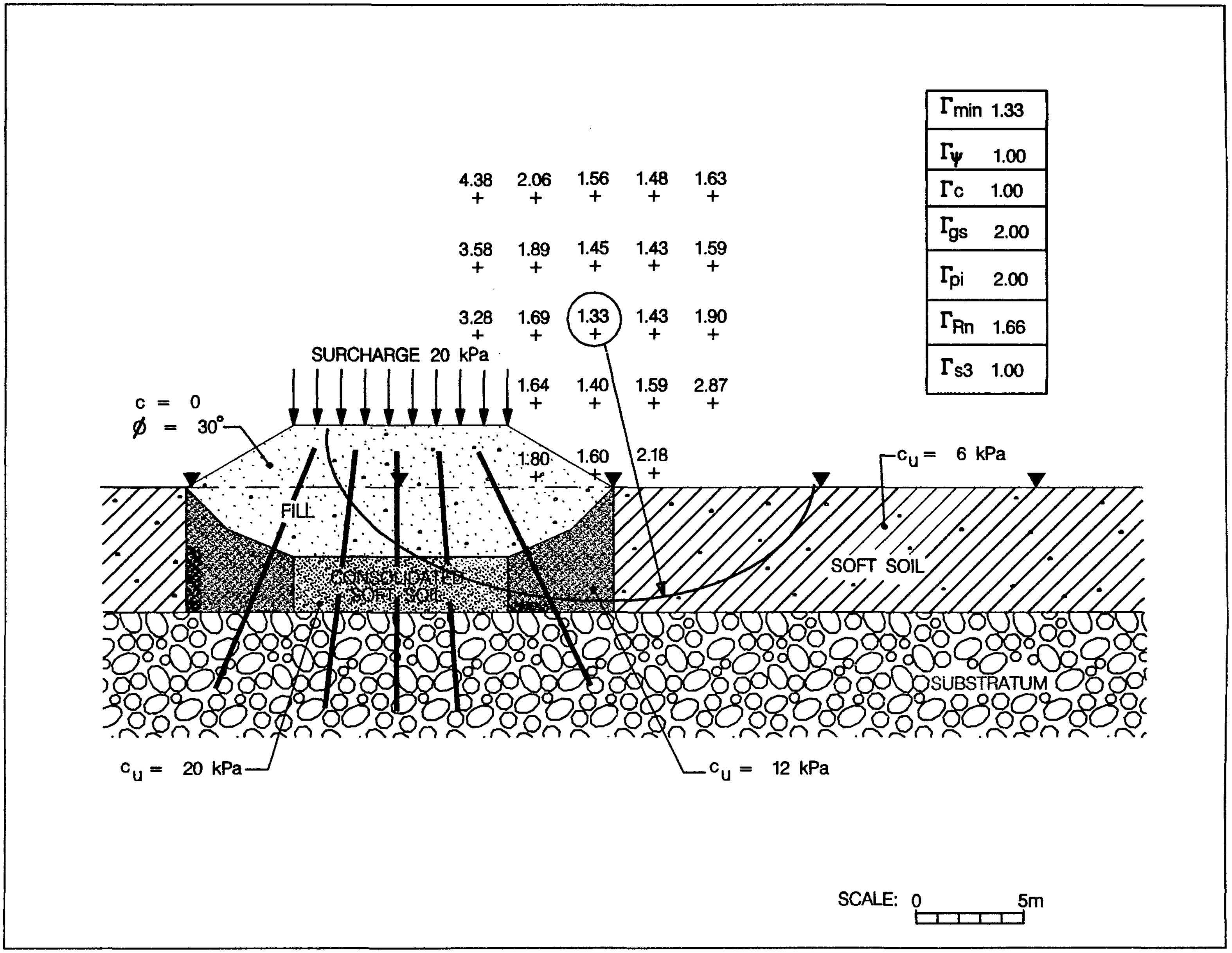
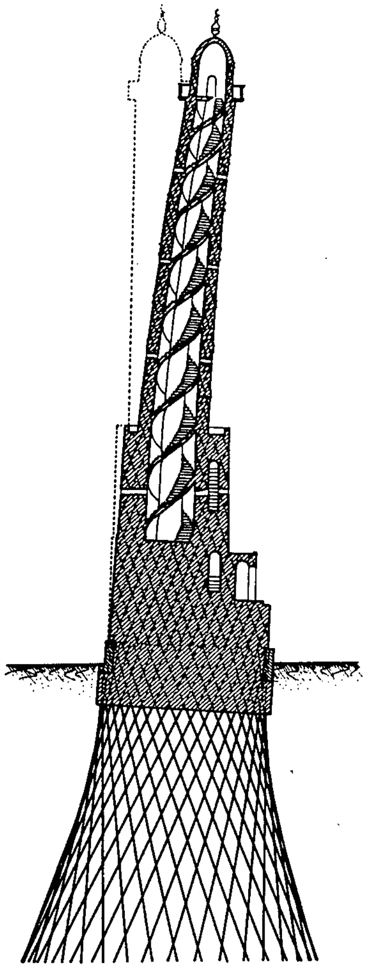
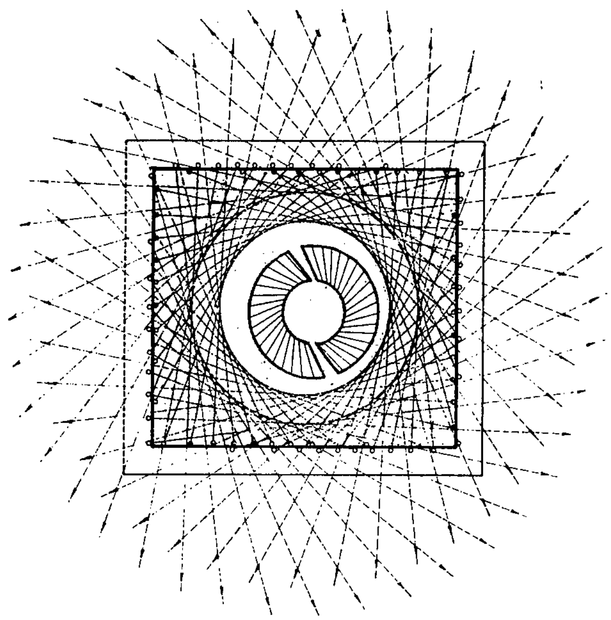
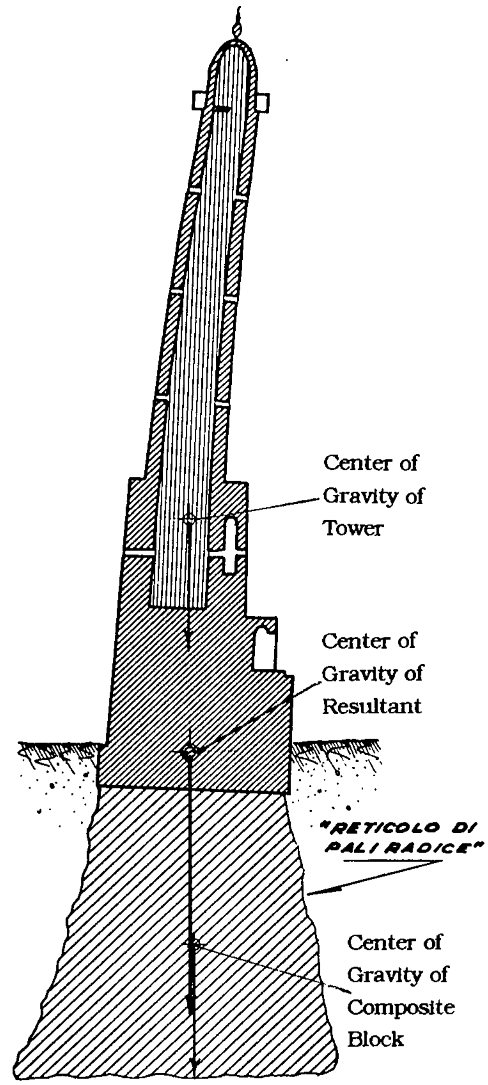


Figure 37. Experimental embankment with micropiles, near Paris, France (Plumelle, 1985).



(a) Arrangement of root piles.





(b) Gravity block concept.

Figure 38. Stabilization scheme for the leaning Al Hadba Minaret in Mosul, Iraq (Lizzi, 1982).

Environmental Constraints

The drilling and grouting techniques used in micropile construction are selected to impart minimal disturbance on the soil, structure, and surrounding environment. Thus, micropiles have been selected when installation is required near structures sensitive to vibration-induced damage (e.g., historic buildings); in areas with soils susceptible to vibration-induced settlement; and in populated areas where vibrations, noise, dust, or disposal of cuttings or flush are problematical. The use of lightweight drilling equipment, coupled with the nature of the drilling process, has also permitted micropile construction on sensitive slopes of marginal stability. Furthermore, underpinning with micropiles does not require the prior dewatering of the site. This has obvious technical and cost advantages, especially where the groundwater may be contaminated and safe spoil disposal is extremely onerous.

Difficult Ground Conditions

Micropiles are an attractive option when ground conditions are difficult. The range of available drilling techniques permits installation through virtually any type of fill, soil, rock, or obstruction, as well as through existing structures. Depth and orientation also are virtually unrestricted within the typical range of micropile applications. Installation has been accomplished in a wide range of challenging ground conditions, including:

- Karstic limestone (with voids or solution cavities).
- Mined rock (with voids or rubble-filled voids).
- Bouldery ground or glacial tills.
- Soils with a high groundwater surface.
- Variable/random urban fills.
- In the presence of existing foundations or other obstructions.

The drilling and grouting procedures used in construction enable micropiles to suitably develop side resistance in almost any soil type, including:

- Stiff or hard clays or silts of low plasticity.
- Sands and gravels.
- All rock formations.
- Combination materials such as glacial tills or fills.

It should also be noted that micropile drilling techniques can accommodate the even wider range of materials, both natural and artificial, which can overly target bearing strata (volume III).

Load/Movement Criteria

Micropiles carry load predominantly through side resistance and, when suitably reinforced, can sustain relatively high loads with only small movement. Therefore, they are attractive when movement criteria are especially strict. It should be noted, however, that there is a major conceptual difference between underpinning historic, delicate, masonry monuments where only minimal additional movements can be tolerated and the support of contemporary reinforced-concrete or steel-framed buildings that may be designed to accommodate relatively large differential movements. Appropriate designs can be made to address each concept of movement control. If a structure is extremely sensitive, preloading of CASE 1 piles can be undertaken in certain circumstances to fully eliminate pile compression before the piles are attached to the structure. However, it must also be recalled that support with minimal movement was one of Lizzi's goals when introducing the original root pile, which was fully bonded to the surrounding soil over its entire length and which therefore had excellent load-resisting properties.

Figure 39 shows the results of early tests in Venice, Italy. The fully bonded root pile had a movement at 510 kN of 1.3 mm, compared to a movement at the same load of 7.2 mm for a "steel" micropile. The latter was bonded to the soil only in a deep bearing stratum. Analysis of this behavior is provided in volume II.

Tests indicate that geotechnical capacity is similar in both compression and tension (since micropiles are essentially stiff elements). Tensile capacity in competent soils and rocks is generally limited to the structural capacity of the reinforcement (Pearlman and Wolosick, 1992), assuming that the pile/structure connection can maintain the limit state demands of the pile. When subjected to lateral forces, micropiles derive resistance from the horizontal response of adjacent soils, and can thus sustain significant lateral deflection before yield levels are exceeded.

For example, lateral load tests on vertical micropiles reinforced with a 178-mm-diameter casing yielded (in variable urban fills) a 19-mm deflection at a lateral load of 85 kN, and in stiff alluvial soils, a deflection of 7.6 mm at 107 kN, both for a free-head condition (Pearlman and Wolosick, 1992). Micropiles also appear to respond favorably to repeated cyclic loading (Herbst, 1994), although data are limited. Further discussion is provided in chapter 4. For all these reasons, micropiles are well suited for satisfying the demands of complex loading conditions.

Connection to Structure

As illustrated in the Review of Applications, micropiles can be incorporated and fixed directly into an existing foundation. Since this connection is usually provided by bonding between the pile steel and the structure, it offers both tensile and compressive strength. In addition, this ability can preclude the necessity of creating a new foundation footing, which, in turn, would need to be connected to the existing footing.

Cost

Cost is clearly an important factor when considering the use of micropiles, and this aspect is explored further in chapter 3. When evaluating *cost-effectiveness*, it is important to assess the cost of a micropile option in light of the other factors listed above, namely physical, environmental, and geological constraints; service performance; and connection to structure. For example, for a "green" field site, with soft, clean uniform soils and unrestricted access, micropiles may not be a competitive solution. However, for the delicate underpinning of existing bridge piers in a heavily trafficked old industrial or residential area, then they may be the most *cost-effective* choice, if not the *only* option.

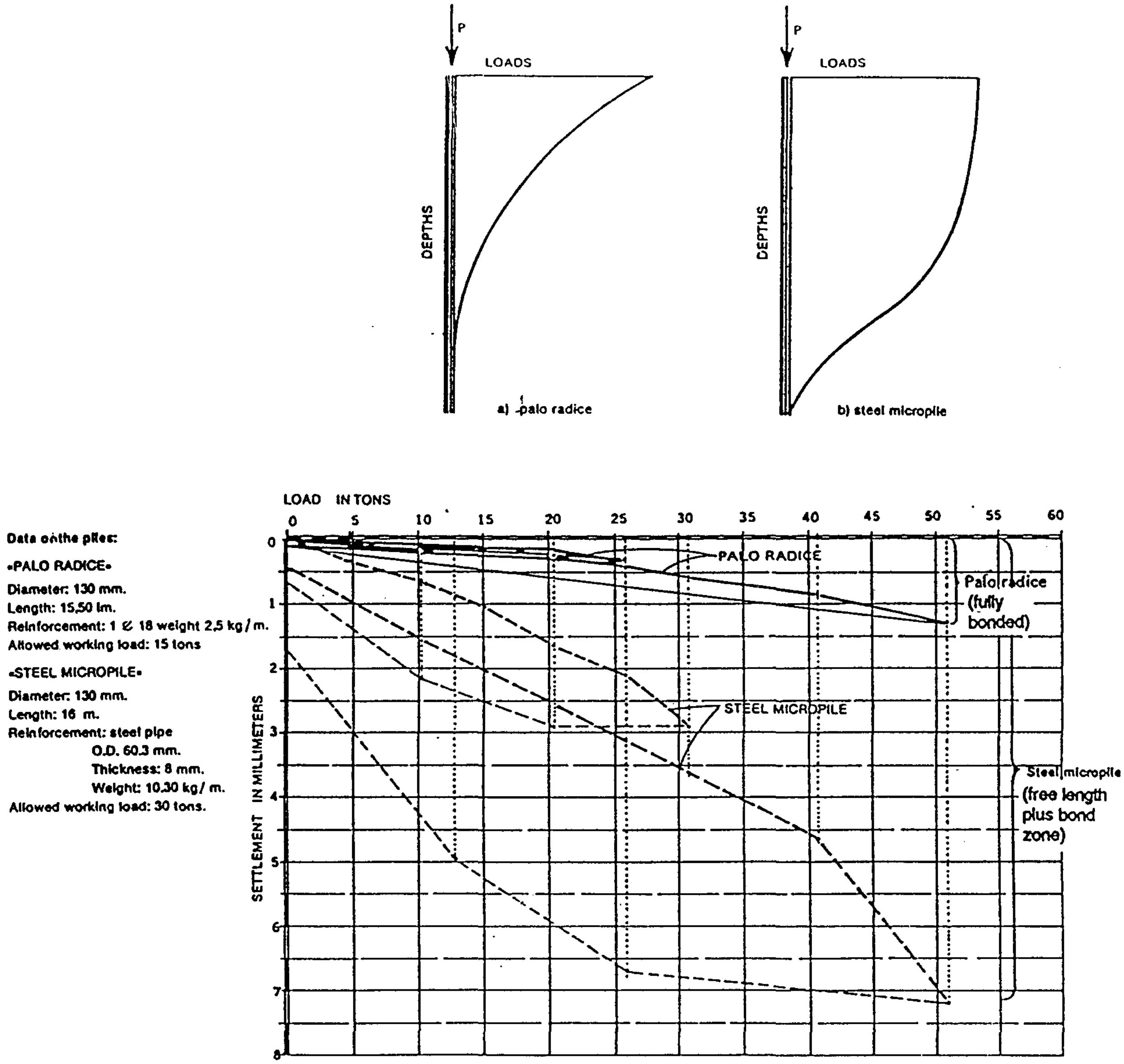


Figure 39. Comparison of performance of fully bonded (palo radice) and partially bonded micropiles (Lizzi, 1982).

CHAPTER 3. FEASIBILITY AND COST

Chapter 2 of this volume, and volumes III and IV demonstrate the wide range of conditions under which micropiles can be used. They are practical in any soil, fill, or rock condition, and can be installed at any angle. They can accommodate potentially restrictive access and environmental problems, and they have wide application both for new construction and in association with existing structures. Micropiles are used for structural underpinning and in situ earth reinforcement (figure 16). As described in volume II, dealing with design, there remain many theoretical and behavioral issues that are incompletely understood, and, in general, capabilities in these fields tend to lag behind those in the more practical aspects of the technology.

However, micropile installation is subjected to a variety of quality assurances and controls, typically and mostly applied by the contractor. This is quite different from installers of driven piles whose practices – at every stage of the process – tend to be specified and monitored by a second party. There is, therefore, not the same risk to successful performance with micropiles as one could otherwise anticipate, given the incompleteness of design theory.

Nevertheless, this situation has restricted the growth of the use of micropiles in certain fields of application, of which seismic retrofit is a good example. However, this is not so much a reflection of their unfeasibility, rather it is an admission of incomplete understanding, inadequate design rules, and weaknesses in predictive ability. By designing very conservatively, if necessary, micropile-based solutions can invariably be "made to work" – technically – in any given piling application. However, costs may escalate to an extent that micropiles, though feasible technically, may not be cost-effective.

The selling price of a micropile is the product of many cost-determining factors, including:

- Physical, access, and environmental conditions on site.
- Subsurface conditions.
- Required capacity.
- Pile length.
- Pile inclination.
- Local labor employment regulations.
- Contractor overhead and margin percentages.
- Country of construction.
- Risk assessment.
- Contractual arrangement.
- Quantity.
- Performance risk.

There is, therefore, no one "typical selling price," but instead a wide and responsive range. As a guideline, however, and assuming:

- No physical, environmental, or access restrictions.
- "Easy" subsurface conditions.
- Average load-holding capacity and average length.

- Vertical inclination.
- Non-Union operation.
- Typical contractor overhead and margin percentages.

then the "base" selling price in the United States is in the range of $130 to $200/per linear meter of pile (1995 costs). This is probably higher than in certain other countries, for example, Britain. However, it must be recognized that every country has its own cost structure that affects not only every type of piling, but each alternative structural support method as well. A sensitivity analysis using this base linear price would indicate the influence on cost by the following factors:

•	Physical and access conditions	Very easy -25 percent	Very difficult +100 percent
•	Geology/soil conditions	Very easy -15 percent	Very difficult +50 percent
•	Pile capacity	Very low -15 percent	Very high +30 percent
•	Inclination	Vertical 0 percent	Shallow incline 15 percent
•	Union	Non-union 0 percent	Very strong 30 percent
•	Overhead and profit margins (reflecting risk)	Very low -10 percent	Very high +10 percent

Thus, applying an unattractive combination of all these factors (for example, a site with very difficult access and geology, and high-capacity inclined piles in a very strong union area) might raise the linear price by much more than 200 percent of the base (i.e., to more than $300/linear meter). In such instances, micropiles, although feasible, may not be *apparently* cost-effective, and so an alternate technology may be investigated. Usually in such an instance, however, exactly the same factors that raise the micropile price will have at least the same cost implication as these other options, which may not even be technically or practically feasible in any case.

In addition, care should always be taken to clearly define the "true cost" of a solution based on micropiles. For example, when comparing linear costs of micropiles to those of other types, the high linear capacity of micropiles should be factored in and a truer assessment will then be made on the basis of price per meter per ton.

Likewise, a micropile option may necessitate less support work (from the General Contractor) or may significantly reduce the overall project schedule, thus providing a cost savings to the owner that is disproportionate to the cost difference of the various piling options.

There are several ways of paying for micropile work, ranging from "lump sum" to "cost plus." However, it is more common to find a basis of payment that includes the following items:

	Item	Unit
•	Mobilization/demobilization.	Per visit to site.
•	Set up drill rig.	Per hole.

• Install pile (of nominated capacity).	Per linear meter of specified capacity.
• Test pile.	Per specified type of test.
• Penetrate obstructions.	Per linear meter or per hour.
• Grout (beyond specified volume per pile).	Per cubic meter.
• Delay time.	Per crew hour.

Regarding construction <u>time</u>, this will be strongly influenced by the same factors determining cost, and, of course, the two are usually closely and inversely related. Under good conditions, a micropile productivity in excess of 100 m per rig per 8-h working shift can be expected. This figure will be reduced by a factor of 2 or 3 in extremely adverse conditions.

When arriving at micropile linear costs, the following approximate cost proportions may be expected (excluding site mobilization, demobilization, and general contractor-type services or facilities):

• Labor	25-40 percent
• Equipment	15-30 percent
• Materials	15-30 percent
• Consumables	10-25 percent
• Subcontracts	0-20 percent

As a final word, it is reiterated that the linear cost of micropiles is usually in excess of that of conventional piling systems, especially those of the driven variety. However, under certain combinations of circumstances, micropiles will be the cost-effective option, and occasionally, it will be the only feasible technical option.

CHAPTER 4. CONTRACTING PRACTICES

Although contracting practices vary between different countries and cultures, there are certain common elements that are apparent. First, most micropile works are carried out under subcontracts to general contractors, and the value can typically range from 5 percent to 40 percent of the value of the overall project. Second, although there is a growing trend to base awards on other criteria, such as technical merit, resources, and experience, price usually remains the most important factor for obtaining a contract award. Third, as a reflection of the growing complexity and sophistication of micropile works, an increasing proportion of jobs are let on a design/build basis. In this scenario, a geotechnical and/or structural consultant will often work for the micropile contractor or on an official team, as opposed to the conventional case where the consultant alone designs the work in advance and then oversees and directs its execution. In association with this trend, performance-type specifications are becoming more common, in preference to the "prescriptive" types that frequently do not have the inherent technical or contractual flexibility synonymous with a cost-effective, non-litigious, and successful execution. Assuming that the reader is well versed with respect to this conventional "low bidder" approach, emphasis is placed in this chapter on the more innovative design-build options.

As noted by Nicholson and Bruce (1992), the American specialty geotechnical construction community has historically been a follower rather than a leader. This has been due to the nature of past construction demands (e.g., building roads through the deserts rather than soft ground tunnels under old cities), and a tradition of litigious and confrontational operating conditions. One can cite the import of ground anchors, diaphragm walls, jet grouting, and soil mixing as examples of foreign concepts, whereas compaction grouting (Baker et al., 1983) is the sole "uniquely American process" being exported internationally from domestic roots. Micropiles, as noted frequently in this report, are another example of a European technology import, notwithstanding the local flavor it now displays.

Still, the most common delivery system for a construction project is the traditional "Design-Bid-Build" system, in which the steps are well defined:

- Design: The owner, or a consultant selected by the owner, designs the project.

- Bid: A suitable bidding period is established, usually 4 to 6 weeks, and any contractor who can secure suitable bonding bids the work.

- Build: The low bidder is then determined and the project is built in accordance with the plans and specifications of the owner and the consultant.

There may be some opportunities to innovate – the engineer or constructor can propose Value Engineering – but these opportunities may be limited, especially on "fast track" projects in which micropiling may be a minor element.

This present low-bidder system, used for the overwhelming majority of public works construction in the United States, has led to other problems in the industry in addition to not encouraging innovation and development. Disputes, formal arbitration, and lawsuits are becoming an everyday occurrence for many contractors. Low profit margins have led to the failure of many companies involved in the high-risk arena of specialty geotechnical contracting.

Despite these constraints, micropiling and other such innovative technologies are, of course, increasing in usage, and this is steadily encouraging new or alternative practices, mainly in the bidding and procurement stages. The American Society of Civil Engineers Specialty Conference at Cornell University, Ithaca, NY, in June 1990, dealt with the subject of "Design and Performance of Earth Retaining Structures." One section was devoted to contracting practices, and A.J. Nicholson described these innovative modifications. All are valid for establishing procurement methods for micropiles and are summarized as follows.

It is common to find contract documents that include qualification clauses that call for some limited review of a contractor's experience record by the owner or consultant. The owner's approval is (nominally) required before the specialty subcontractor may be employed by the general contractor. However, it is difficult to ensure that these clauses will perform as intended. In a very competitive bidding atmosphere, the successful general contractor usually feels he or she has a "right" to use the subcontractor of his or her choice. This often means that the general contractor will choose on the basis of cost over experience, and will rely on his or her own interpretations of the clauses to justify the selection. For example, certain "experienced" individuals can be hired temporarily, or materials or equipment suppliers can be engaged to furnish "technicians" to supervise certain more critical phases of the work. This state of affairs affords no incentive or encouragement to the innovative specialty contractor and ultimately leads to a deterioration of product quality through error, omission, carelessness, or, simply, lack of experience.

A more attractive method has been the concept of prequalification, whereby only pre-approved contractors are permitted to bid to the general contractor. About half of the State highway departments are currently using or are considering this method for certain types of work. Typically, though, general contractors find themselves besieged by subcontract bids at the last minute from "new" companies claiming to have the suitable level of expertise. If the offer is low enough, the contractor is tempted, and rarely does the owner intervene because his or her fine intentions are submerged in the self-justification of "fair and open competition." The good intentions of prequalification are further diminished by the fact that no national forum exists where standard guidelines are set. The U.S. Army Corps of Engineers and others, however, are experimenting with a contractor rating program to prohibit a contractor with an "unsatisfactory" record from previous work from bidding on other work. Each owner typically has his or her own prequalification system, and tight bidding and award schedules rarely leave time for submitted references to be verified. A drawback of even rigidly applied prequalification is that it rules out the potential for contribution by the innovative specialist during the project's conceptual and design phases. This is everyone's loss, as the team is potentially weaker.

The <u>Value Engineering Proposal</u> is a form of alternate proposal, long established in American practice. Although more progressive in concept, it has found limited use in specialty geotechnical contracting. This is not solely because cost savings must be shared, rather it is because at bid time the general contractor is typically unable or unwilling to assess the inherent risks. These risks include the fear that change may disrupt the work; the owner may not accept the scheme; and there may be insufficient time for approval. When presented with such concerns, often for little reward in return, most general contractors simply reject value-engineered proposals.

In contrast, the challenge of equitably procuring a solution using micropiles -- or any other specialty geotechnical technique -- is often best met using the Design-Build concept, common in the bidding climates of Europe and Japan and promoted for many years by the FHWA. The Design-Build concept allows the geotechnical specialist contractor to introduce cost-effective solutions that meet or exceed the owner's performance criteria. Such contracting practices promote innovative design and accelerated construction, often with the use of equipment specially built for the purpose. They are based on the use of performance-type specifications as opposed to the prescriptive types common for traditional technologies in conventional bidding methods. The traditional role of the owner's representative -- the design consultant -- is often modified and may be expanded. The design consultant not only sets the performance criteria within practical limits, but also provides assurance that the owner's needs are satisfied. Review and critique of competitive proposals from specialty contractors and consultants employed by them ensure that the most economical solution is found. There are four distinct options:

- <u>Post-Bid Design</u>. The owner prepares a set of special design criteria (special provisions) that are included in the bid invitation and define the parameters for the alternate design. An owner-designed or "as-designed" system may also be included. After successfully pricing the project and obtaining a contract, the specialist then provides a design to the owner for review and approval, and this design must satisfy the performance parameters.

A difficulty with this approach concerns the ability of the owner and contractor to agree on the design after the award has been made. Disputes and delays may result, and often the contractor must modify the design, which usually compromises potential profitability. Also, to protect himself or herself, the owner may overspecify the design parameters, and this may reduce design flexibility. However, this can be a very useful option, especially for smaller, highly technical projects.

- <u>Pre-Bid Design</u>. Prequalified, selected specialists prepare designs for the owner's review prior to the bid. Approved designs become part of the bid package and the specialty subcontractor prepares a price for construction of the proprietary design only. This method works best when the contractor is permitted to prepare plans of a conceptual nature only. Such plans exclude details that the contractor feels are unique to his or her design. As long as the supporting calculations address these details, the bid documents may include only enough information to make other contractors aware of the nature of the work.

This is a positive opportunity for innovative contractors, who of course, must still remain cost-effective.

- Negotiated Work. The owner is committed to a team approach wherein the contractor becomes an important part of the team for all foundation and ground-support aspects of the project. Risk-sharing is integral. The contractor is responsible for the adequacy of the design and its construction; the owner is responsible for the accuracy of the information upon which the design is based. Costs are reduced as the contractor includes fewer contingencies, and innovation is encouraged because the contractor is rewarded for economies of design and installation. Quality is enhanced due to the team approach.

- Two-Phase Bidding. In many ways another type of negotiated bid, two-phase bidding, has gained favor in recent years with many Federal and State agencies. Prequalified contractors are invited to submit separate very detailed technical and financial offers. The technical aims of the project and special restrictions are clearly specified, but a wide scope is afforded to the inventive bidder. Each proposal is assessed independently by separate committees and is graded on a point system disclosed in advance. The value of the technical proposal may often exceed that of the price proposal, and it emphasizes technical competence, personnel and corporate experience, and safety. There may be successive "rounds" of bidding, with the responsive contractors being interviewed between times so that they can optimize their proposals to a "best and final" submittal.

During the negotiations, the successful contractor should have developed a full understanding of the requirements of the job, and so there should be no subsequent controversy over the specifications, scope of work, or the quality level intended. Also, the successful contractor may not have the lowest bid. Unsuccessful contractors will have incurred a great deal of bidding cost, but this prospect alone will deter all but the most serious contenders. This process also involves considerable effort on behalf of the owner, and so is really viable only on particularly large and/or complex projects.

Overall, there are significant opportunities for the owner and the consultant, as well as the contractor, in pursuing design-build options based on performance specifications as opposed to the traditional low-bid approach using prescriptive specifications. In summary, the former provides benefits by:

- Providing optimum solutions at the lowest possible cost.
- Encouraging innovation through contractor-sponsored research and development.
- Fostering improvements in quality, performance, and cost.
- Incorporating the most advanced and practical designs by prequalified contractors who are regularly and exclusively engaged in the business.

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