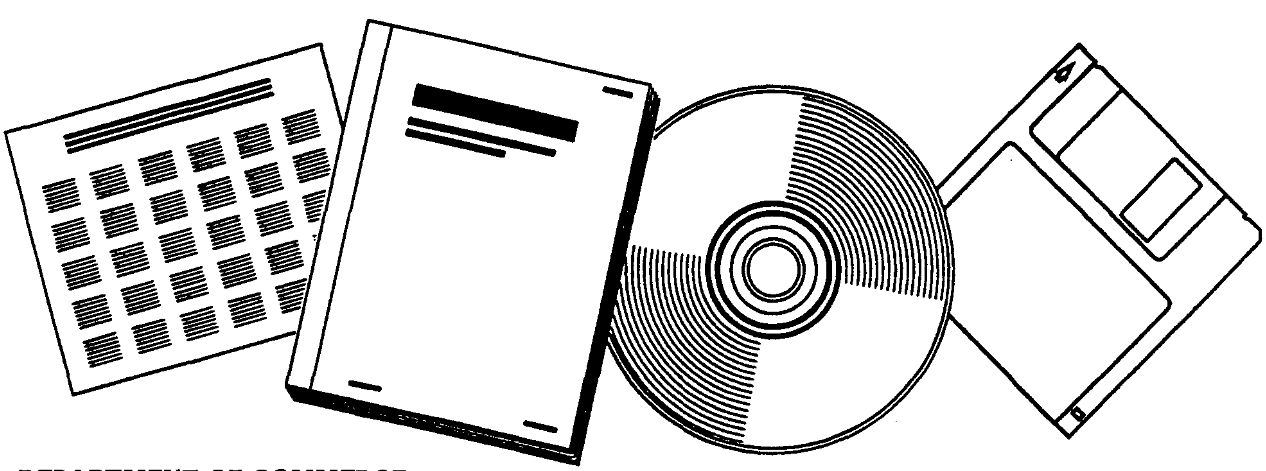
PB97-187959

DRILLED AND GROUTED MICROPILES STATE-OF-PRACTICE REVIEW VOLUME 4. CASE HISTORIES

NICHOLSON CONSTRUCTION CO., BRIDGEVILLE, PA

JUL 97



U.S. DEPARTMENT OF COMMERCE
National Technical Information Service

Drilled and Grouted Micropiles: State-of-Practice Review

Volume IV: Case Histories

PUBLICATION NO. FHWA-RD-96-019 JULY 1997

U.S. Department of Transportation
Federal Highway Administration

Research and Development
Turner-Fairbank Highway Research Center
6300 Georgetown Pike
McLean, VA 22101-2296

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

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

NOTICE

This document is disseminated under the sponsorship of the Department of Transportation in the interest of information exchange. The United States Government assumes no liability for its contents or use thereof. This report does not constitute a standard, specification, or regulation.

The United States Government does not endorse products or manufacturers. Trademarks or manufacturers' names appear herein only because they are considered essential to the object of this document.

Technical Report Documentation Page

PB97-187959

1. Report No.	2.	3. Recipient's Catalog No.
FHWA-RD-96-019		

4. Title and Subtitle	5. Report Date
DRILLED AND GROUTED MICROPILES: STATE-OF-PRACTICE REVIEW, Volume IV: Case Histories	July 1997
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Donald A. Bruce and Ilan Juran	
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
Nicholson Construction Company P.O. Box 98 Bridgeville, PA 15017	3E3A0292
	11. Contract or Grant No. DTFH61-93-C-00128
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Office of Engineering R&D Federal Highway Administration 6300 Georgetown Pike, McLean, VA 22101-2296	Final Report September 1993-July 1995
	14. Sponsoring Agency Code

15. Supplementary Notes

Contracting Officer's Technical Representative (COTR): A.F. DiMillio, HNR-10. Technical guidance provided by Ron Cheney and Jerry DiMaggio.

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

FHWA-RD-96-016 Volume I: Background, Classifications, Cost
FHWA-RD-96-017 Volume II: Design
FHWA-RD-96-018 Volume III: Construction, QA/QC, and Testing

17. Key Words	18. Distribution Statement
Drilling, grouting, micropiles, performance, design, construction, field testing, analysis, laboratory testing, state of practice	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	156	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

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)

TABLE OF CONTENTS

VOLUME I

TABLE OF CONTENTS

VOLUME II

TABLE OF CONTENTS

VOLUME II (Continued)

TABLE OF CONTENTS

VOLUME III

TABLE OF CONTENTS

VOLUME IV

LIST OF FIGURES

VOLUME I

LIST OF FIGURES

VOLUME I (Continued)

LIST OF FIGURES

VOLUME II

LIST OF FIGURES

VOLUME II (Continued)

LIST OF FIGURES

VOLUME II (Continued)

LIST OF FIGURES

VOLUME II (Continued)

LIST OF FIGURES

VOLUME II (Continued)

LIST OF FIGURES

VOLUME III

LIST OF FIGURES

VOLUME III (Continued)

LIST OF FIGURES

VOLUME IV

LIST OF FIGURES

VOLUME IV (Continued)

LIST OF TABLES

VOLUME I

LIST OF TABLES

VOLUME II

LIST OF TABLES

VOLUME II (Continued)

LIST OF TABLES

VOLUME III

LIST OF TABLES

VOLUME IV

Table | Page

LIST OF TABLES

VOLUME IV (Continued)

CHAPTER 1. INTRODUCTION

Throughout this report, and particularly in volume I, references have been made to specific micropile projects for illustrative purposes. In this volume, expanded descriptions of many of these projects are provided in standardized format. In selecting these particular 20 case histories, special attention has been paid to their technological merit. However, an equally important criterion has proved to be the completeness and quality of the data released on each project.

Many specialists simply do not publish the details of their micropile jobs for proprietary reasons or because they see no commercial advantage significant enough to warrant allocating valuable human resources. Other sources, especially specialty contractors and major material suppliers, do release information, but typically in the form of brief project descriptions designed for "mass mailings," or distribution at trade shows and conferences. Such publications are very useful in attempting to form an overall impression of the broad practices employed by one particular contractor, or in one particular country.

However, these reports, individually and typically, provide a tantalizingly incomplete account of the project, due to the space restraints of the format. An example is an account by DSI (1994) of the underpinning of the Taschenberg Palace in Dresden, Germany. The structure was completed in 1711, but totally gutted by fire in 1945. It has undergone reconstruction as a hotel, and much of the structure has been underpinned to accept new loadings. In the west wing alone, 100 Type 1D GEWI piles were installed using bars 40 and 50 mm in diameter, and bond lengths of 7 to 12 m. The photographs in the one-page information sheet clearly underline the elegance of the micropile solution. Space, however, prevents a review of design, construction, or performance data, although one may assume the project was successful!

Such partial accounts are captured in this volume in chapter 2, within the series of summary tables referred to as tables 1 through 8.

Much less common, however, is the publication of comprehensive case histories, for the reasons noted above. Such case histories are, of course, extremely valuable since in the field of specialty geotechnical construction, practice often precedes theory and the sharing of successful experiences is fundamental to the development of safe practices.

Chapter 3 provides descriptions of the 20 structural support case histories referred to above. For each, a synopsis is made of data relating to the project, namely background, site and ground conditions, design, construction, and testing and performance. Frequently, few quantitative data are published on the geotechnical conditions, this usually being a direct reflection of the extent of the data actually known. The reader should therefore exercise prudence in applying directly the specific details and procedures of these case histories to future applications. Chapter 4 provides similar details of four case histories of insitu reinforcement applications.

CHAPTER 2. TABULATED CASE HISTORIES

Table 1 presents details of American projects reported by Bruce (1988-1989; 1992). These are arranged to reflect the application classification (for structural underpinning) introduced in volume I, and as recapped in figure 1.

All these examples are of structural support involving CASE 1-type applications. No CASE 2 structures have been installed to date in the United States. These examples highlight:

- Wide range in the scale and scope of individual projects.
- Wide range in design service loads.
- Increasingly large test loads being achieved.
- Their installation in virtually every ground condition.
- Relatively narrow range of pile dimensions.
- Typical applications of restricted headroom and access conditions, within existing structures and in operational facilities.
- Common use of a permanent steel casing from the surface to the load transfer zone in certain practice.
- Excellent load-holding performance with minimal movement.

Similar trends are apparent in other published tabulations of groups of projects relating to:

Table 2:	Micropile projects in Hong Kong in the early 1980's.
Table 3:	Load-movement data compiled by FHWA (1976), but based on Fondedile's work in Italy.
Table 4:	Micropile projects in England in the early 1980's.
Table 5:	Micropile projects in England in the early 1990's.
Table 6:	Micropiles used specifically for bridge underpinning in the United States in the late 1980s and early 1990s.
Tables 7 and 8:	Micropiles used as in situ reinforcement (both CASE 1 and 2) in the United States.

These tabulations are merely indicative of the widespread use of micropiles. Far more comprehensive and lengthy lists exist within the files of scores of specialists throughout the world. As an illustration, Barley and Woodward (1992) cite 50,000 linear meters of micropiles installed by their company alone, in England, in the period 1989-1991. This would probably be regarded as relatively small in other companies in other parts of the world in the present day.

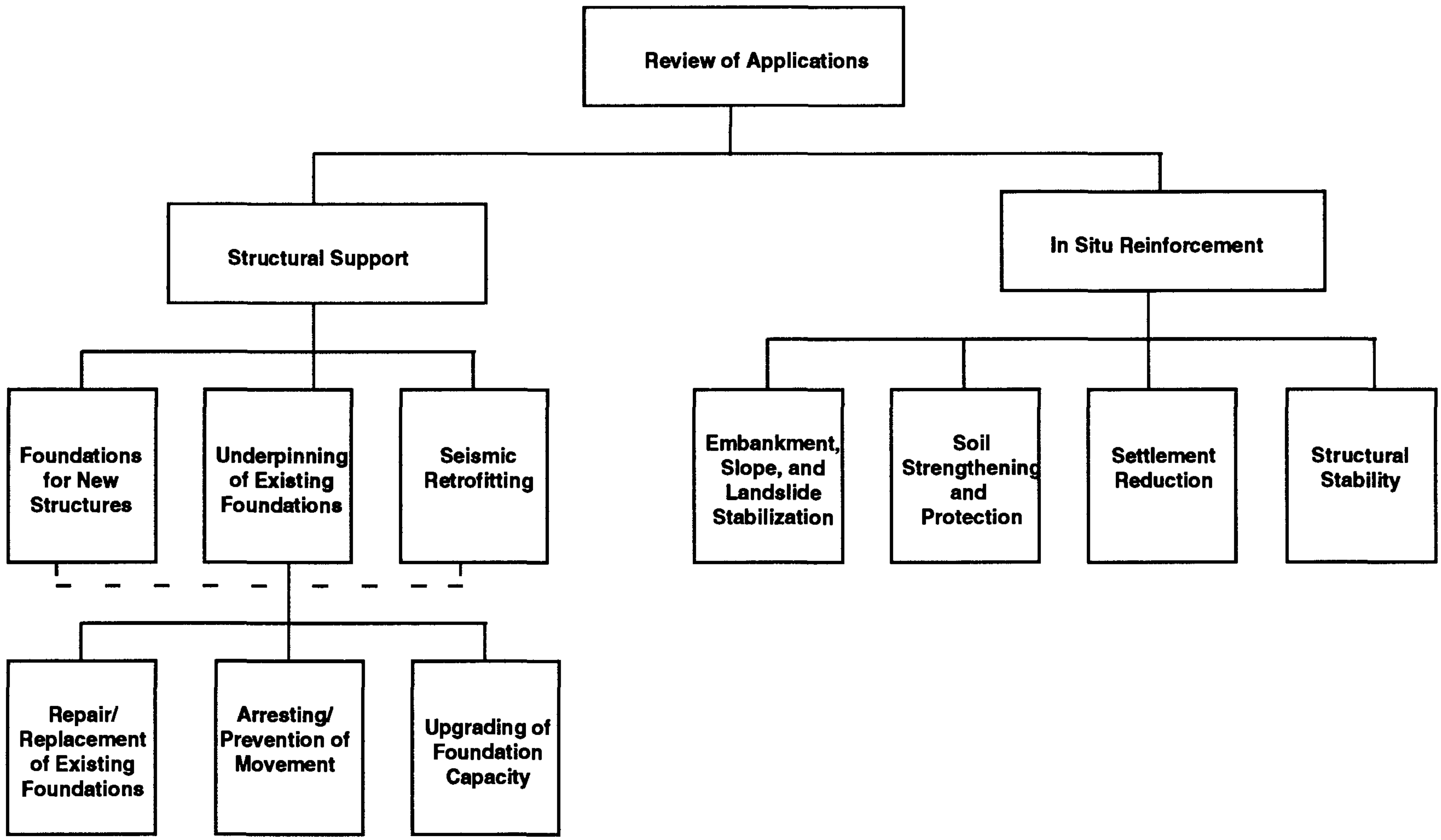


Figure 1. Classification of micropile applications.

Table 1. Details of certain U.S. micropile projects conducted from 1978 to 1990 for structural support (from Xanthakos, Abramson, and Bruce, 1994).

Location	Location/ Application for Foundations Being Underpinned	Ground Conditions	Installation Conditions	Load (kN) Working/ Test	Number of Production Piles	Total Length Installed (m)	Individual Length (m) Typical/ Range	Drilled Diameter in Bond Zone (mm)	Reinforcement and Casing	Grouting	Test Performance/ Special Notes
. Underpinning of Existing Foundations											
1. Arrest/Prevent Movement											
Apollo, PA (1A)	Support for column foundations to permit excavation of hazardous waste	Low-level radioactive fill and silty clay with rock fragments over siltstone and shale	Interior of operating steel mill, minimum 3-m headroom	534	20	244	12	140	140-mm casing	Type I [1]w/c = 0.45 Gravity	–
Burgettstown, PA (1B)	Existing gantry runway	Slag, silty sandy clay and shales over sandstone and limestone	Maximum headroom 7 m. Soil saturated with sulfuric acid	89	20	195	10	102 (for 1-m rock socket)	89-mm casing full length	Type II w/c = 0.45 Max pressure: 0.28 MPa	–
Cleveland, OH (1B)	Support to spread footings of existing pipe bridge, already settled 0.46 m	Stiff clay	Very difficult access to and under bridge	111/222	4	85	21	140	140-mm casing	Type I w/c = 0.45 Max pressure: 0.41 MPa	–
Coney Island, NY (1B)	Rehabilitation of existing repair shop	Fill and organic silt over dense sands	Minimum headroom 2.4 m. Very difficult access in fully operational facility	133/266 and 266/534	2300 1900	24,152 26,152	11 14	168 194	19-mm rebar full length, 28-mm rebar full length	Type I w/c = 0.45 Max pressure: 0.41 MPa	Extensive test program See text.

[1]w/c = water to cement ratio

Table 1. Details of certain U.S. micropile projects conducted from 1978 to 1990 for structural support (from Xanthakos, Abramson, and Bruce, 1994) (continued).

Location	Location/ Application for Foundations Being Underpinned	Ground Conditions	Installation Conditions	Load (kN) Working/ Test	Number of Production Piles	Total Length Installed (m)	Individual Length (m) Typical/ Range	Drilled Diameter in Bond Zone (mm)	Reinforcement and Casing	Grouting	Test Performance/ Special Notes
Marion, IN (1B)	Existing body stamping plant	Silty sand over rock	5.5-m headroom	534	24	512	21	178	178-mm casing for upper 8 m, #11 rebar for lower 7.5 m	Type I w/c = 0.45 Max pressure: 0.34 MPa	–
Memphis, TN (1B)	Test pile for underpinning of major transport facility	Clayey fill over sanitary landfill over loose sand and stiff clay	Unrestricted	711	1	40	40	127	127-mm casing to top of bond zone, three each, 16-mm-diameter rebars below	Type II w/c = 0.45 Max pressure: 0.69 MPa	At failure load: Total movement = 27 mm Permanent movement = 2.4 mm
Monessen, PA (1B)	Existing operating coke battery, emission control facility	Fill over clayey sand and gravel	6-to 7-m headroom	445/890 (comp) 311 or (tension) 400	102	1929	17 and 20	127	22-mm rebar full length, 127-mm casing for all except lower 3 m	Type II w/c = 0.45 Max pressure: 0.69 MPa	Test data on one pile: Total movement at 890 kN = 8 mm Permanent movement after = 0.2 mm
Neville Island, PA (1B)	Existing dust collector structure on rapidly compacting soil	Loose fill over compact sand and gravel	3-to 5-m headroom	266/534	32	283	9	127	28-mm rebar in lower 5 m, 127-mm casing in upper 6 m	Type I w/c = 0. 5 Max pressure: 0.69 MPa	Test data on one pile: Total movement at 534 kN = 2 mm Permanent movement after = 0.25 mm (allowable 15 mm)
Orangeburg, NY (1B)	Foundations for exterior stairway for existing psychiatric center	Loose fill overlying very compact glacial till	4-m x 4-m access to interior courtyard	44-338/ 178-667	103	841	8-10	203	12-to 25-mm rebar	Type I w/c = 0. 5 Max pressure: 0.55 MPa	At test load: Total movement = 2.8 mm Permanent movement after = 0.51 mm

Table 1. Details of certain U.S. micropile projects conducted from 1978 to 1990 for structural support (from Xanthakos, Abramson, and Bruce, 1994) (continued).

Location	Location/ Application for Foundations Being Underpinned	Ground Conditions	Installation Conditions	Load (kN) Working/ Test	Number of Production Piles	Total Length Installed (m)	Individual Length (m) Typical/ Range	Drilled Diameter in Bond Zone (mm)	Reinforcement and Casing	Grouting	Test Performance/ Special Notes
Pittsburgh, PA (1A)	Supporting existing columns of operating hospital to permit adjacent and ulterior excavation	Siltstone, shale, claystone	Interior of very sensitive building with 3-m headroom	1112/ 2891	18	236	13	178	178-mm casing	Type II w/c = 0.45 Gravity	See text
Pittsburgh, PA (1A)	Existing parking garage	Fill and alluvials over sandstone/ siltstone bedrock	2.4-m to 3-m headroom	489	46	603	13 (range 11.6 to 13.4)	127	127-mm casing to rock head	Type I w/c = 0.45 Gravity	--
Pittsburgh, PA (1B)	Existing structure adjacent to deep excavation	Fill and fine alluvials over dense sands and gravels with trace silt	Open air	445	21	192	9	127	127-mm casing for upper 6 m	Type I w/c = 0. 5 Max pressure: 0.41 MPa	Pile installed in conjunction with subhorizontal soil nails for excavation stability
Seattle, WA (1B, 1D)	Test program for underpinning of historic building	Sands and silts over fine and silty dense sands	Through concrete footings in old structure, headroom as low as 2.4 m	663/ 1201-1334	4 (test)	43	9-12	140	3- to 9-m casing, bond lengths with full-length 32-mm rebar	Type II w/c = 0. 5 Max pressure: 0.55 MPa	Excellent test data, including use of post-grouting
Warwick, NY (1B)	Existing gymnasium building (use of preloaded piles)	Loose sandy silt and glacial till becoming denser with depth	Minimum headroom 6 m	245/489	62	1228	20	127	Two 15-mm-diameter strands (for preloading 127-mm casing in upper 8 m)	Type I w/c = 0.45 Max pressure: 0.83 MPa	Test data on 2 piles: Total movement at 489 kN = 4.6 and 6.3 mm Permanent movement after = 0.05 and 0.127 mm, respectively

Table 1. Details of certain U.S. micropile projects conducted from 1978 to 1990 for structural support (from Xanthakos, Abramson, and Bruce, 1994) (continued).

Location	Location/ Application for Foundations Being Underpinned	Ground Conditions	Installation Conditions	Load (kN) Working/ Test	Number of Production Piles	Total Length Installed (m)	Individual Length (m) Typical/ Range	Drilled Diameter in Bond Zone (mm)	Reinforcement and Casing	Grouting	Test Performance/ Special Notes
Washington, D.C. (1B)	Existing structure at Castle Building, Smithsonian Institute	Fill over dense sands with gravel	Very restrictive access and hole entry conditions	445/890	21	485	23 (range 21 to 23.5)	140	36-mm bar full depth, 140-mm casing between footing and bond zone	Type I w/c = 0.45 Max pressure: 0.96 MPa	Piles combined with subhorizontal soil nails to stabilize excavation adjacent to structure, Data on Test Pile 2: Total movement at 890 kN = 16.6 mm Permanent movement after = 2 mm
2. Repair/Replace Existing Foundations											
Baltimore, MD (1B)	Intensive underpinning of historic 5-story building threatened by deterioration of original wood piles	Peats and clayey silt over silty fine sands	Very restricted access, 2.4- to 3-m headroom	663/ 2224	121	1372	11-12	178	4.5 to 6 m of upper 178-mm casing with 6 to 7.6 m of 35-mm rebar in bond zone	Type I w/c = 0.45 Max pressure: 0.69 MPa	Described in *Civil Engineering* in December 1990
Mobile, AL (1B)	Two existing sodium hydroxide storage tanks under which wood piles had failed	Soft organic silt and clay over dense sand with gravel	Very restricted access, 2.4- to 4.6-m headroom. Caustic chemical spills	302 480	171 7	2926 122	17 (range 14 to 18)	127 168	127 or 168 mm for full length except lower 2.4 m	Type I w/c = 0.5 Max pressure: 0.55 MPa	Piling part of major overall structural repair. See text.
Pocomoke City, MD (1B)	Replacement foundations of 60-year-old delicate bascule bridge	River bed silts and clays over 9-m dense fine to medium sands	Most from bridge deck, 4 from very limited access/ headroom	445/890	52	1585	30	178	178-mm casing plus 35-mm high-yield rebar in bond zone	-	See text

Table 1. Details of certain U.S. micropile projects conducted from 1978 to 1990 for structural support (from Xanthakos, Abramson, and Bruce, 1994) (continued).

Location	Location/ Application for Foundations Being Underpinned	Ground Conditions	Installation Conditions	Load (kN) Working/ Test	Number of Production Piles	Total Length Installed (m)	Individual Length (m) Typical/ Range	Drilled Diameter in Bond Zone (mm)	Reinforcement and Casing	Grouting	Test Performance/ Special Notes
Providence, RI (1A)	Test to assess viability of underpinning existing granite block seawall	Quay, bearing on silt, sand and till overlying sandstone bedrock	Open air	489/978	1 (Test)	20	20	182	127-mm casing for 17 m	Type I w/c = 0.45 Gravity	Test data on one pile: Total movement at 978 kN = 17.7 mm Permanent movement after = 0.76 mm
Rome, GA (1A)	Support for foundations in operational paper mill	Fills over shales with quartzitic seams	Access through doorways, minimum 3.6-m headroom	845/ 1690	33	402	12	140	140-mm casing	Type I w/c = 0.45 Gravity	-
State College, PA (1A)	New column foundations for fire-damaged church	Clay over karstic limestone	Difficult access, low headroom	178/311	50	533	11	140	140-mm casing to rock, 25-mm rebar in bond zone	Type I w/c = 0.45 Gravity	-
3. Upgrade Load Capacity											
Augusta, GA (1D)	Underpinning of footings subjected to additional loads in operational detergent factory	7-m clay over various medium to fine sands with interbedded clays	Very restricted access, minimum 2.4-to 3-m headroom	445/890	143	1613	11	194	35-mm-diameter high-yield rebar, plus 194-mm casing in upper 3 m for lateral resistance	Type I w/c = 0.45 Max pressure: 0.41 kN	Routine use of post-grouting to enhance soil-grout bond
Boston, MA (1B)	Underpinning of existing building being redeveloped	Fill and soft clay over bouldery till	Very restricted access, minimum 2.4-m headroom	534/ 1068	97	1478	15	178	50-mm high-yield rebar	Type I w/c = 0.45 Max pressure: 0.41 kN	Two tests to 1067 tons: Total movement = 5.7 mm Permanent movement after = 1.27 mm

Table 1. Details of certain U.S. micropile projects conducted from 1978 to 1990 for structural support (from Xanthakos, Abramson, and Bruce, 1994) (continued).

Location	Location/ Application for Foundations Being Underpinned	Ground Conditions	Installation Conditions	Load (kN) Working/ Test	Number of Production Piles	Total Length Installed (m)	Individual Length (m) Typical/ Range	Drilled Diameter in Bond Zone (mm)	Reinforcement and Casing	Grouting	Test Performance/ Special Notes
Boyston St. Boston, MA (1B)	Existing building being redeveloped	Soft fills and organics over medium dense sand	Minimum headroom 2.4 m in very restrictive basement conditions	356/818 (comp) 107/240 (tension)	262	2155	8	140	25-mm rebar full length, 140-mm casing in upper 6 m	Type II w/c = 0.5 Max pressure: 0.41 kN	Test data on two piles: Total movement at 818 kN = 11.2 and 8.6 mm Permanent movement after = 6.4 and 4.1 mm. See text.
Pittsburgh, PA (1A)	Restoration of existing Timber Court Building	Sands and gravels, over sandstone bedrock	3-m headroom	445	15	320	21	140	140-mm casing full length	Type I w/c = 0.45 Gravity	--
Washington, D.C. (1B)	Underpinning for new and existing foundations for historic, massive building being refurbished	Fill over various alluvial fine to medium sands with cobbly/ clayey horizons	Existing basement with 2.4-to 5.2-m headroom in three areas	667/ 1334	609	11430	16 to 18	178	7-to 9-m casing plus 7.6 m of 35-mm rebar in bond zone	Type II w/c = 0.45 Max pressure: 0.69 MPa	See text
Foundations for New Structures											
1. Restrictive Site or Access											
Ann Street, Pittsburgh, PA (1A)	To support new soldier beams for new retaining wall	Weathered shale and sandstone over competent sandstone	Open air	427/605 (comp) 71/107 (lateral)	86	305	4	152	36-mm high-strength rebar full length	Type I w/c = 0.45 Gravity	Pile subjected to vertical, lateral, and moment testing Compression test data on six piles: Total movement at 605 kN = 1.5 to 2.5 mm Permanent movement after = 0.15 to 0.5 mm

Table 1. Details of certain U.S. micropile projects conducted from 1978 to 1990 for structural support (from Xanthakos, Abramson, and Bruce, 1994) (continued).

Location	Location/ Application for Foundations Being Underpinned	Ground Conditions	Installation Conditions	Load (kN) Working/ Test	Number of Production Piles	Total Length Installed (m)	Individual Length (m) Typical/ Range	Drilled Diameter in Bond Zone (mm)	Reinforcement and Casing	Grouting	Test Performance/ Special Notes
Apollo, PA (1B)	New tank in existing wastewater treatment plant	Loose fill with concrete obstructions over clay over medium to very dense sands with silt and gravel	Plant measured 11.6 m x 14.6 m in plan. Maximum headroom 5.5 m	89/178	45	411	9	127	36-mm rebar in lower 6 m plus 127-mm casing in upper 4.5 m	Type I w/c = 0.5 Max pressure: 0.69 MPa	Test data on two piles: Total movement at 178 kN = 1.2 and 2.0 mm Permanent movement after = 0.2 and 0.56 mm
Brooklyn, NY (1B, 1D)	Temporary and permanent piles to support overhead roadway	Fine to medium glacial sands with silts and clays	Reasonable access, 4.9 m plus headroom	534-890/ 1067-2224	77	1295	15 to 18	178	178-mm casing	Type II w/c = 0.45 Max pressure: 0.55 MPa	Excellent vertical and lateral testing with post-grouting. See text.
Cleveland, OH (1A)	Foundations for new electric furnace in existing building	Slag fill, soft silty clay over shale bedrock	Low headroom	311/__	12	457	38	140	140-mm casing plus 25-to 35-mm rebar in bond zone	Type I w/c = 0.45 Gravity	–
Covington, VA (1B)	Foundations for pipe bridge foundations for mill expansion	6-m soils over 4.6-m shale and limestone	Through and around existing foundations	890/__	172	1835	11	152	178-mm casing to rock, three each 44-mm rebar in bond zone	Type I w/c = 0.45 Max pressure: 0.62 MPa	–
Huddleston, VA (1A)	Foundations for new river bridge	4.6 m of alluvials and weathered rock over granite	Good access, unlimited headroom	623/1246	72	579	8	178	178-mm casing plus 35-mm high-yield rebar in bond zone	Type I w/c = 0.45 Gravity	–

Table 1. Details of certain U.S. micropile projects conducted from 1978 to 1990 for structural support (from Xanthakos, Abramson, and Bruce, 1994) (continued).

Location	Location/ Application for Foundations Being Underpinned	Ground Conditions	Installation Conditions	Load (kN) Working/ Test	Number of Production Piles	Total Length Installed (m)	Individual Length (m) Typical/ Range	Drilled Diameter in Bond Zone (mm)	Reinforcement and Casing	Grouting	Test Performance/ Special Notes
Kingsport, TN (1A)	New storage tank in existing building	Silts and sands over limestone	3.4-m headroom	356/712	115	1227	11	140	25-mm rebar in lower 4.5 m, 140-mm casing to bedrock	Type I w/c = 0.45 Gravity	No measurable permanent movement after testing to 712 kN
Pittsburgh, PA (1A)	Foundation for pedestrian bridge	Backfill over claystone	6-m headroom within 0.45 m of existing structure	667/___	12	165	14	152	140-mm casing	Type I w/c = 0.45 Gravity	–
Trafford, PA (1B)	New printing press in existing building	Loose cinder fill over silty clay and weathered shale bed-rock	4.3-m headroom	89/178	20	219	11	127	127-mm casing full length	Type II w/c = 0. 5 Max pressure: 0.69 MPa	Total movement at 178 kN = 1.4 mm Permanent movement after = 0.127 mm
2. Difficult Geologic Conditions											
Alcoa, TN (1A)	New building in existing mill	Limestone	Open air	623/ 1246	N/A	N/A	12	140	N/A	N/A	Total movement at 1245 kN = 11.6 mm Permanent movement after = 2.0 mm
Apollo, PA (1C)	New nuclear power structure in existing building	Loose fills with clay over medium sands with gravel	6-m headroom	89	24	168	7	140	22-mm rebar full depth, 140-mm casing for upper 5.5 m	Type II w/c = 0.45 Max pressure: 1.03 MPa	–
Baltimore, MD (1A)	Foundation for temporary highway bridge	7.6 m of alluvials and weakened material over schist	Unrestricted	667	4	30	8	178	178-mm casing	Type I w/c = 0.45 Gravity	–

Table 1. Details of certain U.S. micropile projects conducted from 1978 to 1990 for structural support (from Xanthakos, Abramson, and Bruce, 1994) (continued).

Location	Location/ Application for Foundations Being Underpinned	Ground Conditions	Installation Conditions	Load (kN) Working/ Test	Number of Production Piles	Total Length Installed (m)	Individual Length (m) Typical/ Range	Drilled Diameter in Bond Zone (mm)	Reinforcement and Casing	Grouting	Test Performance/ Special Notes
Cleveland, OH (1A)	New addition to existing control building	Slag fill and soft silty clay over shale bedrock	Open air but difficult access due to ongoing steel plant operations	534	45	1948	43	165 (for 1.5-m rock socket)	178-mm casing to rock head, 25-mm rebar for 1.5-m rock socket and 3 m into casing	Type I w/c = 0.45 Gravity	–
Jeanette, PA (1A)	New machine in existing building	Fill, silt, and clay over bedrock	6-m headroom	Total of 1334 kN of structural weight	27	288	11	140	140-mm casing full depth	Type I w/c = 0.45 Gravity	–
Montgomery County, PA (1A)	Foundations for new bridge abutment	Silty soil over karstic limestone	Overhead power lines	685/ 1201	48	313	9 to 24	216	244-mm casing to rock, 178-mm casing full length	Type I w/c = 0.45 Gravity	At test load: Total movement = 7.4 mm Permanent movement after = 0.5 mm
Warren County, NJ (1B)	New bridge pier	Karstic limestone with voids and gouge	Open air, small area	890/ 1993	24	576	24(ranges 13 to 61)	216	178-mm casing full length	Type III w/c = 0.5 Max pressure: 0.34 MPa	Test data on one pile: Total movement at 1824 kN = 10.2 mm Permanent movement after = 1.8 mm. See text.
3. Environmentally Sensitive Areas											
Aliquippa, PA (1B)	New emission control building at existing coke battery	Slag fill over dense sand and gravel	7.6-m headroom	445/890 (comp) 667/ 1334 (tension)	31 8	661 183	21 23	127 127	18-mm rebar for lower 7.5 m, 127-mm casing for upper 15 m	Type I w/c = 0.5 Max pressure: 0.83 MPa	Test data on one pile: Total movement at 890 kN = 5.0 mm Permanent movement after = 0.5 mm

Table 1. Details of certain U.S. micropile projects conducted from 1978 to 1990 for structural support (from Xanthakos, Abramson, and Bruce, 1994) (concluded).

Location	Location/ Application for Foundations Being Underpinned	Ground Conditions	Installation Conditions	Load (kN) Working/ Test	Number of Production Piles	Total Length Installed (m)	Individual Length (m) Typical/ Range	Drilled Diameter in Bond Zone (mm)	Reinforcement and Casing	Grouting	Test Performance/ Special Notes
Brookgreen Gardens, SC (1B)	Supported masts of suspended net forming "natural" aviary in swamp, with minimal damage to environment	Loose sands and organics over medium to dense sand	Natural cypress swamp	489 generally (133 for center pile)	25	358	9 to 11 for verticals, 17 for rakers	127	28-mm rebar full length, 127-mm casing for upper 6 m	Type I w/c = 0.5 Max pressure: 0.83 MPa	Award-winning solution to unique set of problems. See text.
Dunbar, PA (1A)	Addition to water treatment plant	Fill over fine sand and sandstone	Open air	400	7	55	7.9 (ranges 7.6 to 7.9)	127	#6 rebar for lower 3 m, 127-mm casing for upper 6 m	Type III w/c = 0.45 Gravity	–

Table 2. Micropile projects conducted by Bachy Soletanche Group, Hong Kong (Bruce and Yeung, 1983).

Example	Contract Details	Purpose of Piles	Ground Conditions	Pile Characteristics	Working Load	Justification forSelecting Minipiles
1	Ma Tau Kok Gas Works (Owner: Hong Kong and China Gas)	Transfer of light machine loads through compressible fill.	Fill overlying cdg	100-mm-dia. hole, 6 m long. Single 32-mm HY bar. Gravity grouting. Qty 20	50-kN Compression	Very restricted working area between existing installations.
2	Hong Kong & Shanghai Bank Annex (HSBC)	Underpinning of annex founded on raft to eliminate movements due to adjacent excavation.	Fill, marine deposits, and cdg overlying granite	125 mm min. dia 25 m long. API tube (114/87 mm dia.) ±40-mm HY bar. Socketted 3 m into rock. Gravity grouting. Qty 154	660-kN or 945-kN Compression	Restricted headroom (less than 4 m).
3	Power Transmission Line (China Light & Power)	Foundations for pylons	Silt (N:10-20)	168-mm-dia. hole, 20 m long. Single 50-mm HY bar. Pressure-grouting. Qty 654	410-kN Compression and Tension	Difficult access to large number of tower locations (frequently only by helicopter)
4	MTRC 809: Cotton Tree Drive (MTRC)	Foundations for footbridge.	Fill, marine deposits, and cdg overlying granite	168-mm-dia. hole, 26-37m long. One to three HY bars (40 to 50 mm). Socketted 5-6 m into rock. Gravity grouting. Qty 23	Up to 1230kN (for three 50-mm bars) Compression	Small loading, bouldery ground. Restrictions on ground-water lowering and cost-eliminated hand-dug caissons

Table 3. Summary of load test data (prepared by FHWA, 1976 from Bruce and Yeung, 1983).

Soil Type	Nominal Diameter mm	Length m	Assumed Effective Length m	Max. Test Load kN	Settlement at Max. Load mm	Location
G	101.6	6.4	6.4	224.0	1.02	School Building, Milan, Italy
C	101.6	12.2	12.2	224.0	4.06	Olympic Swimming Pool, Rome
G	304.8	27.4	27.4	515.1	8.13	Bausan Pier, Naples
Si, G	101.6	14.9	6.1	201.6	2.03	Italian State Railroad, Rome
G	101.6	15.8	12.8	179.2	2.29	Bank of Naples
G	215.9	30.2	20.1	1099.4	5.59	Corps of Engineers, Naples
G	127.0	19.8	7.3	509.0	8.13	Washington, DC, Subway
G	228.6	5.9	3.0	458.1	11.43	Queen Anne's Gate, London
G	177.8	8.5	5.5	509.0	7.62	Queen Anne's Gate, London
C-G	101.6	16.1	16.1	235.2	5.99	Salerno Mercatello Hospital
G	203.2	25.1	13.1	1099.4	11.99	Marinella Wharf, Port of Naples
G	203.2	14.5	14.5	604.7	0.89	Main Switching Plant, Genoa
G	203.2	22.3	22.3	636.3	1.65	Mobil Oil Italiana, Naples
G	203.2	20.1	20.1	597.6	0.94	Railway Terminal, Naples (Corso A. Lucci)
G	203.2	19.2	19.2	575.2	1.65	Plant (Brindisi)
G	203.2	18.4	18.4	575.2	0.71	Plant (Brindisi)
C	203.2	22.4	22.4	280.0	6.40	Special Foundations for Transmission (Electrical Towers between Garigliano-Latina)
C	203.2	20.1	20.1	246.4	9.80	Special Foundations for Transmission (Electrical Towers between Garigliano-Latina)
C	203.2	20.1	20.1	493.7	5.21	Special Foundations for Transmission (Electrical Towers between Garigliano-Latina)
G	203.2	30.2	20.1	1121.8	5.41	Belt (Expressway) East-West, Naples
G	203.2	30.2	20.1	897.9	3.23	Belt (Expressway) East-West, Naples
G	203.2	18.1	18.1	695.3	1.55	Swimming Pool — Scandone Pool, Naples
G	101.6	10.1	10.1	218.9	2.21	Casa Albergo in Viace Piave
G	215.9	25.1	25.1	709.5	3.76	Port of Naples
G	215.9	25.1	25.1	709.5	3.81	Port of Naples

G = Granular
C = Clay
Si = Silt

NB: Maximum test load does not mean failure load

Table 4. Summary details of micropile projects in England in the early 1980's (Bruce et al., 1985).

Name	Purpose of Minipiles	Ground Conditions	Working Load (kN)	Nr	Total Length Installed (m)	Av. Length (m)	Pile Diameter (mm)	Pile Construction
Church of St. Stephen's Wallbrook, London	To underpin existing church piers and as foundations for new construction	Fill and gravels over London Clay	200	73	1688	23	200	6Y20 bars and 6-mm helical reinforcement. 150-mm pitch. Permanent steel casing in fill. 1:1 sand:SRC grout, w/c = 0.4
Wilford Toll Bridge, Nottingham	To increase the load-bearing capacity of pre-existing caisson foundations	Alluvial gravel (+ artificial obstructions) over sandstone	625	28	377	13.5	220	5 Hy 16 bars and 6-mm helical reinforcement 1.5:1 sand:OPC grout WSR = 0.5
Canning Dock, Liverpool	To underpin and support 19th Century dockwall	Masonry/fill/ sandstone	Compression 490. Tension: 700	C1:105 C2:105 T: 93	4120 1674	10.1 18	127 mm through masonry and fill 105 mm in rock	Compression: 50-mm Dividag GEWI bar in 80-mm corrugated pvc duct. Tension: 36-mm-dia Dividag bar in 65-mm-dia. pvc duct. Neat OPC grout w/c 0.45. Note: The C2 compression piles were extended above required bond length with 30-mm protected bars to "stitch" the masonry wall.
B.R. Bridges, 16 & 17, Romsey	To underpin existing bridge	Masonry/fill/ clay/gravel	Compression 490 Tension 450	29 18	312 335	10.8 18.6	133	50-mm Dyvidag bar in 80-mm corregated PVC duct.
B.R. Bridge, Herne Bay	To transfer upgraded loading on pre-existing bridge footings	Masonry/fill/ London Clay	130	36	504	14	170	32-mm McCall bar in 80-mm pvc duct. 1:1 sand:OPC grout WSR = 0.45
Warehouse at Narrow St., London	To support increased loadings arising from reconstruction	Alluvium over London Clay	190	138	1874	14.5	170	Permanent steel casing in alluvium 4 x Y12 reinforcing bars and 6-mm spiral reinforcement 1.5:1 sand:OPC grout WSR = 0.45

Table 5. Test pile construction and performance data (Barley and Woodward, 1992).

Pile Location	Arrowhead Quay Isle of Dogs	Ocean Quay Southampton (Shaft)	Ocean Quay Southampton (Underreamed)	Wetherby West Yorkshire	Cousins Lane London
Pile Length	26m	24m straight shaft	24m underreamed	31m	28m
Casing Dia. and Length	220mm x 12m	220mm x 16m	220mm x 16m	220mm x 28m	220mm x 14m
Open Hole Dia and Length	127mm (cased) x 14m	190mm x 8m	190mm x 8m	190mm x 3.0m	190mm x 14m
"Overburden" Strata	Loose silty sands with clay MD sandy gravel	Loose to MD made ground of concrete brick with clay	loose to MD made ground of concrete brick with clay	Fill/sandy clay /soft to firm clay/weathered limestone band	4m fill and 9m terrace gravel
"Designed" Founding Stratum	5.5m stiff silty clay /clayey silt.7m dense silty fine sand	Stiff to very stiff silty occ. bands silt and sand	clay with lenses and	Bedrock Limestone	Stiff London Clay
Pile Slenderness Ratio (Pile Length/Pile dia Min)	205	126	n/a	163	147
"Overburden" Slenderness Characteristic (Cased Length/Casing Dia.)	55	73	73	127	64
Max. Test Load	800 kN	1000 kN	1500 kN	1600kN then 1800kN	750 kN
Total Settlement	5.74mm (4.5% pile dia)	7.41m (4% pile dia)	9.64m (5% pile shaft dia	13.42mm (1600kN) 23.5mm (1800 kN) 67.5mm (700kN)	31.84mm
"Elastic" Pile Length	4.3m	5.1m	5.6m	7.7m & 25.5m	22.0m
Permanent Settlement	1.19m (1.1% pile dia)	1.95mm (1.0% pile dia)	0.69mm (0.4% pile shaft dia)	2.54mm (1600 kN). 49.5mm(after 1800kN) (26% pile dia)	11.31mm (5.9% pile dia)
Max. Bond Stress on Designed Length	200 kN/mm²	209 kN/m²	n/a due to u/ream	1005 kN/m²	90 kN/m²
Estimated Ultimate Capacity from Chin Plot	1123 kN	1510 kN	2800kN		1050 kN
Estimated Ultimate Bond Stress from Chin Plot	225 kN/m²	316 kN/m²	due to under ream contribution		126 kN/m²

Table 6. Summary of case histories of micropiles for bridge underpinning (Bruce et al., 1993).

LOCATION	DESCRIPTION	GROUND CONDITIONS	PHYSICAL CONSTRAINTS	PILE LOAD WORKING/TEST (TONS)	NO. OF PROD. PILES	TOTAL LENGTH INST. (ft)	TYPICAL PILE LENGTH INST. (ft)	NOM. BOND ZONE DIA. (in)	TYPE-STRUCTURAL COMPOSITION
L.R. 45139 Montgomery Cnty., PA	Foundations for new bridge abutment	Silty soil over pinnacled karstic limestone	Overhead power lines	77/235	48	1026	29-60	8-1/2	R-2 - 9-5/8" pipe to rock, 7" casing full length
Route 732 over Goose Creek Huddleston, VA	Foundations for new river bridge	15' of alluvials and weathered rock over granite/gneiss	Narrow bench, unlimited headroom	70/140	72	1901	25	8	R-2 - 7" pipe full length
Route 675 over Pocomoke River Pocomoke City, MD	Replacement foundations for 60-yr-old delicate bascule bridge	River bed silts and clays over 30' dense fine-medium sands	Most through bridge piers, 4 from very limited access/ headroom	60/100	52	5200	100	10	S-1 - 7" pipe plus 1-3/8" high-yield rebar in bond zone and a strand anchor for preloading
Brooklyn-Queens Expressway Brooklyn, NY	Temp. and Perm. piles to support overhead roadway	Fine-medium glacial sands with silts and clays	Reasonable access, 15' + headroom	60-100/ 120-250	368	20,200	50-60	10	S-2 - 7" pipe full length
Jones Falls Rd. Baltimore, MD	Foundation for temp. highway bridge	25' of alluvials and weakened material over hard schist with old footing obstructions	Unrestricted	75/N/A	12	300	25	7	R-2 - 7" pipe full length
Route 9A Millers Highway Viaduct Manhattan, NY	Foundations for new elevated structure bents and abutments	Wood cribbing manstones and obstructions 70' to schist	Unrestricted	100/N/A	93	4644	40-92	6-1/2 - 8	R-1 with 7" pipe and 2 grade 150 rebars, and R-2 with 7" pipe full length
S.R. 1003 Bridge over Mahoning Creek Armstrong Cnty., PA	Foundations for two new abutments and long wing wall	Old sandstone foundations, alluvials, over sandstone	Tight plan area on river bank abutment	65/175	174	5140	21-25-63	8	R-2 - 7" pipe full length
U.S. Route 11 Williamsport, MD	Underpinning of existing bridge piers	Clay over calcarious shale	Tight access, 25' headroom	75/N/A	48	1379	28-32	10	R-1 - 11-3/4" pipe plus 1" 60-ksi rebar in bond zone
U.S. Route 220 Botetourt Cnty., VA	Foundations for new river bridge	Clay and alluvials over karstic limestone	Tight plan area in cofferdam	110/160	91	2995	24-63	8-1/2	R-2 - 9-5/8" pipe through overburden, 7" casing full-length
I-78 Delaware River Crossing Warren Cnty, NJ	Foundations for new pier	Residual soils over karstic limestone	Unrestricted overhead, large voids in rock	100/224	24	2600	45-170	8-1/2	R-2 - 9-5/8" pipe to rock, 7" pipe full length

1 ton = 8.9 kN
1 ft = 0.305 m
1 in = 25.4 mm

Table 7. Summary of reticulated micropile wall case histories (Pearlman et al., 1992).

Number	Project Name, Location, and Reference	Construction Date	Ground Conditions	Slope Geometry Upslope/Downslope	Depth to Slide at Wall (ft)
1	Forest Highway No. 7 Mendocino National Forest, CA Walkinshaw (1977); Palmerton (1984)	1977	Micaceous phyllite (ϕ = 13°, c = 500 lbf/ft^2) and schistose bedrock	2H–1V/2H–1V[b]	55
2	Route 23A Catskill State Park, NY Murray (1984); Palmerton (1984)	1977	Moist, very dense glacial till with boulders (ϕ = 30°, c = 0) and shale bedrock	2H–1V/2H–1.3V	10–26
3	PA-306 Monessen, PA Dash and Jovino (1980)	1979	Random fill and colluvium (ϕ = 17°, c = 100 lbf/ft^2 γ = 134 lbf/ft^3) and weak shale	2.5H–1V/4H–1V	20
4	L.R. 69 Armstrong County, PA Earth, Inc. (1986); Dash (1987)	1985	Random fill, wet sandy clay, and hard clay (ϕ = 30°, c = 0, γ = 120 lbf/ft^3 ϕ = 17° along slide plane)	Level/1.2H–1V	34
5	Glady-Durbin Road No. 44 Randolph County, WV NCC[c] project files	1987	Sandy clay with rock fragments (ϕ = 30°, c = 0, γ = 120 lbf/ft^3) and sandstone bedrock	Level/1H–1V	16–40
6	S.R. 4023 Armstrong County, PA NCC project files	1988	Random fill, stiff colluvial clay with rock fragments (ϕ = 17°, c = 0, γ = 125 lbf/ft^3) and weathered claystone/competent-sandstone	Level/2H–1V	23–35
7	Blue Heron Road Big South Fork River, KY COE[d] project files; NCC project files	1989	Medium stiff/stiff silty clay and shale bedrock (ϕ = 19°, c = 0, γ = 125 lbf/ft^3)	Level/1.5H–1V	25

[a]1 ft = 0.305 m; 1 lbf/ft^2 = 47.9 N/m^2 ; 1 lbf/ft^3 = 159 N/m^3
[b]2H-1V—2 horizontal to 1 vertical
[c]NCC—Nicholson Construction Company
[d]COE—U.S. Army Corps of Engineers

Table 8. Summary of reticulated micropile wall case histories: wall geometry and performance (Pearlman et al., 1992).

Number	Cap Beam Geometry (L × W × T) (ft)	Pile Diameter (3) (in)	Rebar Size (#)	Pile Density (pile/ linear ft)	Pile Inclination (° from vertical)	Maximum Length (ft)	Maximum Embedment Below Slide (ft)	Lateral Displacement (in)	Remarks
1	310 × 6 × 3 50 × 5 × 3	5 4.5/rock	9	2.33	19 to −19[b]	69 53	8	N.M.[e]	Cap constructed before pile construction
2	250 × 11 × 1.75	4	10	2.80	15 to −15	50	24–30	4 D.C.[f] 0.3 A.C.[g]	Cap constructed before pile construction
3	200 × 6 × 2.5	5	9	2.25	19 to −19	45	9	0.1–0.8 A.C.	Failure of down-slope after con-struction; little wall movement
4	310 × 6 × 2.5	6	11/14	1.33	8 to −8	50	15	0.1 A.C.	Anchors inclined at 40° to vertical installed for rapid drawdown case
5	115 × 5 × 3 and 50 × 5 × 3	5.5/soil 4.5/rock	11/14/18	1.00	16 to −2	37[c] 53[d]	8	N.M.	Cap constructed before pile construction
6	122 × 4.6 × 3[c] 122 × 4.6 × 3[d]	5.5/soil 4.5/soil	11/14/18	1.25[c] 0.80[d]	26 to 0	60	17[c] 24d	1.0 D.C. 0.7 A.C.	Cap constructed after pile construction
7	34 × 5 × 3	5.5	11/14	0.75	19 to −5	35	10–15	0.6 D.C. 0.3 A.D.	Cap constructed after pile construction

[a] 1 ft = 0.305 m; 1 in = 25.4 mm
[b]Upslope is + angle.
[c]Wall A.
[d]Wall B.
[e]N.M.—Not measured.
[f]D.C.—During construction.
[g]A.C.—After construction.

CHAPTER 3. MICROPILES AS STRUCTURAL SUPPORT: SELECTED CASE HISTORIES

UNDERPINNING OF EXISTING STRUCTURES

Table 9 lists the projects referred to in this report in terms of both their application and the *major* factor influencing their choice. Clearly, certain factors are more important or more common than others, although rarely is there only one factor of significance.

Arrest or Prevention of Structural Movement or Improvement of Stability

St. Stephen's Church, London, England (Bruce et al., 1985)

Background. The church was designed by Sir Christopher Wren and completed in 1679. It stands on a site that had been in human use since the third century, and it had suffered from progressive differential settlement over many years. The effects were most noticeable along a line mirroring the original course of the Wallbrook, an old tributary of the River Thames, and which had been filled in with miscellaneous materials over the centuries. Certain piers had settled by as much as 80 mm. Given the extreme delicacy of the structure, the localized nature of the support required, the anticipated highly variable ground conditions, and the very restricted entry/exit and access conditions, Type 1A micropiling proved the logical solution for its underpinning. Support by grouting had been examined, but was rejected on a number of technical grounds, including the question of permanence. In addition, the equipment used (diesel-hydraulic drilling rig and electric grouting plant) minimized noise emission – a critical factor, bearing in mind the Church's location in the heart of the city.

Site and Ground Conditions. Entry into the structure was extremely difficult. The drilling rig had to be introduced by crane through a 3.2-m-square temporary opening formed halfway up one of the rear walls. Inside, the floor and columns had to be protected against mechanical damage and construction debris by wood and plastic sheeting. There was, however, sufficient headroom to use the full 4-m-long drill mast. The soil profile comprised about 8 m of miscellaneous saturated fills and alluvial gravels overlying stiff London clay.

Design. To satisfy the calculated pile service load of 200 kN, 14 m of penetration into the London clay was specified, based on a factor of safety of 2.5 at the grout/clay interface. Pile head movement at this load was anticipated as being about 5 mm – an acceptable amount. Columns supporting the cupola were underpinned by groups of six piles at 950-mm centers, while the other columns had groups of four piles. Six piles were installed to support the altar and a central beam for a new floor slab. A total of 73 piles were foreseen.

Construction. Drilling was conducted with 200-mm-diameter hollow stem augers to minimize vibrations, to prevent the removal of fines, and to ease the collection and disposal of drill spoils. That length of each pile above the clay was permanently lined with an 8-mm-thick steel casing to strengthen the pile through the fill, to minimize grout travel, and to provide a positive cut-off

Table 9. Matrix summarizing significance of case histories described in detail in chapter 3 (structural support applications).

Factors Influencing the Choice of Micropiles	Structural Support						
	Underpinning of Existing Structures			Foundations for New Structures			Seismic Retrofitting
	Arrest or Prevent Structural Movement	Repair and/or Replacement of Existing Foundations	Upgrade Foundation Capacity	Restrictive Site or Access	Difficult Geologic Conditions	Environmentally Sensitive Area	
Physical Constraints	•Railway Tunnel, Salerno, Italy *St. Stephen's Church, London, UK	*United Grain Terminal Vancouver, WA •Law Courts, Marseilles, France	*Boylston St., Boston, MA *Old Post Office Building, Washington, D.C. •Vandenburg AFB, California	Adda River Bridge, Italy *Brindisi, Italy (ENEL) *Brooklyn-Queens Expressway, New York City			*I-110, Los Angeles, CA *Britannia School, Vancouver, BC
Environmental Constraints	*Coney Island, NY	*Pocomoke River Bridge, MD	*Presbyterian Univ. Hospital Pittsburgh, PA *Taschenberg Palace, Dresden Germany			*Brookgreen Gardens, SC *Hong Kong Country Club *Mobile, AL	
Challenging Soil/Ground Conditions		*Albert Docks, Liverpool, England			•PanPacific Hotel, Malaysia •"Le Fermentor" Monte Carlo, Monaco *Delaware River Bridge, NJ	Cut-off Wall, Barcelona, Spain	
Special Load and/or Movement Criteria	•El Habda Minaret, Iraq •Subway Tunnel, Milan, Italy		•Ponte Vecchio Florence, Italy				
Ease of Connection to Structure							

Notes:

(1) Case history is shown in box corresponding to the Primary reason for selecting micropiles.

(2) Case histories denoted with * are described in this volume. Others are referred to in summary elsewhere in this report.

against the effects of potentially aggressive groundwater. A 1:1 sand/cement grout (using sulfate-resisting cement) of w/c ratio 0.4 was placed prior to introduction of the reinforcement – a 140-mm-diameter group of six low-yield bars with helical reinforcement, typical of contemporary practice. The grout provided 28-day crushing strengths of greater than 45 MPa.

Numerous obstructions to the progress of the drilling were recorded, principally hard rock inclusions in the fill, ancient column and wall footings, and the remnants of lead-lined coffins. Most were overcome by predrilling with a 240-mm-diameter rock roller bit, thereby permitting the installation of the permanent liner and the progress of the subsequent augering in the clay. In other cases, where even this approach proved unsuccessful, piles were relocated slightly.

Testing. Due to the severely restricted working space and budget restraints, full-scale load tests were not allowed – a fact reflected in the conservative pile design parameters. However, as one level of construction quality assurance, each pile was subjected to ultrasonic testing (transient dynamic method) to confirm its integrity. As a result of the micropile installation, all total and differential movements were halted and this historic structure has been in full service since.

Coney Island, New York (Munfakh and Soliman, 1987)

Background. The Coney Island Main Repair Facility of the New York Transit Authority has been in operation for almost 80 years and is the largest of its kind in the world. It encompasses, including the rail yards, about 400,000 m^2, of which 50,000 m^2 are covered building space. Constructed on the former Coney Swamp, the repair shop was built on a loose fill surface with no pile support for the floors. The steel frame, columns, and outside walls were supported on piled foundations. Consolidation had produced major underfloor voids, which had led to many floor collapses, such as a 460-mm drop in the main shop in 1980.

During the original construction, the swamp filling had apparently created mud waves, resulting in uneven thicknesses of the soft organics underlying the structure. The subsequent settlement of the ground surface due to the loading by the fill and the structure had thus been irregular in magnitude across the site.

After "Years of Band-Aids," a $100 million repair program was initiated in 1984 that coincided with the installation of new equipment whose additional weight would also have accelerated the settlement problem. Foundation repair had to be carried out in a fashion guaranteeing minimal disruption to continuing shop operation, as well as constituting a proven, compatible, and cost-effective solution.

Remedial options under consideration included compaction grouting, chemical grouting, and concrete-filled steel shell piles. However, Type 1B micropiles proved to be the most appropriate solution from all viewpoints, and a contract was let in early 1987 to install more than 4200 piles.

Site and Ground Conditions. Four distinct soil layers were identified under the slabs: fill; peat with organic silt; gray sand; and brown sand. Short- and long-term consolidation testing confirmed that the organic layers were the cause of the settlement. These strata experienced long-term secondary consolidation and peat degradation, either from oxidation or micro-organisms. Typically, the medium dense, fine sands recognized as being adequate load-bearing materials commenced 3.0 to 7.6 m below the surface. The piezometric level was at about 1.2 m below the surface.

Access and headroom conditions were always restrictive and frequently obstructive, being as little as 2.4 m overhead. In addition, as the work was to be carried out in a busy, fully operational facility, in collaboration with other major structural repairs, it had to be executed in restricted "packages" in a piecemeal fashion.

Design. Approximately 2300 each 133-kN piles and 1900 each 267-kN piles were required. The engineer's design allowed for the load to be taken on 19-mm- and 28-mm-diameter bars, respectively, without the addition of permanent steel casing in the soft upper zones, wherein resistance to buckling was analyzed and judged to be adequate. Standard design procedures, based on ϕ = 30 degrees, were used to arrive at total lengths of 10.7 and 13.7 m for the 133-kN and 167-kN piles, respectively, that is, 3.0 or 6.0 m into the competent sand.

Construction. Before installing the piles, the existing underslab voids were filled with a lightweight, foamed concrete, selected to inhibit additional settlement and corresponding downdrag forces to the piles. The fill would also protect against erosion by blocking water flow through such voids.

The access and headroom restraints over much of the site demanded the use of specially constructed drilling equipment featuring short masts and remote power units. Whenever possible, conventional crawler-mounted units were employed, with care being taken in all cases with the disposal of exhaust fumes and drilling spoil.

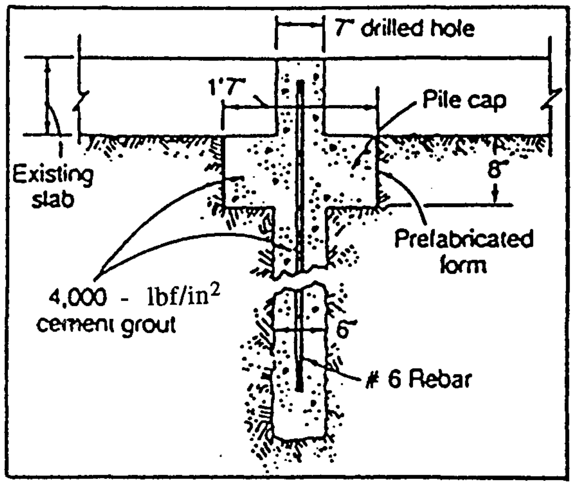
The 133-kN piles were drilled and cased to 168-mm nominal diameter and the 267-kN piles to 194-mm nominal diameter. Water flush was used. The drill casing was completely withdrawn during the pressure grouting of the sand using neat Type 1 with w/c = 0.50 to a maximum of 0.4 MPa, following the placing of the full-length reinforcing bar.

Load transfer to the existing slab structure was provided by an underreamed supporting zone formed under the slab (figure 2).

Performance and Testing. A program of 10 full-scale test piles was executed to verify assumptions regarding design and performance for the two pile types. Poly Vinyl Chloride (PVC) liners were provided as bond breakers – from the slab to the top of the sands – to ensure transfer of load only in the lower horizons.

In the first three compression tests, the load was applied directly to each pile. Munfakh and Soliman (1987) reported that the high concentration of stress crushed the top portion of each pile. The remaining test piles were given an enlarged cap, providing better load transfer to the grout and reinforcement.

Load tests were run to twice service load in compression, and to 445 kN in tension.



1 in = 25.4 mm
1 ft = 0.305 m
1 lbf/in^2 = 0.007 MPa

Figure 2. Schematic arrangement of micropile and existing base slab, Coney Island, New York (Munfakh and Soliman, 1987).

The steel casing was left in place in one pile (number A/8) so that a performance comparison with the standard pile number A/9 could be obtained (figure 3).

The first four piles (table 10) experienced significant creep at maximum load (8.9 mm in 4 hours), whereas those tested through the cap had less (0.81 to 1.6 mm in 4 hours at 267 kN). The cased pile had less than half this amount of creep in 5 hours at 267 kN.

Such performances were acceptable to the structural designers and the benefits of the cased pile were not required in the production piles subsequently installed. This repair was completed in the late 1980's and the facility has remained fully operational since then.

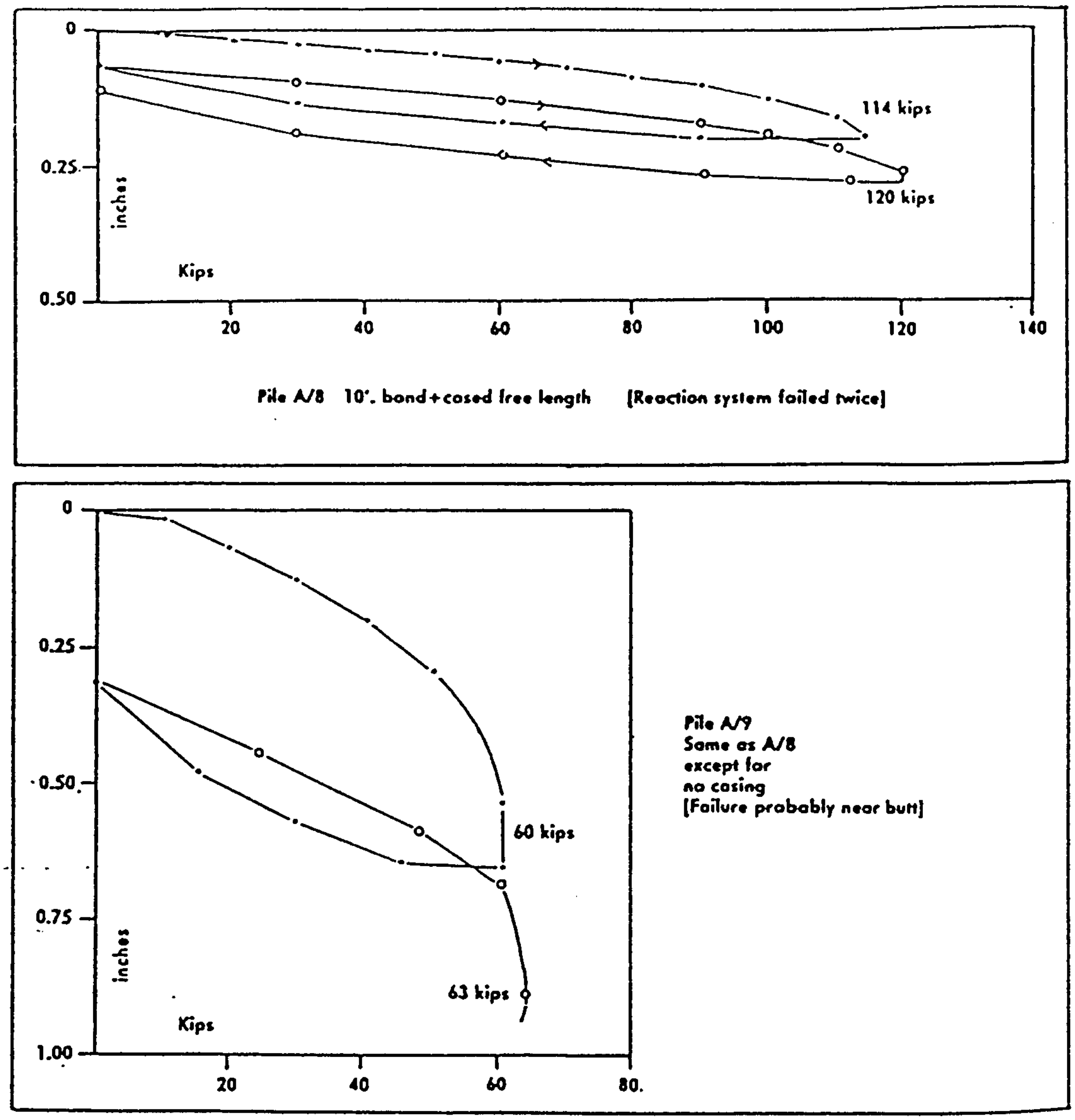
Repair/Replacement of Existing Foundations

<u>United Grain Terminal, Vancouver, Washington</u> (Groneck et al., 1993)

Background. The "A House" facility of this grain terminal includes a main elevator structure and three silos -- two built in 1934 and the third in 1939 (figure 4). These structures were built on a very fast track, which unfortunately precluded the use of treated wooden piling. The absence of preservatives led to progressive deterioration of the pile, so that by the early 1990's, the facility was faced with either replacing the underpinning or being forced to close down completely. Differential settlements had reached more than 130 mm on the 30-m-high silos, sending one of them 110 mm out of plumb. Considerable concrete cracking was found in the silo bodies.

Fourteen site investigation holes were drilled around the perimeter. Ten caissons sunk along the side of the structures, with short tunnels extending beneath the foundation mats, allowed visual inspection of the upper 3 m of the wooden piles below the foundation mat and above the water table. The piles were found to be damaged at all 10 inspection points, and in some cases, the tops of the piles were entirely missing. The only sign of their former presence was a round pocket cast in the bottom of the foundation mat. It was obvious that the foundation support for all three storage houses had to be replaced if the silos were to remain in service.

Several methods of underpinning were considered, including driven piling, drilled caissons, and jet grouting. Many difficult conditions had to be accounted for in planning the project, including:



1 in = 25.4 mm
1 kip = 4.45 kN

Figure 3. Comparative test performances of two 11-m-long micropiles, with and without permanent casing, Coney Island, New York (Bruce, 1988).

Table 10. Comparative performance of 133-kN service load piles, Coney Island, New York (Bruce, 1988, 1989).

Pile #	Description	Ratio of grout volume to hole volume	Stiffness in linear portion (kN/mm)	Maximum load (kN)	Total Displacement (mm)	Notes
A/3 A/4	Loaded annulus only	1.2 3.7	28 30	178 276	31.75 16.51	Failure premature and most probably due to crushing of pile head
A/4 A/9	Loaded full section	2.5 2.9	33 25	258 276	19.05 21.59	Failure possibly due to soil/grout failure although distress of head also noted
A/7	Includes original concrete slab in cap	2.9	106	623	22.86	Soil-grout failure likely
A/10	Excludes concrete slab in cap	3.4	62	498	10.67	Test suspended upon failure of pile cap
A/8	With sacrificial casing for 7.5 m	7.7	135	534	7.62	Test suspended when reaction pile pulled

Notes:

(1) All piles were 175 mm in diameter, 11 m long (including 3-m bond), and had a full-length 19-mm-diameter rebar.

(2) Pile stiffness is calculated by dividing the maximum load over which displacement is linear by the displacement at that load.

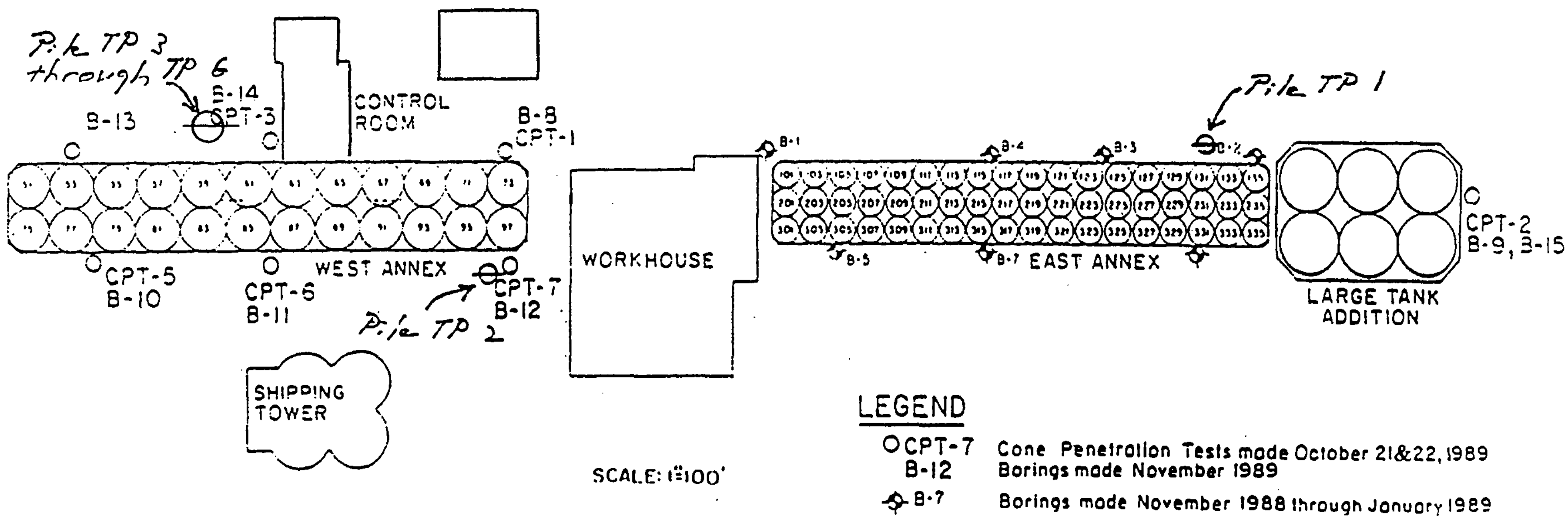


Figure 4. Plan of part of the United Grain facility, Vancouver, Washington, showing location of test piles (TP1-TP6) and site investigation holes.

- The operation of the silos could not be interrupted. This precluded removing conveyor belts to provide interior access, or disrupting the almost continuous railcar unloading activity occurring along one side of the facility.

- Access was also extremely limited for work in the basement of the silo structures.

- The explosive nature of the grain dust prohibited use of combustion engines and any metal welding, cutting, or grinding activities in the basements.

- Extending the underpinning elements between the existing timber piles – which were very competent below the top of the water table – down to the competent bearing soils would be difficult.

- There was a potential for overstressing of the minimally reinforced concrete walls and slabs resulting from the concentration of the foundation support around the columns and walls.

Considering these difficulties and wishing to take advantage of qualified contractors' experience and techniques, the owner pursued a design-build proposal. The final request for proposals envisioned jet-grouted columns or friction piles as alternatives. All the systems were to be embedded in the dense sands and gravels. Load factors and maximum movement requirements were specified, along with the need for thorough pre-production and production load test programs. Major emphasis was placed on the contractors having to establish their experience and qualifications. A complete set of design plans and calculations had to be submitted with each proposal.

The micropile solution proved cheaper. However, there were other advantages over jet grouting, including the fact that each micropile's capacity was independent of the upper layers of soft soils and of the timber piles, whereas jet grout strength and column regularity would have been negatively impacted there. The piles were designed as Type 1B elements.

Site and Ground Conditions. The 4050 timber piles had been driven on a 760-mm grid pattern through the upper soft dredged sand and silt layers and were founded in dense sand and cobbly gravel. The timber piling supports 610- to 760-mm-thick reinforced concrete mats, which form the silo basement slabs. Hollow concrete box columns rest on the mat slabs, supporting the 30-m-tall silos. Micropiles had to be installed around the outside perimeter of the silo structures, and inside and around the interior concrete box columns. Doorways (0.9 m x 2.1 m) were cut through the concrete walls of the box columns to create access corridors into the interior of the box columns and through the basement areas. Interior headroom ranged from 2.5 to 3.6 m, with work room inside the columns being as little as 1.5 m x 2.1 m.

Design. The structural design called for 840 vertical micropiles, each with a nominal service load of 1330 kN. They were approximately 21 m long, thus providing at least 9 m of bond in the very dense gravels and cobbles. The specification called for the underpinning system to ensure additional differential settlements of less than 38 mm in 30 m, and additional uniform total settlements of less than 150 mm. For the two silos built in 1934, which measure 97 m x 17 m and 91 m x 16 m, 336 and 324 piles, respectively, were foreseen. The 1939 structure (42 m x 23 m) required 180 piles.

Construction. The interior piles were installed through 250-mm-diameter holes cored through the basement slab. Specially fabricated, skid-mounted rigs were used in the cramped conditions, with the diesel-hydraulic power units placed outside to avoid being a source of ignition for the grain dust. The lengths of 178-mm-diameter, 13-mm-thick casing, and the 57-mm-diameter reinforcing bars were coupled mechanically, eliminating any need for welding. These piles were connected to the mat slabs by grouting the upper casing (equipped with shear connectors) into the roughened-up core holes with a high-strength proprietary mortar. Reinforcing bars doweled into the column walls and a 600-mm-thick concrete mat poured on top of the basement slab helped to distribute the support of concentrated pile groups inside the box columns and to reduce the induced stresses on the floor slabs and column walls. Larger track-mounted drill rigs installed exterior piles around the perimeter of the silos. Concrete pile caps doweled to the exterior basement walls and the edge of the basement slab connected the piles to the structure. Where access was difficult, such as between the silo structures, the small indoor rigs were also used for exterior pile installation, with excavation and backfill done with a backhoe or by hand. In both sets of piles, neat cement grout with a w/c of 0.45 was used at injection pressures of up to 0.8 MPa. Rotary drilling with water flush was used throughout, for a total pile length of almost 17,000 linear meters. This included almost 1000 linear meters of drilling through wood piles.

Testing and Performance. The pre-production test program required the successful loading of three test piles to 200 percent service load (2660 kN), held for a minimum of 12 hours. Although each pile reached and held this value for a short period (3 to 45 minutes), sudden load losses, indicative of internal material failure, were recorded. A second group of three piles was then installed with appropriate modifications -- the most significant being an increase in the bond zone reinforcement size from 44 to 57 mm in diameter. Each passed and was subsequently tested to explosive failure at loads of up to 3360 kN. Data are provided in tables 11 and 12. Close analyses (Bruce et al., 1993), based on the elastic ratio method (volume II), seemed to suggest that at the service load of 1330 kN, barely 9 m of the pile (from ground level) appeared to have debonded, and was acting elastically.

To provide additional verification of the micropile capacity, 3 percent of the production piles (a total of 25, later reduced to 10) were tested to 200 percent of service load. The final calculated service loads varied from 1120 to 1300 kN, depending on pile location. Deflection of the pile tops at these loads averaged approximately 10 mm, with an average deflection at twice service load of 22 mm. In addition, two mockups of the structural connection between the pile and foundation slab were load tested in tension to 2660 kN without failure.

Soil strata depth encountered, pile depth, grout quality, and grout pressures attained were monitored and recorded for each pile installation to ensure quality. If a structural defect was found in a pile during installation, the pile was load tested to a minimum of its service load. Only two of these tests were required; one pile failed to hold the test load and was replaced.

Table 11. Soil strata thicknesses encountered, United Grain Terminal, Vancouver, Washington (Bruce et al., 1993).

Test Pile	Upper Length Soil Strata			Bond-Length Soil Strata		
	Sand Fill (m)	Silt (m)	Medium Dense Sand (m)	Medium Dense Sand (m)	Very Dense Sand (m)	Very Dense Gravels (m)
TP-1	6.6	4.8	1.5	3.9	3.6	1.5
TP-2	6.1	4.6	1.5	2.1	2.7	4.3
TP-3	3.0	6.4	2.4	1.2	4.3	3.6
TP-4	3.0	6.4	1.8	1.8	4.3	1.5
TP-5	3.0	6.4	1.8	1.8	4.3	1.5
TP-6	3.0	6.4	1.8	1.8	4.3	1.5

Table 12. Pile installation details, United Grain Terminal, Vancouver, Washington (Bruce et al., 1993).

Test Pile	Installation Order	Total Pile Depth (m)	Bond Length (m)	Casing Insertion into Bond Zone (m)	Casing Length from Grade (m)	Rebar Diameter (mm)	Rebar Length (m)
TP-1	2	22.3	9.1	1.5	14.6	44	9.1
TP-2	1	21.3	9.1	1.5	13.7	44	9.1
TP-3	3	21.0	9.1	3.0	14.9	44	9.8
TP-4	4	18.9	7.6	3.0	14.3	57	8.2
TP-5	5	18.9	7.6	3.0	14.3	57	8.2
TP-6	6	18.9	7.6	3.0	14.3	57	8.2

Operation of the silos continued uninterrupted during installation of the underpinning except for the allowed maximum of 500,000 bushels of empty storage capacity. That was kept empty where connections between the piles and the silos were being completed to allow pile connection to the structure without the elastic deflections generated by the grain load.

The weight of the grain was a considerable portion of the pile design load, making up approximately two-thirds of the total weight of the silos. The magnitude of the grain weight became particularly evident during production testing of the piles: the silos above the test pile had to be loaded with grain to avoid raising and cracking the structure. Large steel beams provided additional distribution of the test loads to adjacent box columns.

An automated movement-monitoring system continuously checked any movement of the silos, with secondary measurements provided by weekly surveys of the silo perimeter. No significant additional movements occurred during the construction period. Monitoring has continued to verify performance of the system within the specified criteria.

The underpinning was completed on January 29, 1993. Two months later, the area experienced an earthquake with a magnitude of 5.6 on the Richter scale. Its epicenter was approximately 60 km distant. Observations taken after the event revealed new movement at several points. This movement, which was no greater than that expected from full load transfer from timber piles to micropiles, caused no damage to the structures. Given the questionable condition of the timber piles, it is likely that serious structural damage would have occurred had the micropiles not been in place during the earthquake.

Pocomoke River Bridge, Maryland (Bruce et al., 1990)

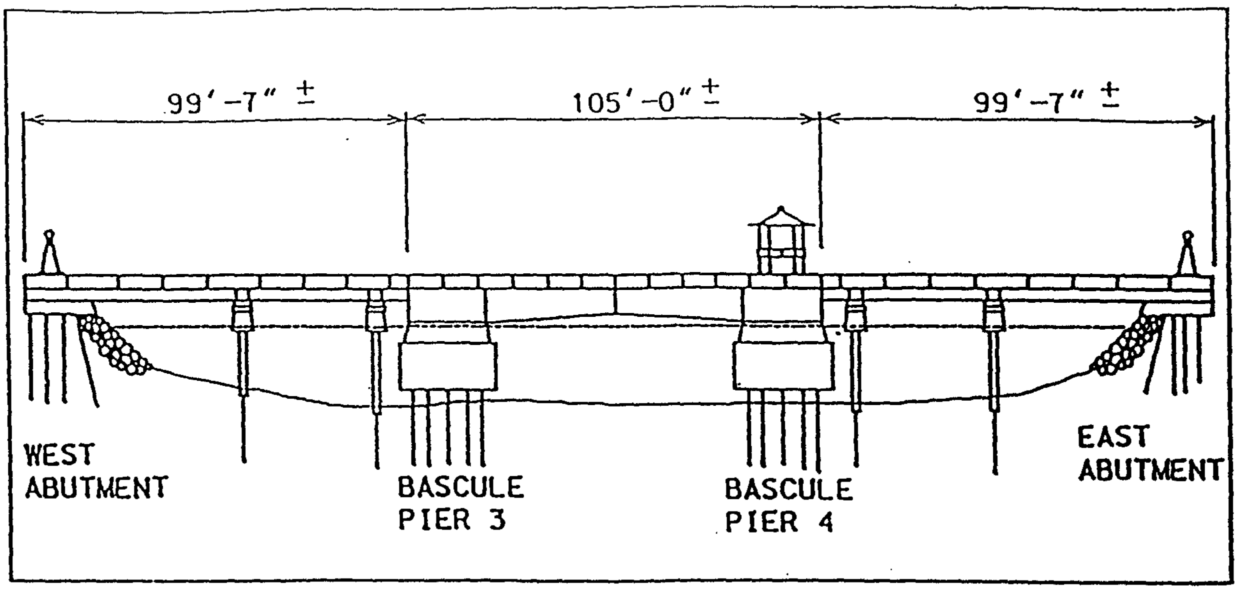
Background. This long movable bascule pier drawbridge (figure 5) was built over the Pocomoke River in 1921. Bascule Piers 3 and 4 were originally supported on wooden piles driven through the soft riverbed muds into the underlying compact sand. The support offered by these piles had been compromised by river scour that had exposed them in several places. The Type 1B micropiles designed to stabilize the structure were remarkable on three counts:

1. They had to be installed through the structure and through the scour zone.

2. They had to provide support without allowing any additional structural settlement. This necessitated the use of preloading techniques.

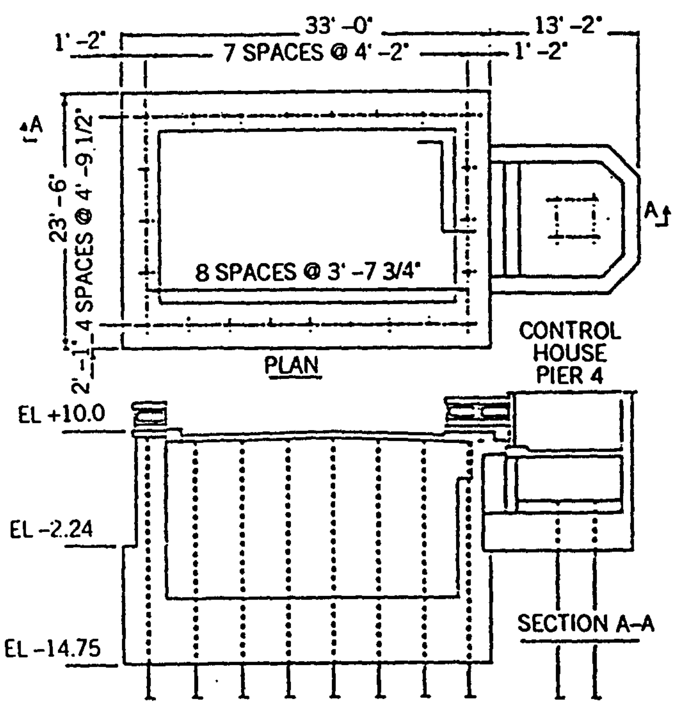
3. An intensive test program was required on special test piles to verify the concept of the preloading, in particular, and the performance of the pile system in general.

Site and Ground Conditions. In each of Piers 3 and 4, a total of 24 piles had to be drilled from the bridge deck. In addition, four more piles were installed from the restricted access of the Control House of Pier 4 -- 2.4 x 2.4 m in plan with 4.3 m of headroom (figure 6).



1 in = 25.4 mm
1 ft = 0.305 m

Figure 5. General configuration of Pocomoke River Bridge, Maryland (Bruce et al., 1990).



1 in = 25.4 mm
1 ft = 0.305 m

Figure 6. Plan and section of Bascule Pier 4, Pocomoke River Bridge, Maryland, showing micropile locations (Bruce et al., 1990).

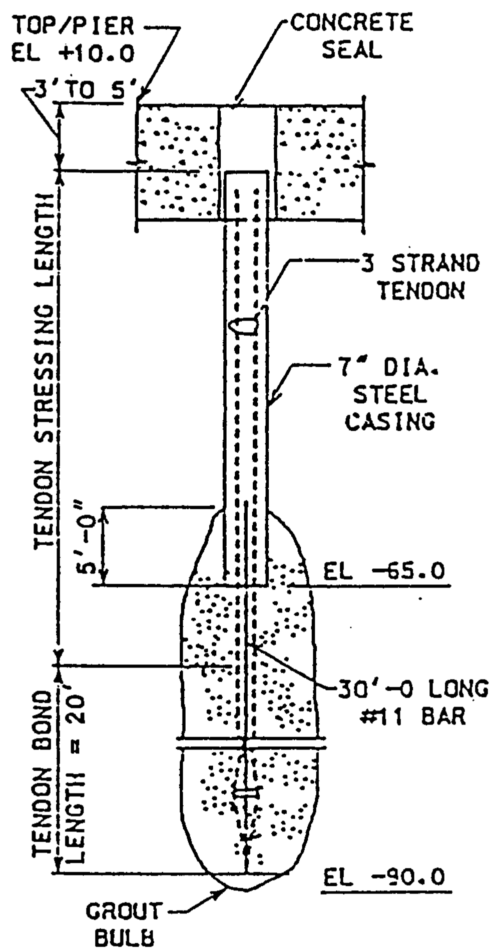
The riverbed materials into which the underpinning was installed were comprised of soft alluvial sediments. The founding horizon was dense medium to coarse sand (N = 25 to 30), beginning about 18 m below river surface level.

Design. To meet the 365-kN service load, each pile comprised a 178-mm-diameter, N80 casing, 13 mm thick, installed to the top of the sand (figure 7). A 36-mm-diameter Grade 60, epoxy-coated reinforcing bar was used in the bond zone from the bottom of the pile to 1.5 m above the bottom of the casing. The 28-MPa grout was also permitted to carry load. The allowable stresses used in the design were 30 percent of the grout crushing strength, plus 40 percent of the yield strength of both the casing and the epoxy-coated rebar. To permit preloading of the pile, a tendon comprising three 15-mm-diameter, seven-wire strands was also installed through each hole, the 6-m bond zone extending to 7.6 m below the toe of the pile casing. The piles were designed for a maximum allowable soil/grout bond value of 0.21 MPa along a bond zone assumed to be 230 mm in diameter by 6 m long in sand. The actual ultimate bond stress subsequently proved to be about 0.4 MPa.

Construction. The micropiles were installed as shown in figure 7 with a diesel-hydraulic, crawler-mounted rig. Holes measuring 222 mm in diameter were predrilled through the pier concrete and infill grout, the latter having been placed previously to fill existing voids below the pier. Rotary drilling methods with air flush were used. The 178-mm-diameter steel casing was then advanced full depth using water flush. Neat cement grout with a water-cement ratio of 0.45 was placed by tremie, followed by the rebar and the preloading strands. Pressure grouting, at pressures up to 0.7 MPa, was carried out during simultaneous extraction of the casing over a length of 9.1 m. The casing was then reinserted 1.5 m into this pressure-grouted zone, leaving a completed bond zone length of 7.6 m.

After the grout had reached a crushing strength of 24 MPa, the tendon was stressed against the top of the steel casing, to the service load of 365 kN. The annulus between casing and structure was then grouted with special high-strength grout. Between 5 and 7 days later, the prestress was released at the tendon head, thereby allowing full structural load transfer to the pile, but without obviously causing further pile compression. Slightly amended procedures had to be adopted in the restricted access of the Control House, but the same basic principles were followed. A total of 52 micropiles were installed.

Testing and Performance. Two special pre-production test piles were installed on the adjacent west bank for intensive testing, 44 m north and 17 m west of the west abutment. Each pile had a 9.1-m-long outer casing of 219 mm in diameter, predrilled from the surface. The 178-mm-diameter casing of the pile was then installed in standard fashion through this large casing, but without being bonded to it in any way. This arrangement was intended to simulate in the test the lack of lateral resistance afforded by the river and the very soft soils on its bed, as well as the portion of the pile that was within the confines of the bascule pier. Each of the identical piles had 7.6 m of pressure-grouted bond zone, 9.1 m of 36-mm-diameter rebar, and 21 m of 178-mm-diameter casing from surface to 1.5 m into the bond zone. Soil anchors provided reaction to the test loads.



1 in = 25.4 mm
1 ft = 0.305 m

Figure 7. Typical detail of micropile, Pocomoke River Bridge, Maryland (Bruce et al., 1990).

The test had three phases:

Phase I: "A preload-unloading" test designed to verify the performance efficiency of the preloading system.

Phase II: A conventional pile load test to establish load-movement performance within the scope of the specification (progressively to twice working load).

Phase III: One pile, loaded to failure.

The test was heavily instrumented, with load being measured independently by load cell and by hydraulic jack gauge, and movement monitored by dial gauges supported from an independent reference beam, and by piano wire and mirror scale. Dial gauges were also used to indicate movements of telltales located at elevation -21 m (top of bond length) and at elevation -27 m (bottom of bond length).

PHASE I TESTS (preloading-unloading). The anchor tendon in each pile was loaded to 365 kN, creating elastic shortening of 3.12 and 3.50 mm, respectively. Upon unloading to zero (releasing the prestressing load), the pile head rebounded totally elastically, indicating no measurable permanent shortening. As the procedure was demonstrated to work, and since the performance was totally elastic, this phase of testing was accepted as being successful.

PHASE II TESTS (load/deflection test to twice design working load). Each pile was loaded progressively to 890 kN in 89-kN intervals, each with a 5-minute hold period. Details are summarized in table 13. Major observations were:

- The performance of each pile was very similar, being virtually elastic, linear, and with minimal creep at intermediate holds.

- The total pile movements (anticipated and observed) at 890 kN were less than 13 mm, and the permanent movements upon unloading were around 1 mm some 2 hours after final load release. After a further 12 hours, the piles had returned to full extension (i.e., there was no measurable permanent shortening).

- The performance of the telltales was wholly consistent. This reflected the internal elastic performance of the piles, and so provided movements less than the total pile displacement (i.e., elastic plus permanent). Predictably, the upper telltale, monitoring a shorter length, provided the smaller movements. These data compare closely with the net elastic deflection obtained by subtracting total head movement at 890 kN from the residual (at zero), as shown in table 14.

- Total creep at 890 kN ranged from 1 to 1.5 mm over 24 hours. However, the amount of "internal" creep was smaller and more uniform (0.53 to 0.84 mm, average = 0.69 mm).

Table 13. Highlights of load/movement data, test piles 1 and 2, Pocomoke River Bridge, Maryland (Bruce et al., 1990).

	Movement at 890 kN (mm)	Creep in 24 h at 890 kN (mm)	Permanent Movement upon Unloading from 890 kN Instantaneous/After 2 h (mm)
Pile 1			
Pile cap	11.23	0.97	1.12/0.51
Upper telltale	8.74	0.71	0.61/0.53
Lower telltale	9.50	0.79	2.41/2.36
Pile 2			
Pile cap	11.10	1.50	1.19/0.69
Upper telltale	9.78	0.84	1.04/0.94
Lower telltale	10.67	0.53	1.70/1.60

Table 14. Comparison of net and measured elastic pile performance, Pocomoke River Bridge, Maryland (Bruce et al., 1990).

Pile Number	Net Elastic Movement [a] at 890 kN (mm)	Measured Elastic Movement [b] at 890 kN (mm)
1	10.11	9.50
2	9.91	10.67
Average	10.01	10.08

[a]Total movement at 890 kN *less* permanent movement at subsequent zero.
[b]From lower telltale.

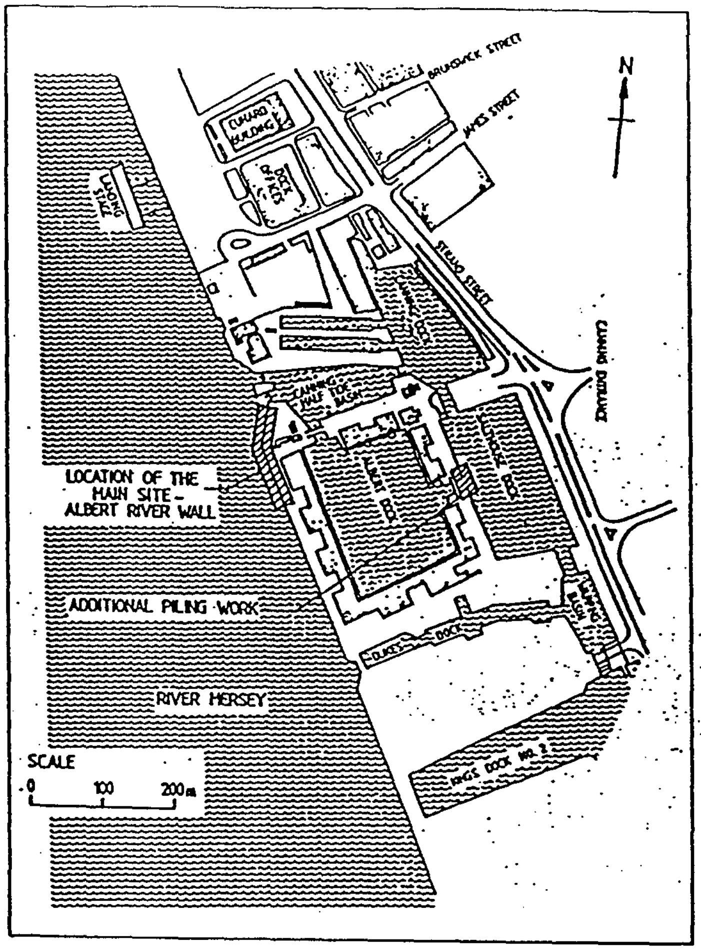


Figure 8. Location of Albert River Wall at Albert Dock Village, Liverpool, England (Turner and Wilson, 1990).

- There was a time-related "rebound" evident in all points of measurement after unloading. Overall, this was 0.51 to 0.69 mm at the pile cap, including 0.05 to 0.10 mm of "internal" pile rebound.

PHASE III TESTS (load test to failure, test pile 2). Once the required tests to 890 kN were satisfied, an attempt was made to determine the ultimate skin friction by testing one pile to failure. The testing system was sized for about 1600 kN, which was initially felt to be sufficient to fail the pile. Surprisingly, after four successive cycles to about 1600 kN, the pile had not yet failed, despite a cumulative permanent movement of 14.4 mm. Thereafter, the test setup was overhauled and the test was rerun: a maximum load of 1730 kN was reached before pile failure was recorded.

Again, the evidence of the telltales indicated virtually perfect elastic performance within the pile. The difference, at maximum load, between overall elastic performance (lower telltale) and total movement was 44 mm, very close to the measured permanent set at zero load of 43 mm. The difference is probably due to the fact that the telltale was not exactly at the pile tip. Creep values were only significant from loads of about 1510 kN upwards.

Albert Docks, Liverpool, England (Turner and Wilson, 1990)

Background. The Canning and Albert Docks, which form part of the Liverpool South Docks complex, had not been used since the early 1970's, until the original dock buildings and structures were restored as part of the Albert Dock Village scheme (figure 8). Part of the restoration involved the stabilization of a 135-m-long section of the Albert River Wall, which separates the Albert Dock complex from the Mersey River. The crest of this 12-m-high retaining wall was rotating towards the river, relative to the toe, as was evidenced by longitudinal cracking along the ground surface behind the wall, and by substantial near-vertical cracks that had formed at intervals along the wall. Preliminary investigations could not determine whether the cause of the movement was due to a gradual failure of the original timber pile foundations, or to increased surcharge loads during the life of the wall.

The existing river wall was constructed of sandstone blocks with granite facings and occasional timber insets. It had originally been founded on a timber grillage, supported by timber piles driven to the underlying sandstone. Given the geological and structural restraints, it was decided to stabilize the wall with micropiles, to be drilled at near-vertical angles, through the existing river wall and accompanying fill material, into the underlying river alluvium and boulder clay, and finally into the underlying sandstone bedrock. More than 550 Type 1A micropiles were designed with service loads ranging between 490 and 530 kN. Because of the various design conditions, the piles were designed to withstand both tensile and compressive loads.

Site and Ground Conditions. The founding stratum was medium-hard Bunter Sandstone of Permo-Triassic age, located at depths up to 7 m below the river wall. This was overlain generally by soft alluvial silt and clay, and dense sand and gravel or boulder clay. The backfill behind the river wall was formed from the material excavated for the original docks and consisted of weathered sandstone, clays, sands, and gravels. In places, the fill contained numerous obstructions in the form of timber, iron, and granite blocks.

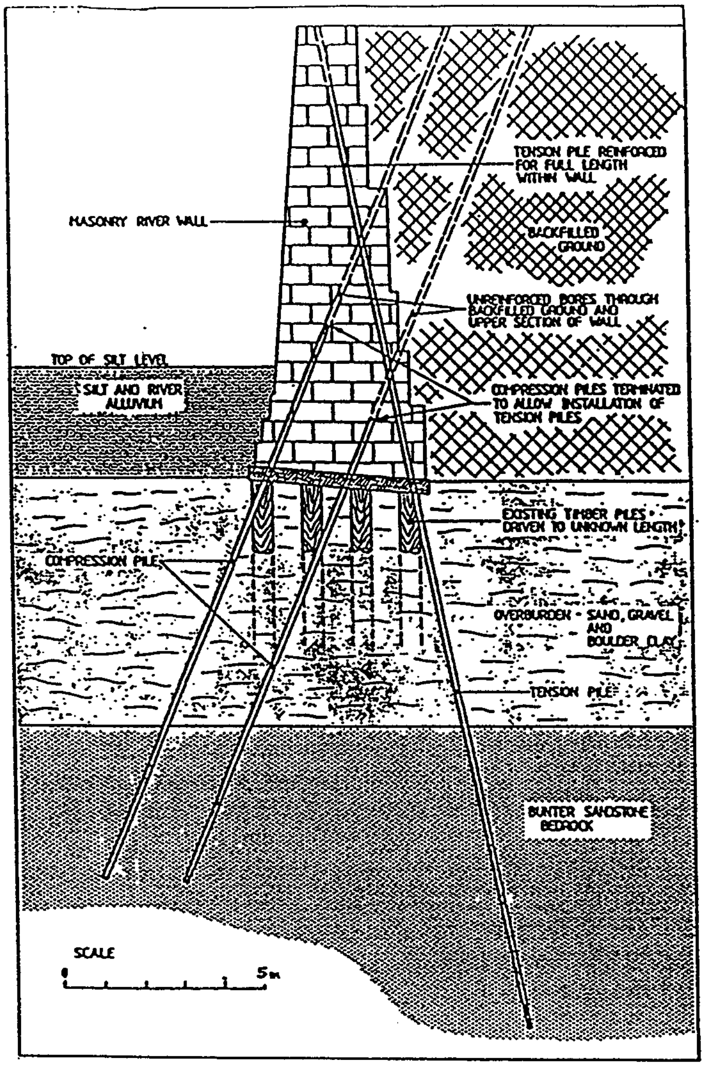


Figure 9. Typical cross section through piled wall, Albert Docks, Liverpool, England (Turner and Wilson, 1990).

Design. The micropile scheme had to resist not only the vertical loads imposed by the wall, but also overturning forces imposed by active pressures behind the wall exerted by the retained soil, porewater pressures, and surcharge loads. In addition, no support contribution could be assumed from the existing pile foundations, since their condition could not be assessed.

In determining the pile spacing, a number of general design cases were considered. The first case constituted the simultaneous action of all design vertical and horizontal loads, including uplift loads due to water pressure (full drawdown case). The second case allowed for the situation where only design vertical and horizontal loads were acting (i.e., zero uplift). In the third case, only vertical dead load was considered to act. The stabilization system had to be able to resist the worst combination of these three cases.

The cross section through the Albert Dock river wall shown in figure 9 illustrates the pile layout chosen. The underpinning configuration consisted of a main row of compression micropiles, acting approximately through the wall centroid, and two rows of tension and compression micropiles on either side of, and approximately equidistant from, the central compression pile row. The pile rows were inclined so as to provide horizontal components of resistance to sliding forces at the base of the wall. In both rows of compression piles, the steel reinforcements were terminated at a predetermined point within the masonry wall, to allow for later installation of the tension piles. The reinforcement in the tension piles was continued to within 2 m of the upper wall surface to act as additional reinforcement for the rear face. Towards the ends of the wall, a transition zone was established over which the pile spacing was progressively increased to provide a gradual transition between the remediated wall section and the adjacent untouched sections.

Each compression micropile was constructed using a 50-mm-diameter GEWI bar as its main load-carrying member. The tension piles were constructed of threaded, deformed, high-yield McCalls steel reinforcing bar (again, 50 mm in diameter), chosen because it had a slightly higher tensile strength than the GEWI bar. Figure 10 illustrates the configuration of a typical micropile as installed at the site.

The piles were installed with drilled diameters varying between 112 and 146 mm. The bond lengths for these piles were designed as for prestressed rock anchors (Littlejohn and Bruce, 1977). For calculation purposes, an average value of 0.5 MPa was used as the working bond stress between the micropile grout and the sandstone. It was thus determined that adequate load transfer would be achieved with a minimum penetration into the sandstone of 4.5 m for the compression piles and 8 m for the tension piles. The longer bond length for the tension piles was prescribed to allow a sufficient weight of rock to be conceptually engaged to provide the required overall uplift capacity.

Because of the proximity of the piled structure to the tidal effects of the Mersey River, additional corrosion protection was warranted. The steel bar was sheathed with corrugated polyethylene tubing and filled with grout during installation. This protective sheathing was terminated at the top of the sandstone layer. The nominal grout cover to the reinforcement within the sandstone was 31 to 48 mm for the compression piles and 48 mm for the tension piles.

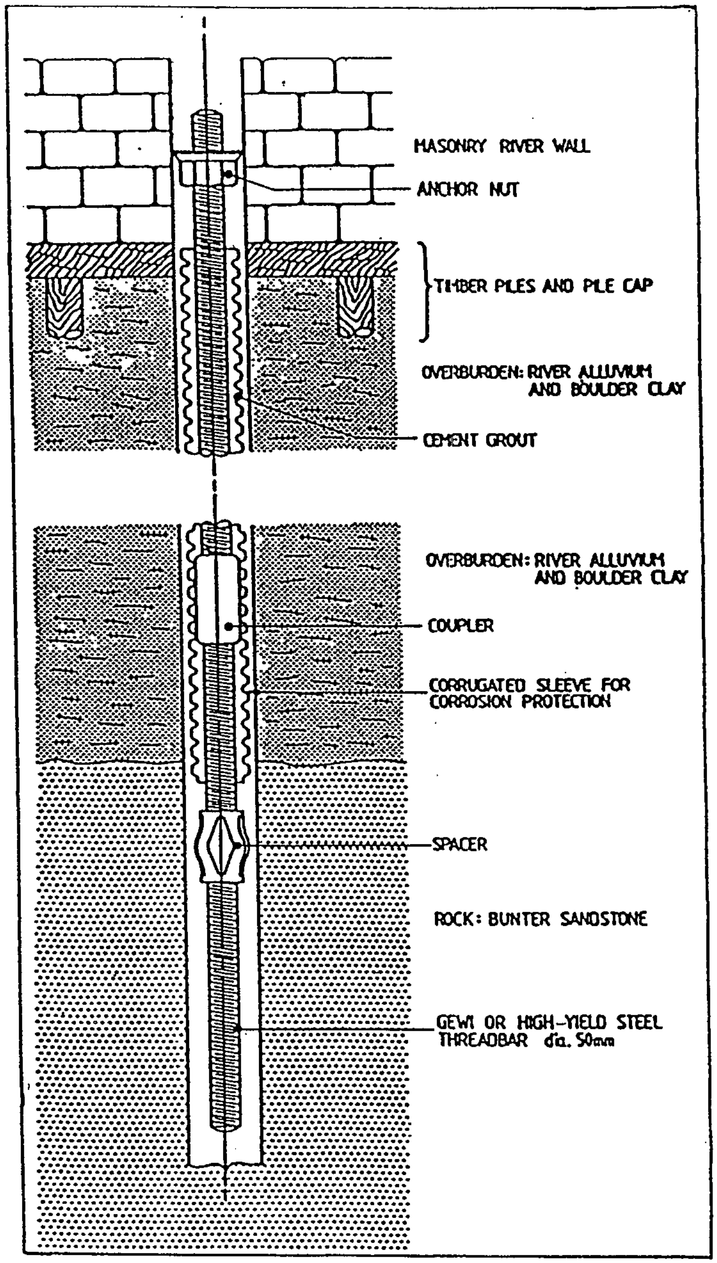


Figure 10. Typical micropile at Albert Docks, Liverpool, England (Turner and Wilson, 1990).

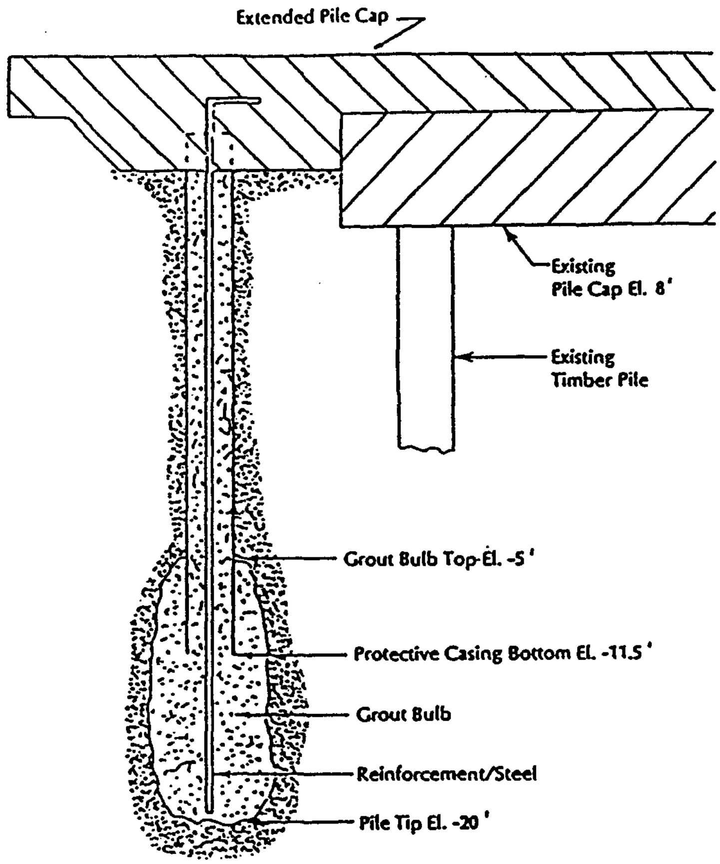
All piles were fully grouted into the masonry of the river wall to allow positive load transfer from the pile reinforcement into the structure. Again, a working bond stress of 0.5 MPa between the micropile grout and the masonry wall was adopted.

Construction. Drilling was carried out by diesel-hydraulic, track-mounted rigs. The two rows of compression piles on the seaward side generally were constructed first so that their reinforcing steel could be terminated clear of the line of the tension piles. The holes through the river wall were drilled using down-the-hole hammers. The holes through the fill were drilled with rotary duplex and water flush. A temporary drill casing was installed through the structure, the underlying timber supports, and overburden directly beneath the wall, and socketed into the underlying sandstone. Drilling in bedrock was accomplished by rotary open-hole techniques.

Once the pile had been drilled to depth, the hole was partly filled with grout and the reinforcing steel was lowered into place with a crane. The temporary casing was then removed and additional grout was added to top off the hole. The variable composition of the fill and the open structure of sections of the river wall resulted in considerably higher grout takes than expected.

The Albert Dock area had a history of extensive construction works, both in connection with the original building in the mid-1800's, and at subsequent times when repairs and alterations were undertaken. It is not surprising that occasional unexpected problems arose during the micropiling works. The problem that aroused the most interest was discovered at the northern end of the site, adjacent to the entrance to the dock complex, where a line of cast-iron sheet piles was encountered when drilling below the toe of the river wall. It was believed that these sheet piles were specifically set for the construction of the dock complex and were driven into the original riverbed to form a temporary cofferdam to allow the construction of the foundation of the river wall in dry conditions. Penetration of this obstacle at a steep, glancing angle presented difficulties, eventually overcome by coring through the cast iron with tungsten-tipped drilling shoes. In addition, within the river wall itself, an extensive culvert system had been built to flush away accumulations of silt around the entrance to the dock complex. This culvert system, with associated granite copestones and timber/iron sluice grates buried within the masonry structure, had to be penetrated by many of the pile holes. This alternation of material hardness, coupled with the silt-filled voids of the culverts and sluices, necessitated a repeated changing of drilling techniques.

As drilling and pile installation progressed, it was found that the most sensitive part of the wall, where the greatest magnitude of previous movement had originally been detected, was again moving out towards the river. This was probably caused by a temporary buildup of drilling water and the general disturbance created by the installation process. At one point, the wall moved more than 50 mm in a single day. By careful programming of the construction activities over this sensitive portion and by adopting additional preventative measures, such as the installation of drainage holes within the wall to drain any excess water, the piles in this section were carefully installed and wall movements were slowed and then halted.



1 ft = 0.305 m

Figure 11. General arrangement of micropiles, Boylston Street, Boston, Massachusetts (Bruce, 1988).

The piles were installed over a total contract period of about 6 months. On completion, the pile holes through the granite coping of the wall were completely filled with grout up to ground surface, and the site was refurbished and repaved to provide a new riverside walk alongside the Albert Dock complex.

Performance. The performance of the wall has been monitored since completion by visual examinations of the wall at regular intervals. No problems or irregularities have been recorded.

Upgrading Foundation Capacity

Boylston Street, Boston, Massachusetts (Bruce, 1988)

Background. The properties at 739-749 Boylston Street in the Back Bay area of Boston were completed in the "Chicago style" in 1906. These derelict commercial buildings, six and three stories high, respectively, were acquired for redeveloping and refurbishing: the former, for example, to provide retail space on the basement and first floors, with office space and a mechanical penthouse level above.

The structure was founded originally on pile caps bearing on timber piles. To accommodate the increased loadings from the new construction, additional support was required under enlarged pile caps (figure 11). The engineer foresaw piles of service loads of 178 kN (compression) and 53 kN (tension), but accepted the contractor's alternative design offering micropiles with service loads of 356 kN and 107 kN, respectively.

Site and Ground Conditions. Piling had to be executed from within the partially demolished basement of the structure (approximately floor elevation +2.4 m) about 3.0 m below existing sidewalk elevation, giving a minimum working headroom of 2.5 m. Access was awkward and restricted, and the position of several piles had to be adjusted slightly to accommodate particular site conditions.

The fill consisted of saturated, loose, gray-brown, fine sand and silt, and overlaid soft gray organic silt with traces of shells, sand, and gravel. The founding layer occurred at about -1.2 m and was 5.5 to 7.3 m thick throughout the site. It comprised medium dense to dense fine medium sand with a trace of silt. Pile lengths were maintained within this horizon so as not to perforate the underlying Boston Blue Clay.

Design. These Type 1B piles were designed assuming an ultimate load 2.3 times service load, namely 818 kN in compression, and 240 kN in tension. The length of the bond zone was designed on the basis of analogous soil anchor experience and assumed $\phi = 35$ degrees for the sand, an effective bulb diameter of 190 mm, and a grout pressure of 0.4 MPa.

Ultimate grout/soil bond (τ_{ult}) was estimated empirically from the relationship:

$$\tau_{ult} = \text{grout pressure} \times \tan \phi$$

$$= 0.4 \times 0.7 = 0.28 \text{ MPa}$$

The required load transfer length (L) was calculated from

$$L = \frac{\text{Ultimate load}}{\pi \cdot d \cdot \tau_{ult}}$$

where d is the bond zone diameter.

Thus, for the maximum load of 818 kN

$$L = \frac{818 \times 1000}{\pi \times 191 \times 0.28} = 4724 \text{ mm}$$

Further routine calculations using the provisions of the Commonwealth of Massachusetts Building Code (1984) demonstrated that:

- The use of 140-mm-diameter casing of 9.2-mm wall thickness and f_y (yield strength of steel) = 379 MPa as the major load-bearing element was safe. (Allowable stress $\leq$ 35 percent f_y.)
- The anticipated pile head movement at working load was acceptable.
- The compressive stresses imposed on the grout of the bond zone were acceptable. (Allowable stress $\leq$ 33 percent f_c (unconfined compressive strength of concrete/grout)).
- A single 25-mm-diameter rebar (f_y = 413 MPa) would adequately sustain the load. (Allowable stress $\leq$ 50 percent f_y.)

The individual piles were as shown in figure 11 and were arranged as in figure 12.

Construction. A diesel-hydraulic track rig was used to install all 260 piles. The 140-mm-diameter casing was first water flushed to about 2.4 m below the surface, before being pushed for a short distance to locate accurately the top of the dense bearing stratum. Rotary drilling then resumed in the sand to full depth. Neat Type I grout with a water-cement ratio of 0.5 was placed by tremie, followed by the rebar. Pressure grouting of the sand was carried out to a maximum pressure of 0.4 MPa during extraction of the casing, for the 4.6- to 4.9-m-long bond zone. The casing was then pushed back down about 1.5 m into this pressure-grouted zone and left in place.

Grout takes generally ranged from 2.5 to 3.5 times nominal hole volume, indicating that the enhanced effective diameter of the bond zone had been achieved. Grout cubes at 14 days gave unconfined compressive strengths greater than 40 MPa.

During drilling, wood piles and granite blocks in the fill were occasionally encountered, but were accommodated by hole relocation or simply by perseverance. Overall, 4 piles had to be replaced due to constructional problems, while the construction of an additional 2 piles lifted the total installed to 262.

Testing and Performance. Prior to the production piling program, compressive and tensile load tests on two typical piles were conducted. Each pile was constructed as described above, except for the addition of a telltale anchored near the toe and the placing of an outer steel liner around the 140-mm casing above the bond zone to prevent any load transfer in the upper soils. Reaction for each test pile was provided by adjacent ground anchors, and the tests were executed in accordance with the Massachusetts State Building Code and ASTM D1143. The data are summarized in table 15, while the performance of test pile 2 (in compression) is shown in figure 13 together with that of a timber pile for comparison.

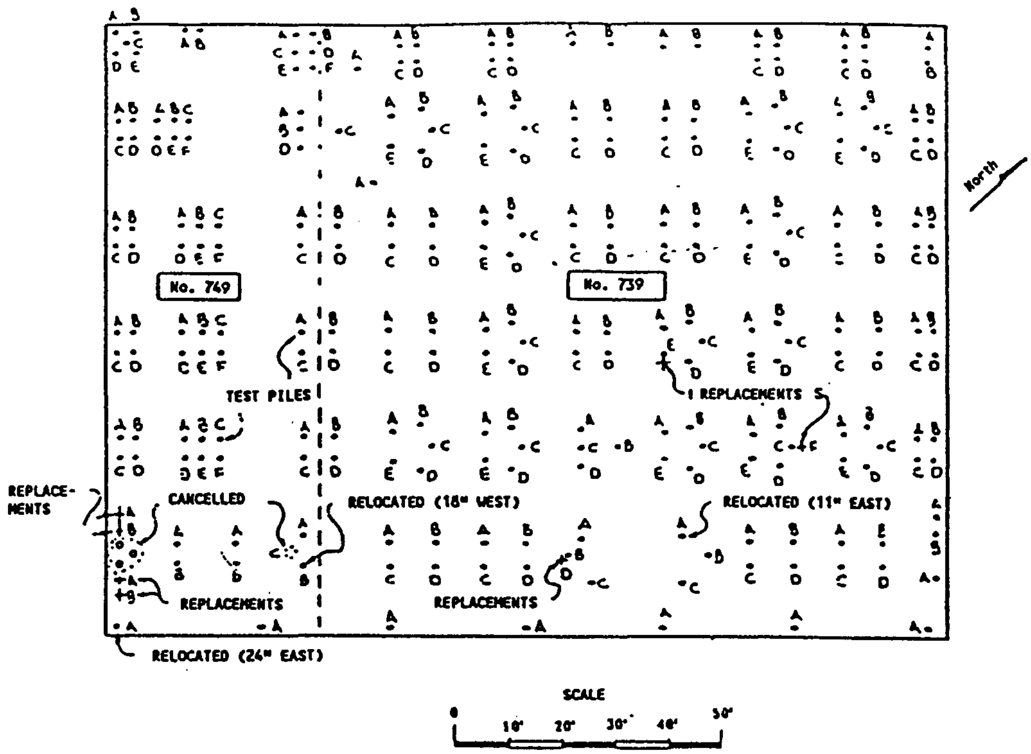
It is noteworthy that the elastic movement at 712 kN was about half the total movement, while no indication of pile or soil failure was evident from the head or toe movement curves. Furthermore, the net head movements were well below recommended building code criteria for maximum net movements. The performance in tension was equally satisfactory.

Most of the major structural rebuilding work was completed in the 8-month period following completion of the micropiles. Readings were taken regularly of the pile cap deflections at 16 locations. The range of cap movements during construction was 1.5 to 6.1 mm, entirely consistent with the test data in table 15 (total movements of 8.6 to 11.2 mm at twice service load, without the benefit of existing timber piles).

Old Postal Building, Washington, DC (Bruce, 1992)

Background. The original portion of the massive Old Postal Building was completed in 1911. A major extension followed in 1931. For many years, it served as the main post office for Washington, DC, being located adjacent to Union Station on Massachusetts Avenue, a few blocks north of the Capitol Building. The developer, acting for the Federal Government, planned to remodel the existing structure by adding new office floors in the center court area and constructing mechanical space below the existing lowest basement elevation of +7 m. This meant that existing foundations had to be upgraded and new columns added to support new interior framing. The existing supports were steel and concrete columns on large concrete footings and 350-mm-square caissons bearing on dense sands.

Originally, a cumbersome underpinning scheme was envisaged for the upgrade, involving hand-dug support, massive spread footings, and large-diameter caissons – both hand-excavated and mechanically drilled. However, the hand work would probably have caused significant undermining of the existing footings, leading to settlement, and drilled caisson work would have been inhibited by the very restrictive access and low headroom. Both techniques would have been unattractively time consuming and costly. This typical micropile alternative proposal resolved both concerns.



1 ft = 0.305 m

Figure 12. Plan of micropile arrangement -- Boylston Street, Boston, Massachusetts (Bruce, 1988 and 1989).

Table 15. Summary of test data on test piles 1 and 2, Boylston Street, Boston, Massachusetts (Bruce, 1988).

	Head (mm)		Toe (mm)	
	TP 1	TP 2	TP 1	TP 2
Compression test 710 kN				
Gross Movement	11.18	8.64	7.87	4.83
Net Movement	6.35	4.06	6.35	4.06
Tension test 210 kN				
Gross Movement	6.09	3.56	4.32	1.52
Net Movement	4.06	2.29	3.81	1.52

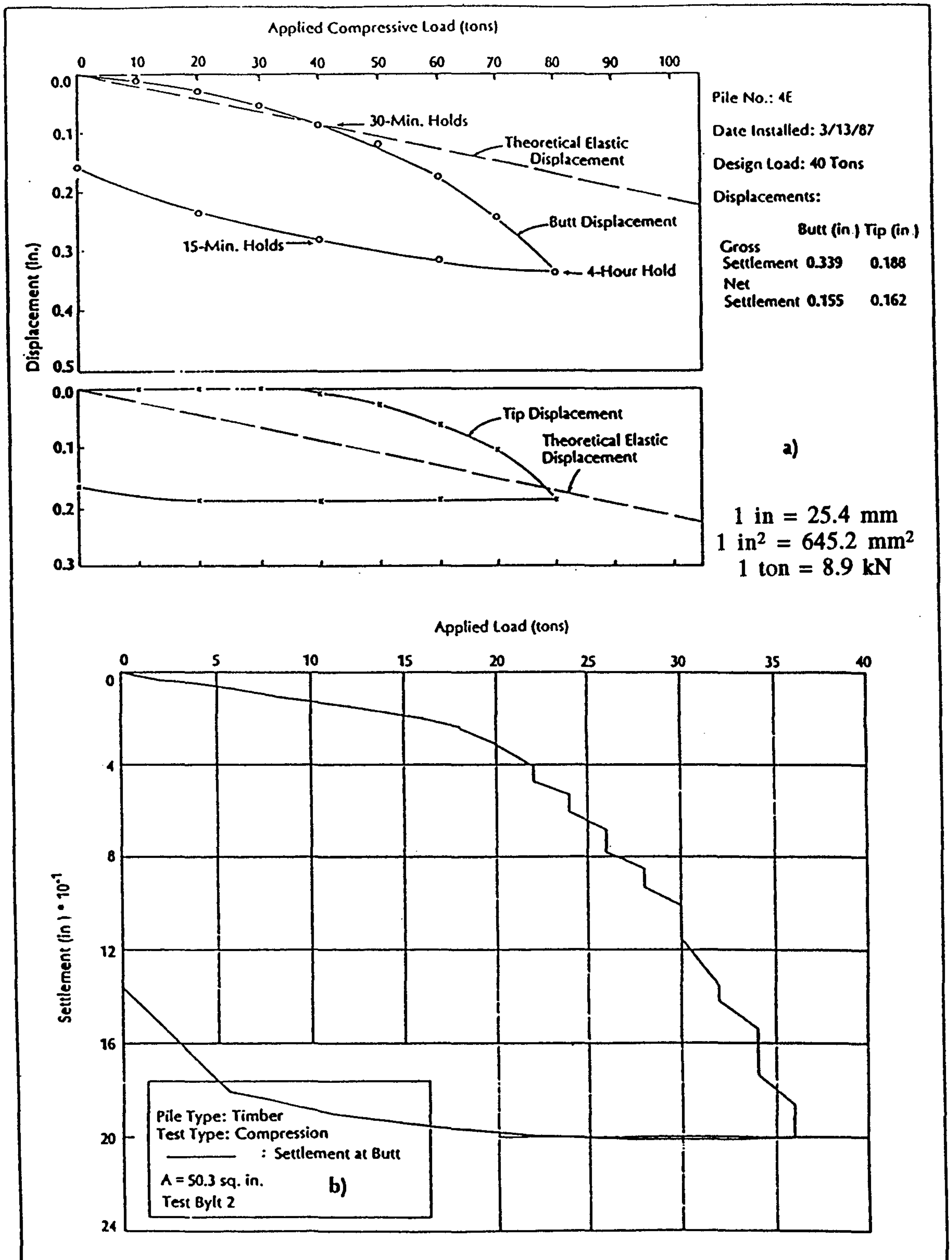


Figure 13. Load/settlement performance of: (a) drilled and grouted micropile and (b) driven timber pile, Boylston Street, Boston, Massachusetts (Bruce, 1988).

Site and Ground Conditions. The work was conducted underground in three main areas in the basement of the existing structure:

- B2 Level (EL 1.8 m): Large level area with about 4 m of headroom. Piles reached EL -13.5 m.
- B2 Level (EL 3.3 m): Most restricted area, headroom was 2.5 m. Piles also reached EL -13.5 m.
- B1 Level (EL 7.0 m): Open access with 4 to 6 m of headroom. Piles reached EL -10.5 m.

Under the concrete footings and 1 m of fill, the natural soils comprised recent alluvials, ranging from coarse to fine sands, laterally and vertically variable. Some gravel and mica were found sporadically, together with thin layers of cobbles or stiff clayey silt and silty sand in lower reaches. Typically, the sands were dense to very dense. The natural groundwater level was at about EL -1.5 m.

Design. Past experience and standard texts were used to design 390 vertical Type 1B piles in the B2 levels and 310 piles in the B1 level, each with a nominal service load of 670 kN. About 25 percent of the piles were installed in groups of 4 or 6, through 15 existing B2 (EL 1.8 m) footings comprising 2.1 to 4.2 m of concrete. Pile centers were often within 500 mm of existing columns.

Totals of 21 new reinforced concrete caps were created in B2 (EL 1.8 m), 17 in B2 (EL 3.3 m), and 53 in B1. These featured standard (and several non-standard, specially designed) plan geometries from 1.6 x 1.4 m (three piles) to 2.3 m square (nine piles). The minimum pile separation was 660 mm center to center, but was typically 760 mm.

Construction. Custom-built, short-mast diesel-hydraulic track rigs were used to rotate 178-mm-diameter, 13-mm-thick N80 casing using water flush, to target depth. Type I grout of w/c ratio 0.45 was injected under excess pressures of 0.6 to 0.7 MPa during progressive extraction of the casing over the lower 7.5 m. The casing was then reinserted 1.5 m into this pressure-grouted zone as permanent support. The lowermost 7.5 m of pile were reinforced by Grade 60, 36-mm-diameter rebar in 3-m coupled lengths. For those holes through existing footings, a 222-mm-diameter down-the-hole hammer was used to penetrate until significant steel was encountered. Thereupon, the hole would be completed with a 200-mm core bit. Load transfer between the casing of the pile and the concrete of the footing was ensured by the use of a special non-shrink, high-strength grout. For the new pile caps, the micropile casing was extended 100 mm up into the subsequent concrete, the horizontal reinforcing of which was fixed 50 mm above the top of the casing.

Testing and Performance. Four special test piles (TP's) were installed prior to constructing the production piles (table 16). TP1 and TP2 were tested cyclically, yielding the analysis provided in figure 14. TP3 and TP4 were also tested incrementally to maximum load in accordance with ASTM D1143. TP1 failed at the grout-soil interface, the founding horizon being on average finer and less dense than those for the other piles. Figure 14 also shows that the elastic movements of TP1 and TP2 were similar at the failure load of the former. This shows that load must have been transferred to similar depths in both piles, despite the nominal difference in "free length" upon construction. The elastic performance of TP3 and TP4 was likewise similar, supporting the observation.

Table 16 also highlights higher total creep amounts in TP1 and TP2 – simply a reflection that there were far more creep monitoring periods during the cyclic loading than in the progressive loading process. This clearly impacts overall permanent movement and is an important point to bear in mind when judging pile performance as gauged by this criterion.

A separate pullout test was conducted in an existing column footing in the B2 (EL 1.8) level to explore grout-concrete bond development. A special element was grouted 1.4 m into a 222-mm-diameter hole drilled through the concrete. A high-strength, non-shrink grout was used. After repeated cyclic loading to 2340 kN (79 percent Guaranteed Ultimate Tensile Strength and equivalent to 350 percent service load), the maximum uplift recorded was 0.13 mm, reduced to 0.02 mm upon unloading. Assuming uniform bond distribution, an average grout-concrete bond greater than 2.4 MPa had therefore been safely resisted at this interface.

Following installation of the micropiles, the structural renovation has progressed and the foundations have performed perfectly.

Presbyterian University Hospital, Pittsburgh, Pennsylvania (Bruce, 1992a)

Background. The Presbyterian University Hospital complex occupies two extremely congested city blocks. When the need for more facilities became apparent, it was therefore decided to vertically extend and laterally link several existing operational structures. Overall, 150,000 sq. m of new facilities were to be built in four major additions. This project, conducted within a fully functional facility, necessitated some complex and innovative foundation engineering solutions involving excavation support and structural underpinning. One of the most delicate operations was associated with the completion of a new Magnetic Resonance Imaging Center. The construction of a new elevator pit called for a 9.1-m-deep excavation directly underneath 3 exterior column footings of the adjacent 13-story hospital structure. The pit, 18.3 x 9.8 m in plan, was further bounded on two sides by five additional footings and these sides required anchored lateral support.

It had become standard practice to support columns in such circumstances by conventional underpinning pits and needle beams. However, in this instance, conventional techniques would not accommodate the difficult access conditions nor the specified limit on column movement of 3 mm. The solution featured high-capacity Type 1A micropiles founded in rock.

Site and Ground Conditions. Access was very restricted laterally, headroom was as low as 3.7 m, and the work had to be conducted within the confines of a fully operational medical facility. The piles had to be installed through 1.1 m of existing reinforced concrete footings cast directly on fractured, fissile medium hard/hard siltstone, occasionally calcareous or limey.

Table 16. Test pile data, Postal Square, Washington, DC (Bruce, 1992).

	TP 1	TP 2	TP 3	TP 4
Area/Level (m)	B2-1.8 m	B2-1.8 m	B2-3.3 m	B2-6.9 m
Length (m)	10.8	15.5	10.8	17.7
Bond (m)	7.5	7.5	7.5	7.5
Maximum Test Load (kN)	834	1330	1330	1330
Elastic Movement at Maximum Load (mm)	4.39	11.71	9.73	9.50
Permanent Movement at End of Test (mm)	7.95	7.39	4.75	2.87
Total Cumulative Creep During Test (mm)	2.44	4.42	1.50	1.55

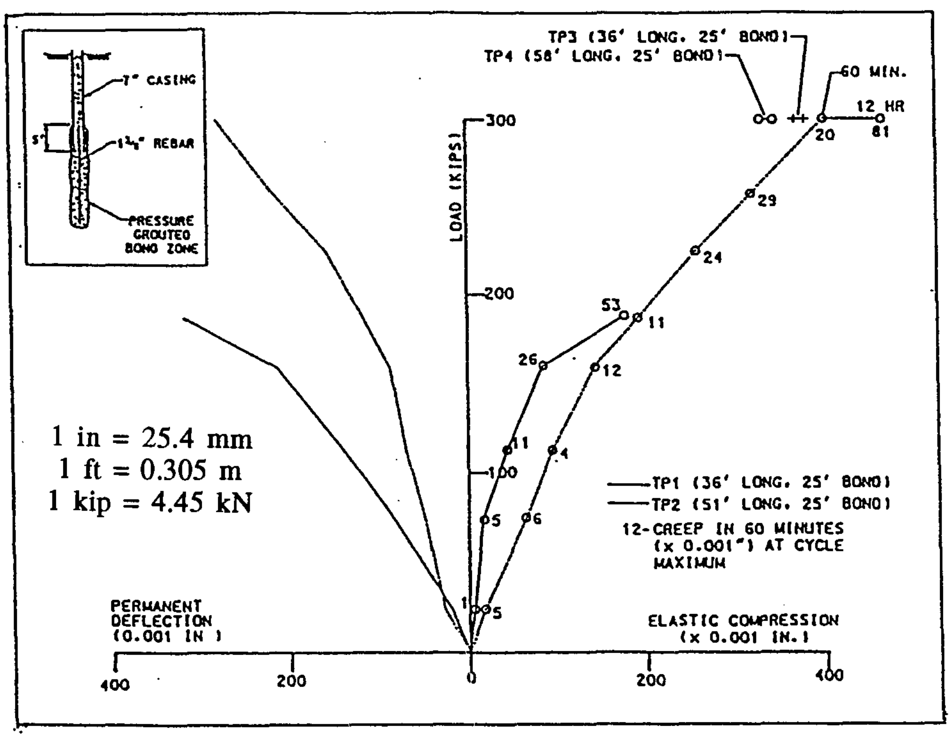


Figure 14. Elastic/permanent performance of test pile 1 and test pile 2, Postal Square, Washington, DC (Bruce, 1992).

Design and Construction. At each existing footing, six micropiles (four service, plus two redundant) were installed in 216-mm-diameter holes, drilled vertically by rotary percussive methods with air flush to the target depth, 13.1 m below the footing. Each pile had a service load of 1112 kN. The reinforcing element consisted of 178-mm-diameter, 13-mm wall thickness, N80 casing placed full depth and then tremied full of neat cement grout (w/c = 0.45). The upper 7.0 m of each pipe was greased on the outside to debond it from the surrounding grout in that region and to ensure load transfer directly into the 3.0-m-long bond zone. The suitability of this design had been proven in the earlier test program, described below.

A structural steel jacking frame was then erected over the top of the piles and fastened to the existing steel column. Each of the steel columns -- supporting an occupied hospital building -- was then sequentially lifted off its existing spread footing by a distance of 1.5 to 3.0 mm. This effectively preloaded the piles to prevent any later settlement of the building, and transferred the column loading into the bedrock, but 7.0 m below. Excavation then proceeded, supported laterally by beams, shotcrete lagging, and prestressed rock anchors. As the excavation deepened, cross frames were welded to the pile casings to limit the unbraced lengths of these piles now exposed and acting as grout-filled steel columns.

Testing and Performance. By the end of excavation, the foundations of the existing structure could be seen resting on the micropile groups, 7 m off the bottom of the excavation. During and after excavation, no movement of the structure was measured, and the whole operation was therefore considered successful. Bearing in mind that one of the most common problems foreseen for micropiles was their potential for buckling or bending, this unique project -- featuring micropiles with no surrounding ground to offer any lateral restraint -- confirmed that correctly designed and constructed micropiles can operate reliably under such conditions.

Testing of production piles was not possible in this case, so a full-scale pre-production test pile was installed beforehand at an adjacent location. Using identical construction methods, a micropile with 6.1-m bond length was formed in the same geological stratum. The total length was 15.2 m, including 9.1 m of debonded free length. The casing was preassembled in the workshop and consisted of five separate lengths, hand-tightened together. Two "telltales" were incorporated -- one each at the top and bottom of the bond zone. A thick, soft wooden plug was attached to the bottom of the reinforcing pipe to eliminate any possible end-bearing contribution and to allow only side shear to be mobilized. As part of the contract requirement, the pile was then tested to twice service load (2224 kN), according to ASTM D1143 (modified to allow cycles at 25 percent steps). Results are summarized in table 17. At 1112 kN, the elastic compression of 5.77 mm was exactly that predicted, while the permanent displacement of 1.3 mm was proven (by the telltales) to be due to some inelastic compression of the steel casing itself. While loading from 1780 to 1890 kN, a "bump" was recorded and the load dropped to 1335 kN. Load was then increased to 2250 kN when an additional "bump" occurred. However, when the data from the cyclic loading and the telltales were analyzed, it became clear that:

Table 17. Summary of test pile performance, Presbyterian University Hospital, Pittsburgh, Pennsylvania (Bruce, 1992a).

Load Cycle Maximum (kN)	Total Head Movement at Maximum (A) (mm)	Permanent Head Movement at Subsequent Zero (B) (mm)	Therefore, Elastic Movement at Maximum (A) - (B) (mm)	Apparent Bottom Telltale Movement (Relative to Head) (mm)
560	3.23	1.07	2.16	0.94
1120	7.09	1.32	5.77	1.12
1235	11.38	1.96	9.42	1.60
1345	16.84	8.36	8.48	7.49
2240	25.91	13.13	12.78	11.96

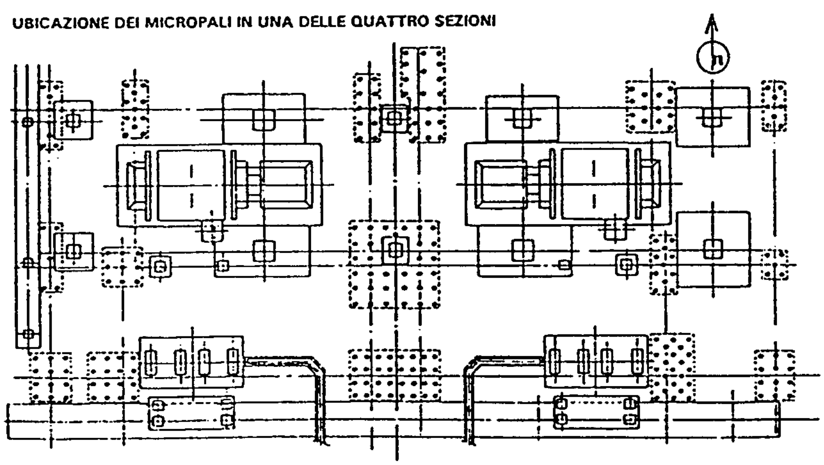


Figure 15. Layout of micropiles in one of the four areas, Brindisi ENEL Power Plant, Italy (Rodio, 1993).

1. The pile elastic movement at 2224 kN was exactly as predicted.

2. The apparently large permanent movement (table 17) was due to a permanent "one-off" shortening of the steel pipe. A review of the assembly records of the pile showed that there had been several "unshouldered" hand-tightened joints between adjacent casing sections. It was suspected that each joint was unshouldered about 3 to 6 mm. Thus, the sudden 12-mm permanent compression of the pile material was understandable, and when subtracted from the measured permanent movement of 12.8 mm, gave a true movement of the pile toe into the rock mass of only 0.8 mm at 2224 kN. There was negligible creep at all load increments.

Thereafter, the pile was tested to a maximum load of 3000 kN before it became clear that material failure of the steel casing was occurring. At this load, the steel had compressed 78 mm (as measured by telltales), compared with the measured head permanent displacement of 82 mm. Thus, at 3000 kN, a true permanent movement of the pile of 4 mm had been recorded, while analysis proved the perfect elastic performance of the pile, with a calculated debonded length of about 1 m into the bond zone.

FOUNDATIONS FOR NEW STRUCTURES

Restrictive Site or Access Conditions

ENEL Power Plant, Brindisi, Italy (Rodio, 1993)

Background. The foundations of a new catalytic densification plant for the Brindisi thermoelectric power plant had to be constructed between existing structures in four areas. High-capacity support had to be provided in very restricted access conditions and in the midst of a fully operational industrial facility.

Soil Conditions. The typical ground profile was recorded as:

0 - 4 m	overburden
4 - 14 m	silty sand
14 - 19 m	stiff clayey silt
19 - 29.5 m	calcarenite
Below 29.5 m	dolomite

Design. A total of 960 "Ropress" Type 1D micropiles, each with 1000-kN service loads, were foreseen in groups of 5 to 33 per plinth (figure 15). Each micropile was about 30 m long, including a nominal embedment of 0.5 m into the dolomite. The design featured a 178-mm-diameter, 14.2-mm-thick steel tube in the upper 6 m and a similar diameter, but thinner (11 mm), casing below. The lowermost 10 m (in the calcarenite) comprised a 114-mm-diameter, 20-mm-thick steel pipe, with sleeves to permit post-grouting – a common feature of such piles in Italy. A total of 30,000 linear meters of piling were foreseen.

Construction. Different drilling methods were experimented with to combat the various soil and rock conditions. For the most part, diesel-hydraulic rotary drilling rigs were used to advance a 269-mm-diameter tricone bit and 219-mm-diameter rods using bentonite mud flush. The reinforcement was placed into the slurry-stabilized hole and then the annulus was grouted through the lowermost sleeve, completely displacing the slurry. Following initial set of this grout, post-grouting was conducted via a double packer. Grout strengths exceeded 30 MPa. The verticality of each pile was verified within the 1 percent tolerance. Despite the geotechnical and access restrictions, production rates exceeded anticipated levels.

Testing and Performance. Preliminary load tests confirmed excellent behavior under the maximum loads foreseen in the design:

- Lateral load test to 79 kN with total deflection of 13.1 mm (including 1.9 mm permanent) (figure 16).
- Compressive load test to 2465 kN (2.5 times service load) with total movement of 20.2 mm (including 3.2 mm permanent) (figure 17).
- Tensile load test to 1220 kN with total extension of 10.0 mm (including 1.7 mm permanent) (figure 18).

These successes were mirrored by the compression tests to 2000 kN conducted on five production piles, each using two piles as reaction (and so themselves tested to 1000 kN in tension) (figure 19).

The pre-production test piles and the production test piles were heavily instrumented (figure 20) for research purposes.

<u>Brooklyn-Queens Expressway, New York (Bruce and Gemme, 1992)</u>

Background. The Brooklyn-Queens Expressway is a six-lane viaduct between Metropolitan Avenue and Kingsland Avenue in the Borough of Brooklyn. It runs in a north-south direction approximately 1.5 km east of the East River and is designated as Interstate Route 278. It is the only controlled-access expressway to connect the boroughs of Queens and Brooklyn, and it provides an expressway link for several counties with the Williamsburg, Manhattan, Brooklyn, and Verrazzano Bridges, and the Brooklyn Battery Tunnel. This section of expressway was completed in the early 1950's and consists of a series of simply supported spans resting on pile-supported bents.

A major improvement program was put into place to replace the deck of the viaduct and to add a new center lane and several new entry-exit ramps. These were needed to correct access, geometric, and safety deficiencies that were exacerbated by severe traffic congestion experienced particularly during rush hours.

Due to this viaduct being a portion of a major arterial highway with high traffic volumes, maintaining traffic became a fundamental criterion for project approval. In order to maintain a minimum of two (out of three) lanes of traffic in each direction during construction, a temporary viaduct (adjacent to the existing structure) had to be constructed prior to lane closures in each direction to accommodate rehabilitation.

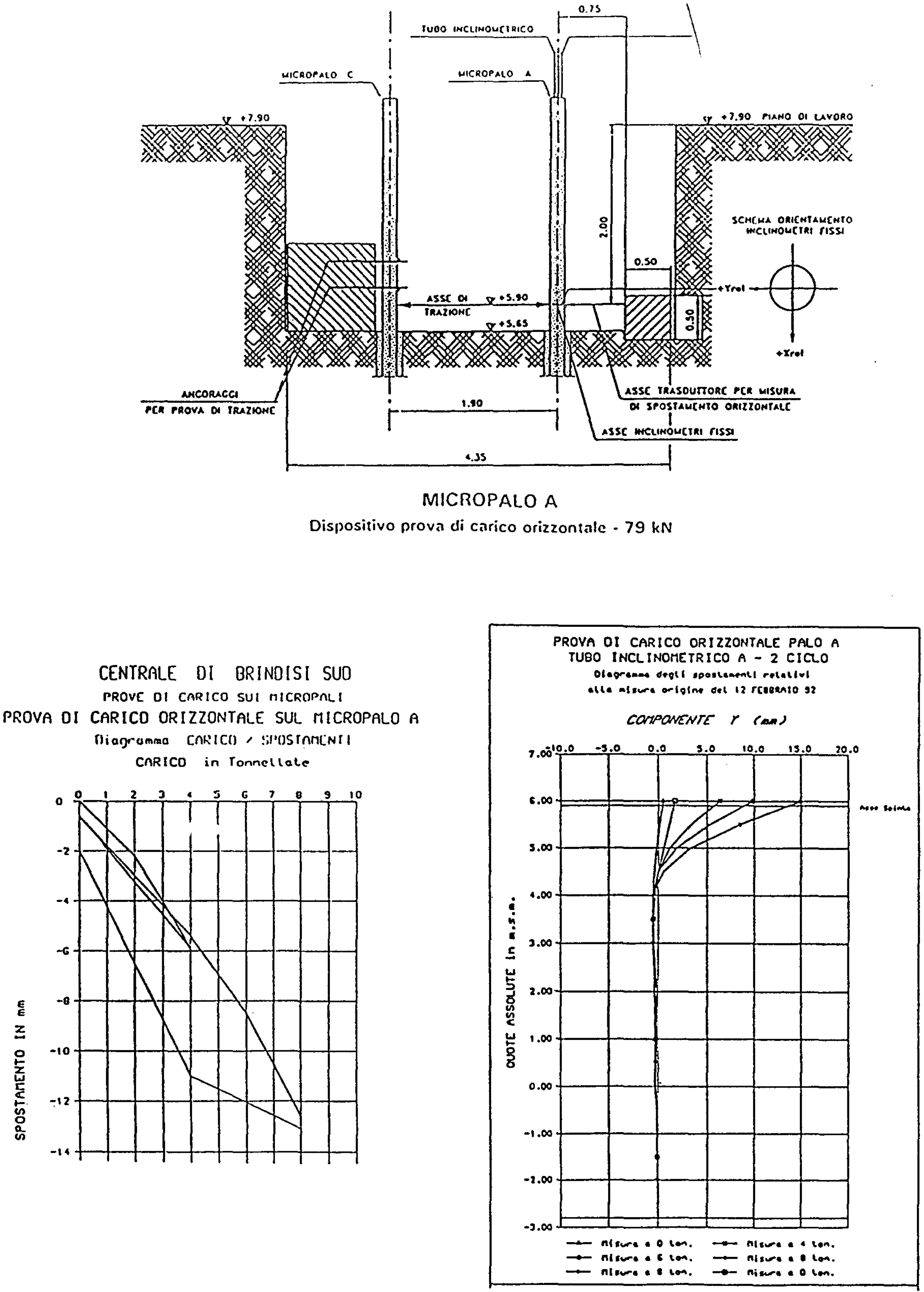


Figure 16. Details of lateral load test on test pile A, Brindisi ENEL Power Plant, Italy (Rodio, 1993).

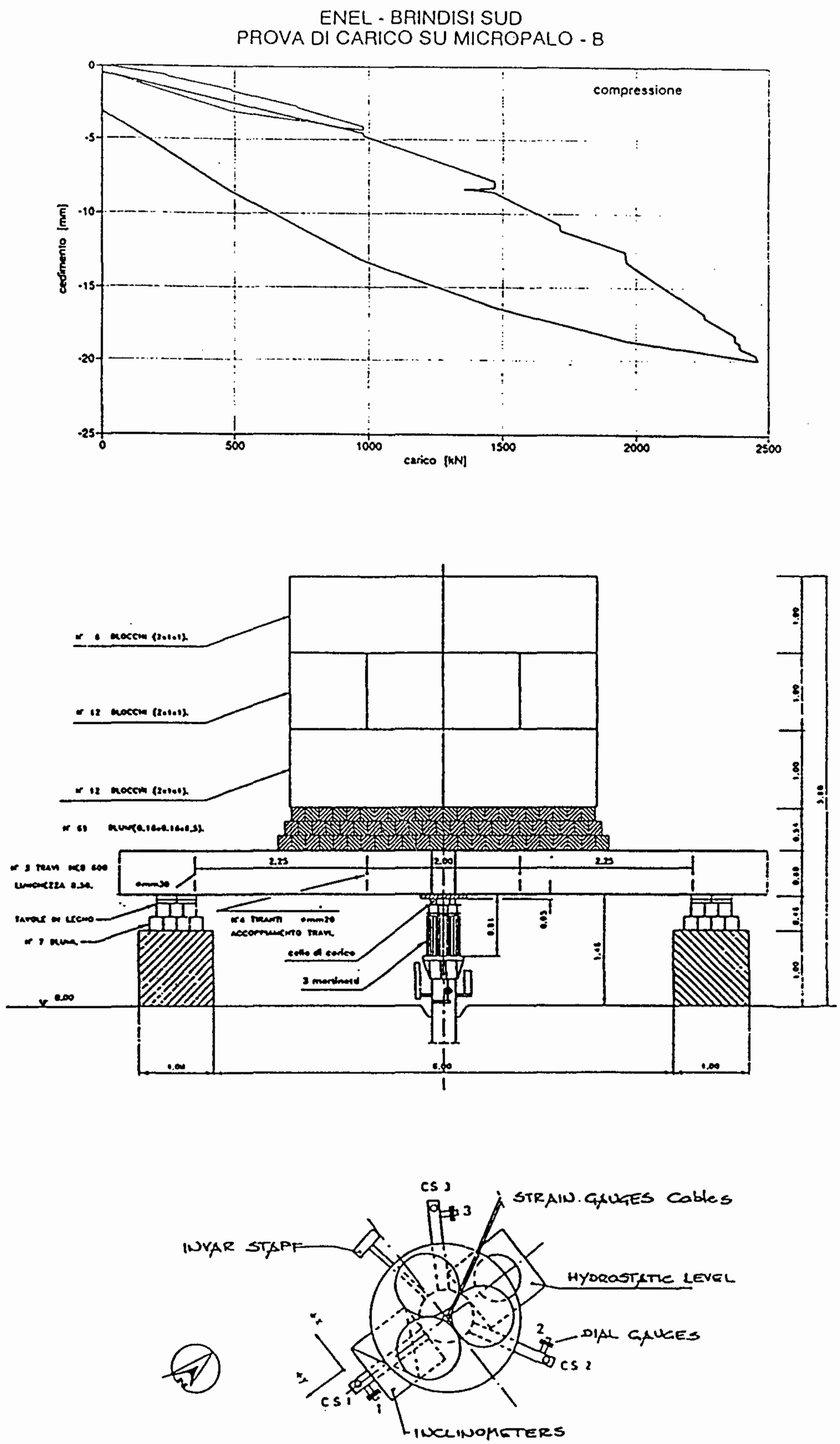


Figure 17. Details of compressive test on test pile B, Brindisi ENEL Power Plant, Italy (Rodio, 1993).

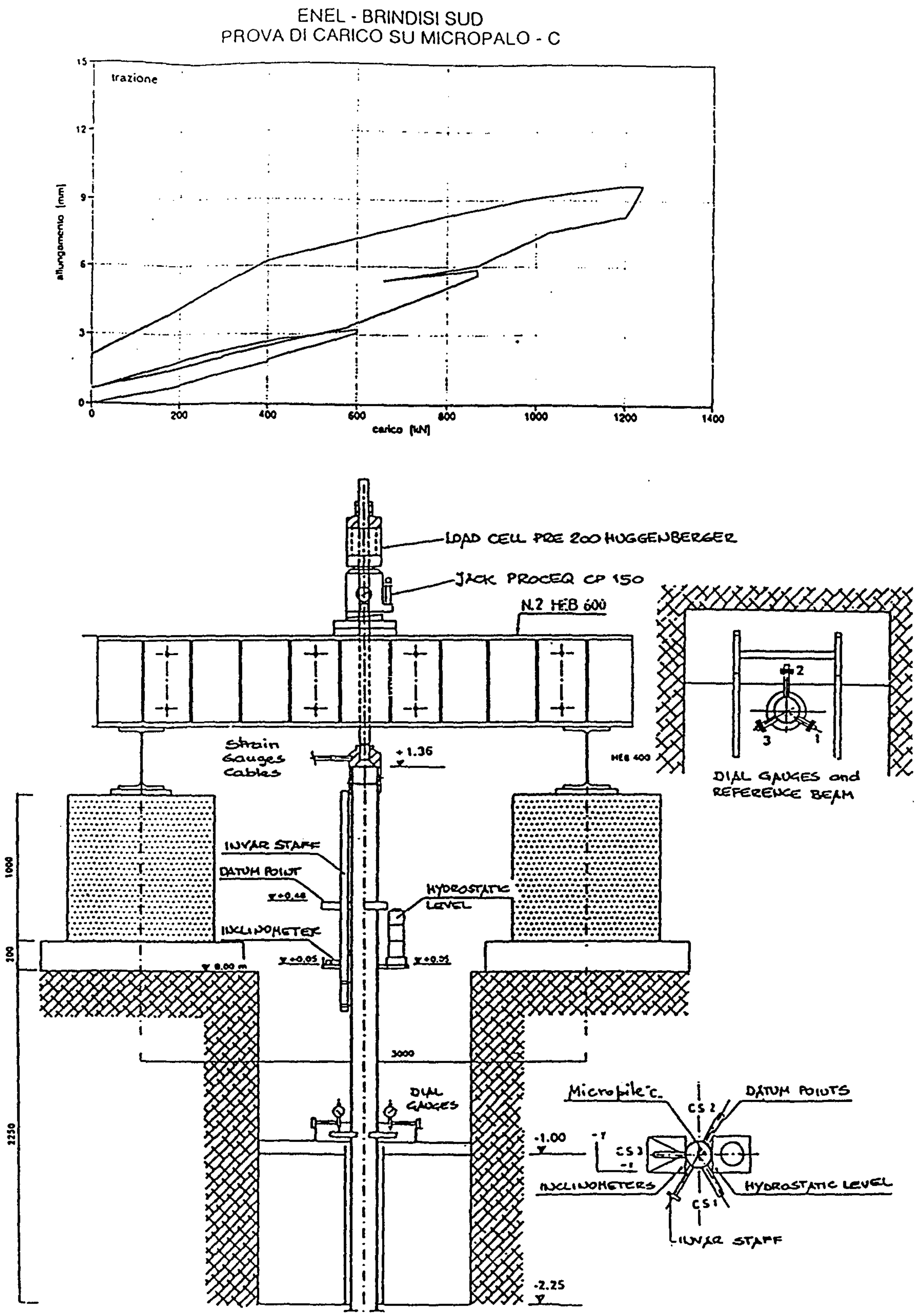
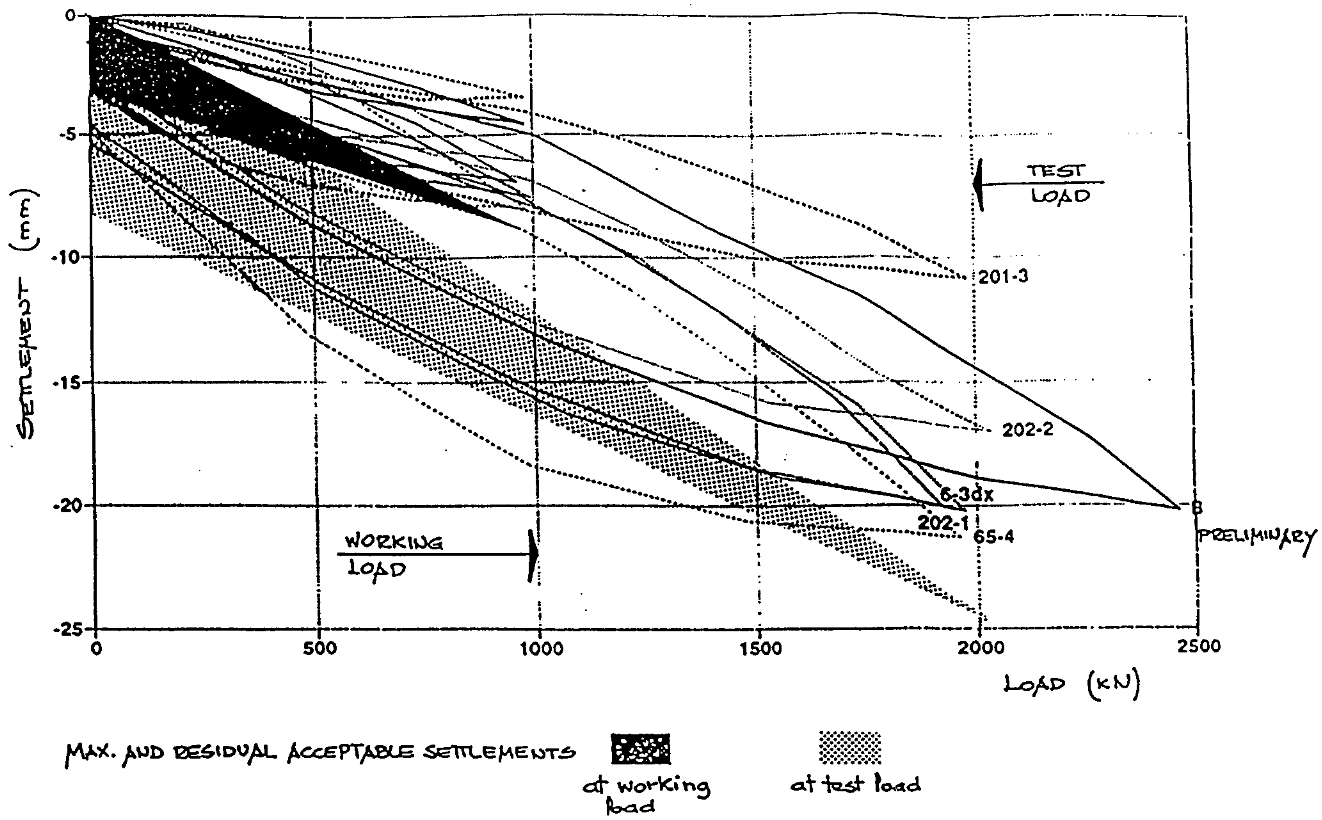


Figure 18. Details of tension test on test pile C, Brindisi ENEL Power Plant, Italy (Rodio, 1993).



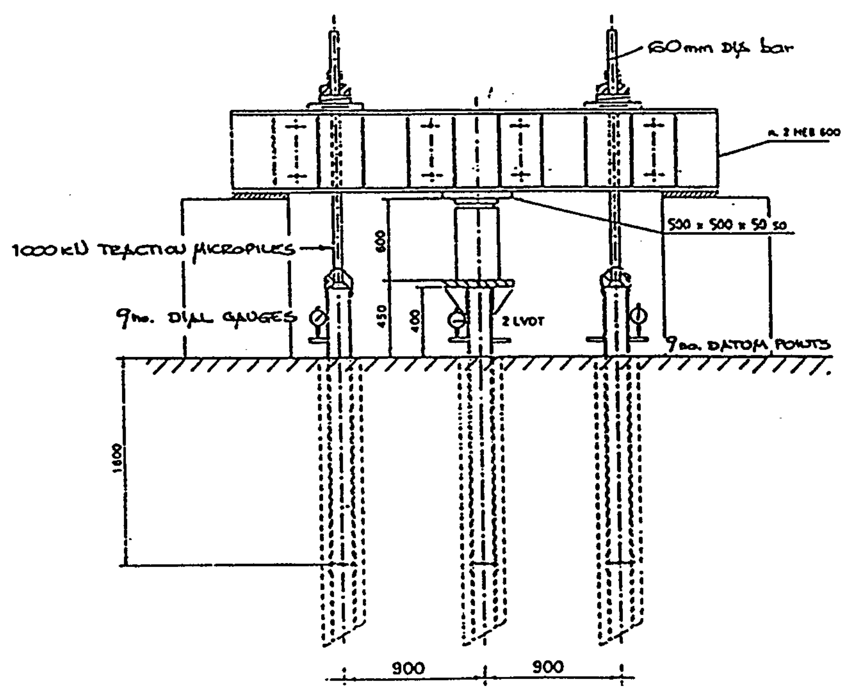


Figure 19. Details of compression tests, Brindisi ENEL Power Plant, Italy (Rodio, 1993).

INSTRUMENTATION USED FOR LOAD TESTS

	INSTRUMENT	DATA ACQ. SYSTEM	MICROPILES																	
			PRELIMINARY			ACCEPTANCE TEST														
			A	B	C	1			2			3			4			CF 3		
						202	199	205	202	199	209	201	198	204	65	63	67	8	2	10
LOAD APPLIED	LOAD CELLS																			
	DAVY TC-50	●	●																	
	HBM RA 100 (n.3)	●		●		●														
	HUGGENBERGER PRE 200 ±1% f.s.				●															
	OG-295 OTR f.s.2000 KN ±0.5 f.s.								●											
	PRESSURE TRANSDUCER																			
	f.s. 500 bar cl.0.09%	●	●	●	●	●			●			●			●			●		
STRAIN ALONG THE MICROPILE — LOAD ALONG THE MICROPILE	STRAIN GAUGES																			
	SIS L311 OTR OG299 f.s. 2500	●		●	●	●			●											
	TUBE SECTIONS INSTRUMENTED BY STRAIN GAUGES					●			●											
INCLINATION	INCLINOMETRIC PROBE																			
	Alluminium casing 90-mm-DIA - probe MK4 Geotechnical Instr.		●																	
	FIXED INCLINOMETERS X-Y																			
	Schaevitz f.s.14.5 precis. ±0.1 mm/m	●	●	●	●	●			●			●			●			●		
SETTLEMENT — LIFTING	HYDROSTATIC LEVEL METER																			
	Enel ULC f.s. 40 mm precis. ±0.1 mm/m	●		●	●	●						●			●					
	HIGH-PRECISION LEVELING																			
	Level Wild NA2000 Wild Bar Code Staff	●		●	●				●									●		
	Level Wild N3 and invar staff			●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●
	DIAL GAUGES (n.3)			●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●	●
	LVDT TRANSDUCERS																			
	LVDT-RDP f.s. 150mm (n.2) linear. 0.5%	●										●			●			●		
	POTENTIOMETRIC TRANSDUCERS																			
	wire type Royelco f.s. 10" acc. ±0.1%	●	●				●	●												

Figure 20. Details of instrumentation of test piles, Brindisi ENEL Power Plant, Italy (Rodio, 1993).

Small-diameter (approximately 300 mm) bored piles were specified for the permanent viaduct and ramps, and larger diameter (600 mm, 760 mm, and 915 mm) bored piles were specified for the temporary viaduct. Bored piles were specified for this project due to the otherwise adverse vibration effects that pile-driving impact hammers would have on the many adjacent older, sensitive buildings.

This was the first New York State Department of Transportation project where small-diameter bored, cast-in-place piles were specified to be designed by a prequalified contractor to meet predetermined design capacities.

Site and Ground Conditions. The general foundation conditions at the site were highly variable, but generally consisted of:

- Between 3 to 5 m of loose to medium, compact, miscellaneous fill containing silt, sand, and gravel with bricks and similar materials.
- Up to 9 m of layers and lenses of loose silt, sand, and clay (organic near surface).
- Up to 15 m of compact silty sand, occasionally gravelly.
- Stiff varved silty clay and clayey silt (Gardiners Clay).

Bedrock was not encountered to the maximum explored depth of about 30 m. Generally, the compact silty sand and the lower lenses of silt, sand, and clay were recognized as being adequate load-bearing materials and commenced at a depth of about 15 m below the existing ground surface. The piezometric level was encountered between 3 and 5 m below the existing ground surface.

There were significant access and headroom restraints, especially where drilling had to be performed under the existing viaduct (about 5 m of headroom). In addition, construction had to accommodate traffic controls, protection of buried and overhead utilities, and noise and vibration impact mitigation.

Design. Approximately 120 new pile caps, each with between 2 and 10 piles per cap, were proposed as follows in different stages to accommodate maintenance and protection of traffic:

Construction Stage		Number of New Pile Caps	Pile Diameter	Service Load (kN)
1	(eastbound permanent viaduct)	Approx. 30	Approx. 300 mm	220, 360, 450
2	(temporary viaduct)	Approx. 60	600 mm, 760 mm, 915 mm	Variable: 270 - 650
5	(westbound permanent viaduct)	Approx. 30	Approx. 300 mm	220, 360, 450

All piles were to be designed as vertical CASE 1 elements. The small-diameter piles were specified to be designed as friction piles by a prequalified contractor. The prequalification consisted of requiring the contractor performing the work to submit proof of: (1) two projects on which he or she had successfully designed and installed similar bored piles or tiebacks using non-displacement methods under similar site conditions; and (2) the foreperson having supervised the successful installation of the same on at least two projects in the past 2 years. The specifications indicated that the grout mix, steel casing, and/or reinforcement had to meet specified minimum requirements, and included general provisions concerning shop drawing submittals, drilling, casing removal, post-grouting, and construction tolerances. The contractor's proposal to found the temporary viaduct also on higher capacity, smaller diameter micropiles was approved by the State with the provision that the piles be inclined where permitted by right-of-way and utility conditions and the pile caps be tied to the permanent viaduct in the direction where piles could not be inclined to provide the necessary lateral restraint.

The specification called for completing several successful static pile load tests as a basis for pile acceptance:

Location	Number of tests on non-production piles prior to installation of production piles	Number of tests on production piles
Permanent Viaduct	4	Bents 19, 26, 31, 37, 44, and at locations designated by the Engineer.
Temporary Viaduct	2	Bents 27, 37, 50, 69, and at 1 percent of remaining bents at locations designated by the Engineer.

Construction. Short-mast diesel-hydraulic crawler-mounted rigs were used to rotate 178-mm-diameter, 13-mm-thick Grade N80 steel casings with water flush to full depth. Neat cement grout was tremied in (minimum 28-day crushing strength of 35 MPa) and then pressurized to about 0.6 MPa as the casing was withdrawn 4.5 m. The casing was then plunged back to full depth, thus providing a fully cased Type 1B pile. A small number of piles were installed as centrally reinforced Type 1B piles by a different contractor.

Testing and Performance. Subsequent testing showed that when founded in silty sands, the 900-kN service load could be readily attained. However, when the bond zone predominantly consisted of looser deposits of silt, sand, and clay, the practical minimum service load was estimated as 550 kN. This inability to reach higher loads reflected the fact that the soils would not naturally "seal" around the drill casing during its withdrawal, thus preventing the application

Table 18. Summary of micropile tests, Brooklyn-Queens Expressway, New York (Bruce and Gemme, 1992).

SUMMARY OF PILE TESTS, BQE, N.Y.

LOCATION	BETWEEN BENTS 35 & 36	BETWEEN BENTS 38 & 39	BETWEEN BENTS 38 & 39	BETWEEN BENTS 30 & 31	BETWEEN BENTS 15 & 16	BENT 34	BENT 31	BETWEEN BENTS 12 & 13	BETWEEN BENTS 12 & 13	BENT 16	BENT 26	BENT 11B-C	BENT 26C	BENT 28C	BETWEEN BENTS 9 & 10	NEAR BENT 1	BENT 27C	BENT 82
LOAD TEST NO.	TP 1	TP 2	TP 3	TP 4	TP 5	P 1	P 2	1	2	3	4	7	5	6	–	–	8	T 1
ORDER NO.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
PERM. OR TEMP.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	PERM.	TEMP
PROD. OR NON-PROD.	NON.	NON.	NON.	NON.	NON.	PROD.	PROD.	NON.	NON.	PROD.	PROD.	PROD.	PROD.	PROD.	NON.	NON.	PROD.	PROD
AGE AT TEST (DAYS)	37	13	N/A	14	29	N/A	N/A	5	6	5	7	19	7	7	N/A	N/A	7 3 POST-GRT.	8
NOMINAL DIA. (INCHES)	9.05	9.05	9.05	9.05	9.05	9.05	9.05	7	7	7	7	9.05	7	7	9.05	9.05	7	7
NOMINAL BOND LENGTH (FEET)	35+/–	35+/–	35+/–	35+/–	35+/–	35+/–	35+/–	15	18	7	6	20	20.5	21	35+/–	35+/–	15	30
TOTAL LENGTH (FEET)	57	59	N/A	65	78	N/A	N/A	40.5	50	50.5	50	50.5	62	66.5	61	61	50	50
ORIGINAL DESG. LOAD (TONS)	50	100	100	100	100	50	75	100	100	100	100	100	100	100	100	100	60	60
MAX. TEST LOAD (TONS)	186	146	125	175	275	100	150	165	200	200	125	150	160	125	175	200	175 200 POST-GRT.	120
TOT. DEFLEC @ MAX.LOAD (INCHES)	N/A	N/A	0.51	0.86	0.36 @200T	0.29	1.2	1.7	0.51	0.42	1.85	0.77	0.43	0.38	2.7	0.58	1.24 0.72 POST-GRT.	0:18
PERM. DISPL AFTER MAX LOAD (INCHES)	N/A	N/A	FAILED	FAILED	0.14 @200T	0.10	FAILED	FAILED	0.13	0.13	FAILED	0.44	0.12	0.13	2.2	0.10	0.74 0.20 POST-GRT.	0.09

1 ft = 0.305 m, 1 in = 25.4 mm, 1 ton = 8.9 kN

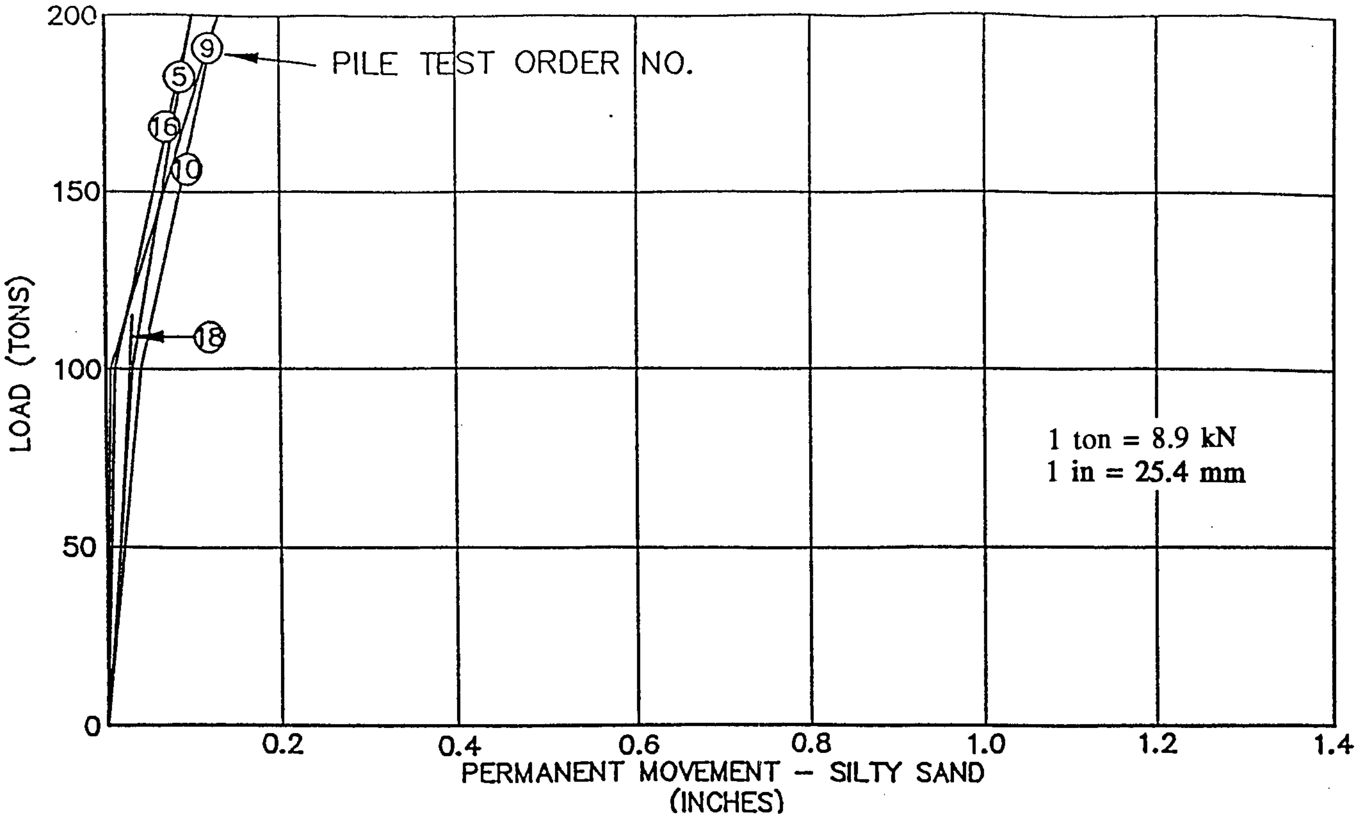


Figure 21. Permanent movement of test piles founded mainly in silty sand, Brooklyn-Queens Expressway, New York (Bruce and Gemme, 1992).

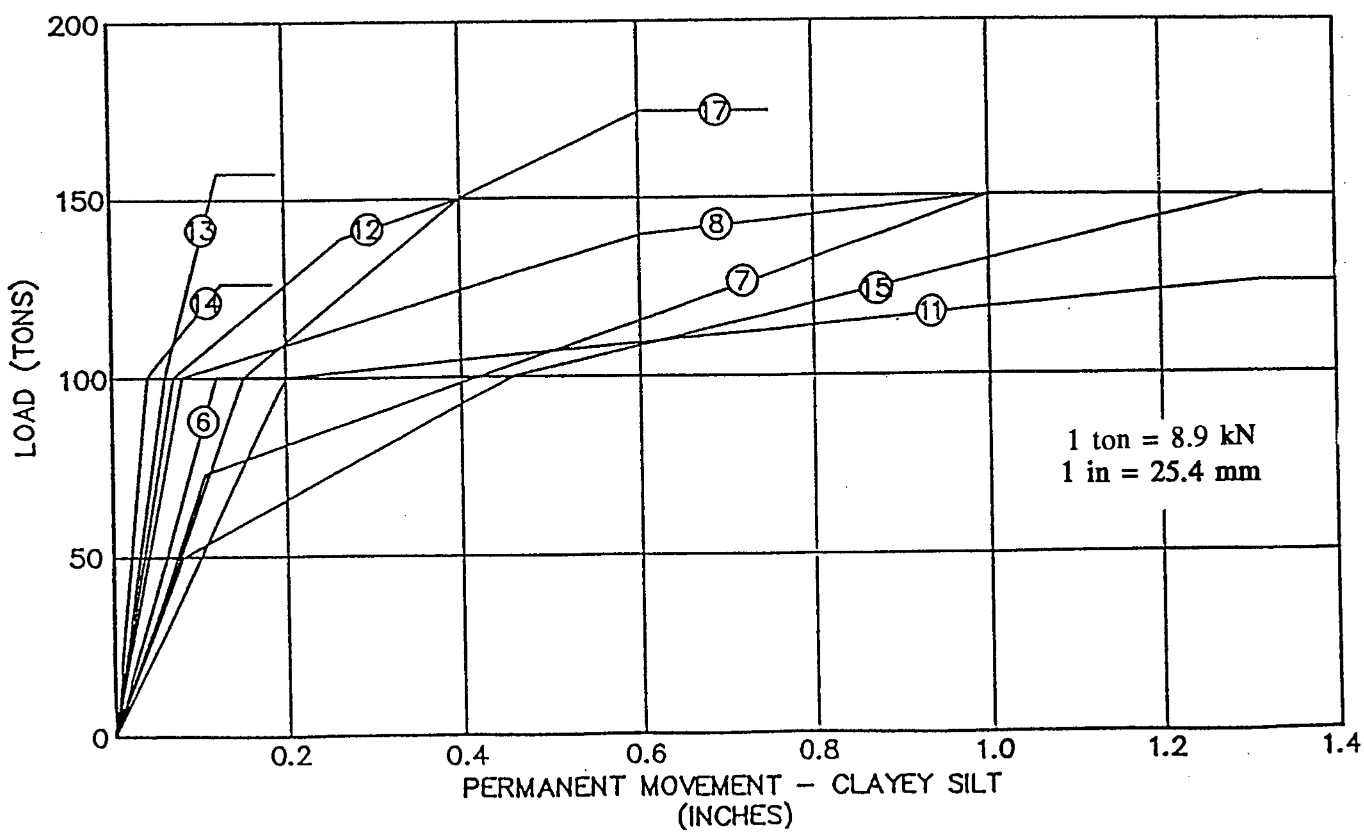


Figure 22. Permanent movement of test piles founded mainly in silts, Brooklyn-Queens Expressway, New York (Bruce and Gemme, 1992).

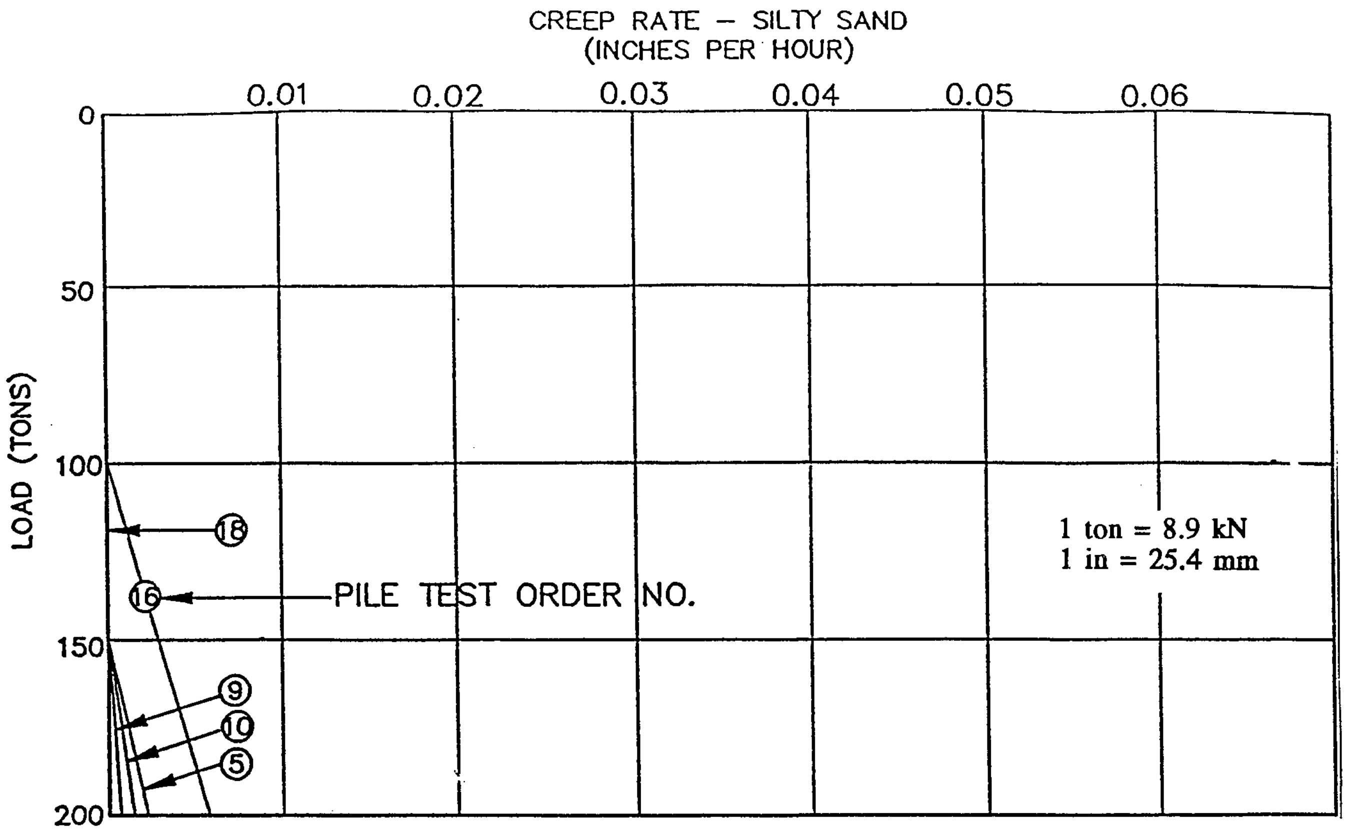


Figure 23. Creep rate for test piles in silty sands, Brooklyn-Queens Expressway, New York (Bruce and Gemme, 1992).

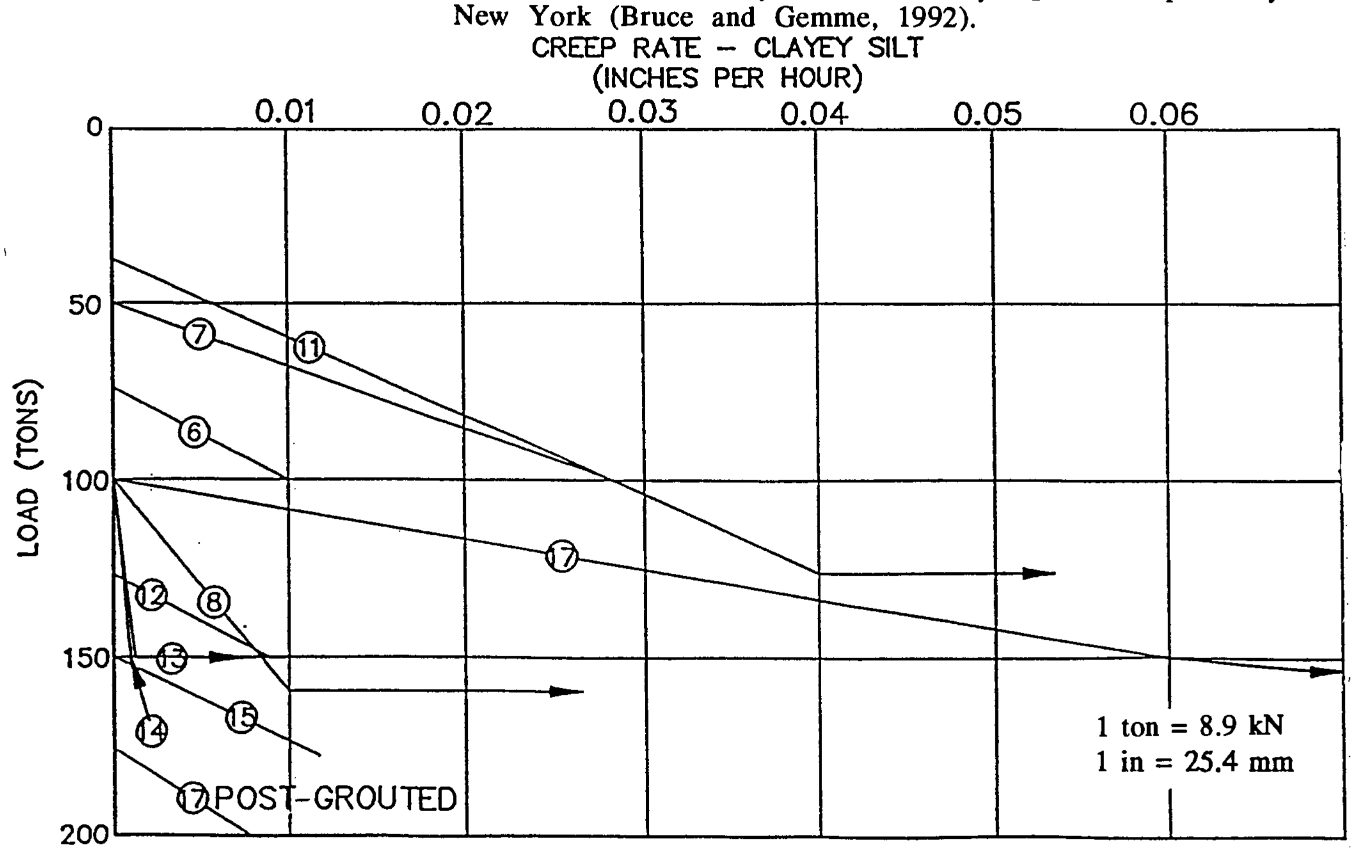
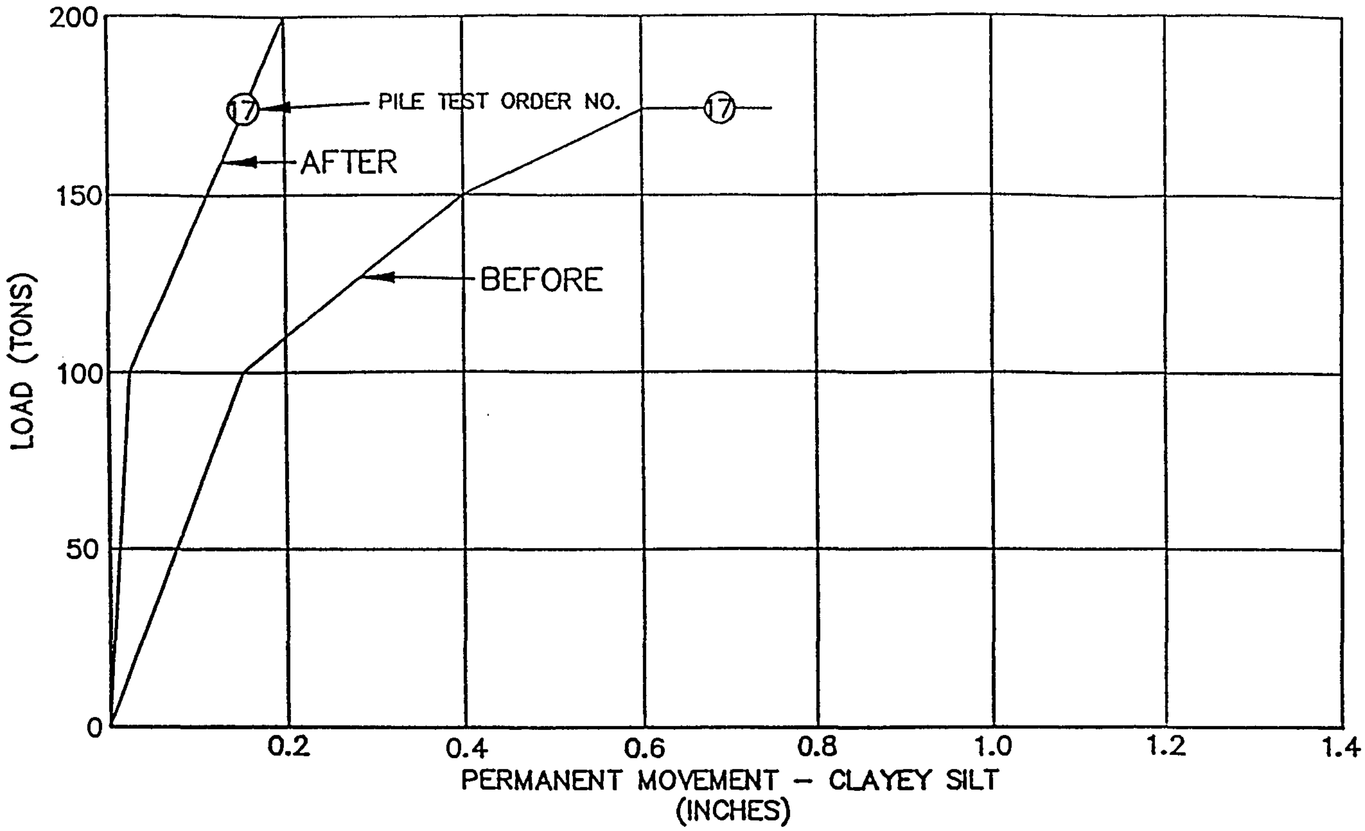


Figure 24. Creep rate for test piles in clayey silts, Brooklyn-Queens Expressway, New York (Bruce and Gemme, 1992).



1 ton = 8.9 kN
1 in = 25.4 mm

Figure 25. Performance of test pile at Bent 27C in clayey silts, before and after post-grouting, Brooklyn-Queens Expressway, New York (Bruce and Gemme, 1992).

of the target grouting pressures. A test with postgrouting techniques did raise grout/soil bond capacities to a level capable of providing the original service-load target. However, the contractor, given the overall site and project restraints, proposed instead to derate the pile design capacity in these areas to 550 kN and to install more piles (at no extra cost to the State). This proposal was found acceptable, and so piles typically varied from 15 to 18 m deep, although in one area they were taken deeper to avoid extra loading on existing subway tunnels.

All piles were loaded cyclically and incrementally in order to provide data on both elastic and permanent movements, basically in conformance with NYSDOT Provisions (1977). Results from 18 tests (9 non-production and 9 production piles) are summarized in table 18.

Regarding permanent movements, figures 21 and 22 summarize the performance of piles founded in the silty sands and clayey silts, respectively. Figures 23 and 24 summarize the creep data for the same groupings of piles.

As shown in table 18, the pile at Bent 27C was post-grouted after initial testing (figure 25). One may compare the original maximum achieved load of 1550 kN (permanent movement of 18 mm) to the subsequent, easily attained load of 1780 kN (5 mm of permanent movement). After post-grouting, the creep was 0.3 mm during the last 4 hours at 1780 kN. One lateral loading test to 90 kN gave a total deflection of 19 mm and a permanent deflection of 3.6 mm.

Difficult Geologic Conditions

Delaware River Bridge, New Jersey (Bruce, 1988, 1989)

Background. The I-78 dual highway was designed to cross the Delaware River between Pennsylvania and New Jersey (Warren County) on seven span, multigirder bridges. Generally, foundations on the Pennsylvania side incorporated driven H-piles, whereas the river piers and the New Jersey piers were intended to be founded on solid rock. This proved to be practical except for pier E-6 on the eastbound structure, since the foreseen excavation for the footing to the planned elevation could not find rock head. Further excavation to an elevation 5 to 6 m below revealed only random rock thicknesses and a highly irregular bedrock surface. The excavation was filled with lean mix concrete and the foundation design was reassessed.

Various options were reviewed, including:

- Enlarged spread footings.
- H-piles in predrilled holes.
- Elimination of the pier.
- Relocation of the pier.
- Deep bored piling.

Only the last option proved feasible and two alternates were considered:

- Six 914-mm-diameter caissons, each with a service load of 3200 kN.
- 24 Type 1B micropiles, each with a nominal service load of 890 kN (allowing an 11 percent redundancy to compensate for the highly variable rock conditions).

Bids were solicited for each option, but due to the extremely onerous geological and programming restraints, only one contractor for each responded. The bid for the 914-mm-diameter caissons was essentially cost plus with an estimated price of about $1 million. The fixed price offer for the micropiles was less than half that figure. The owner, therefore, decided on the latter option on the grounds of cost, installation time, and the ability to demonstrate the effectiveness of the system by a test pile installed in advance. In addition, the action of micropiles in transferring load by skin friction, as opposed to end bearing, eliminated the possibility of pile failure by punching through into any soft underbed immediately under pile toe level.

Site and Ground Conditions. The bedrock was a Cambro-Ordovician dolomitic limestone referred to locally as the Allentown Limestone. It proved to be moderately to highly fissured, cherty, and very susceptible to karstic weathering. Major clay-filled beds were intersected even more than 30 m below the surface. For example, 15 m of soft brown silty clay lay below elevation 32 m at the location of pile 24. Dipping 55 degrees to the southeast, the rock mass proved highly variable laterally and vertically. The shape of the solid bedrock surface, as revealed in site investigation holes and by the subsequent pile drilling, is shown in figure 26.

Design. The owner's design regulations permitted:

- Maximum average rock-grout bond at service load of 0.35 MPa.
- Maximum allowable reinforcement steel stress at service load equivalent to 45 percent f_y.

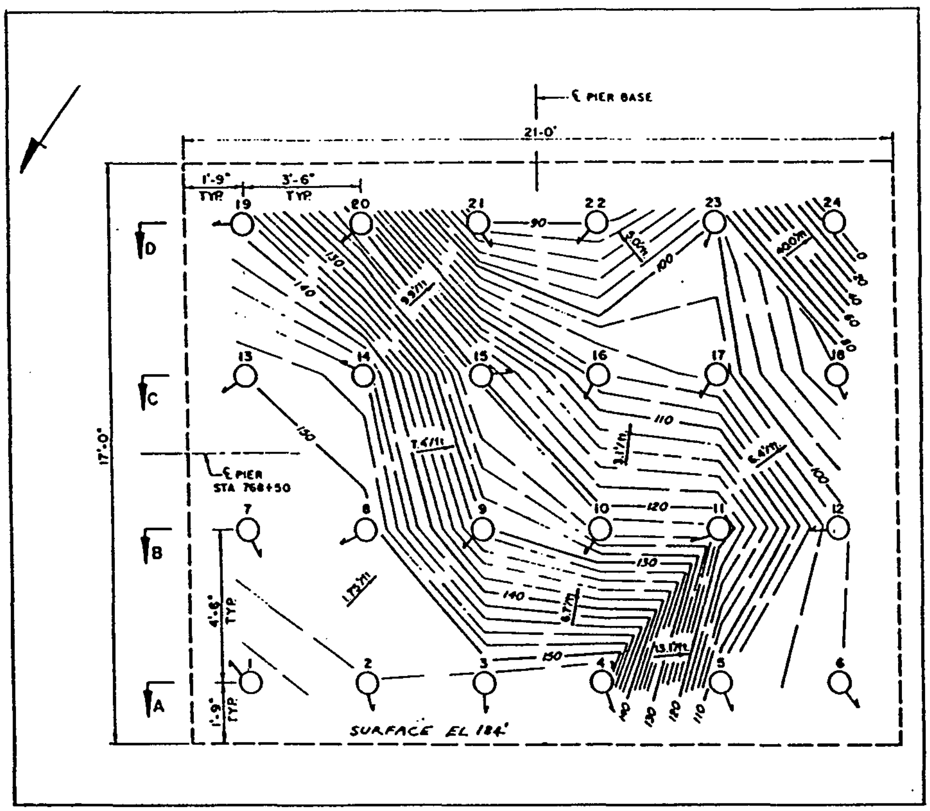
These factors led to the selection of:

- A bond zone 216 mm in diameter and 4.6 m long in competent rock.
- Use of 379-MPa low-alloy steel pipe 178 mm in diameter and a wall thickness of 10.4 mm as the pile reinforcement.

Recognizing that the rock was likely to be very variable, provision was made to allow the 4.6-m-long bond zone to not necessarily be continuous. In most piles, this was subject to the following restrictions:

- The lower part of the zone was to contain at least 3 m of continuous sound rock.
- Soft interbeds were to be less than 0.9 m thick.
- A zone of acceptable load-bearing rock was to be at least 1.5 m thick.
- Regrouting and redrilling of interbeds within the overall bond zone was to be undertaken.

Piles 1, 6, 17, 18, 19, 23, and 24 were required to have a continuous 4.5-m-long bond zone for reasons of extra conservatism.



1 in = 25.4 mm
1 ft = 0.305 m

Figure 26. Interpreted bedrock isopachs, Warren County, New Jersey. [Arrows show direction of drill hole deviation (Bruce, 1988, 1989).]

Construction. Drilling and installation proceeded as follows:

- Install 273-mm-diameter casing through the backfill and socket into the concrete of the cap.
- Drill with 254-mm-diameter down-the-hole hammer through the concrete footing.
- Install 244-mm-diameter casing through the less competent upper horizons (normally 9 to 14 m thick). Survey linearity and grout in place.
- Drill 216-mm-diameter hole by hammer or rotary methods to ensure minimum of 4.6-m bond zone as described above.
- Flush hole and install 178-mm-diameter reinforcing pipe. Survey for verticality, with not more than 2 percent deviation allowed.
- Tremie grout hole pile and pressurize to 0.35 MPa.

Verification of each pile alignment was made through the use of a single shot direction survey instrument. Each pile was surveyed at the top, bottom, and mid-depth. The results are shown in table 19 and indicate that every pile fell within the criteria, with most being within 1 percent deviation.

Grout was mixed in a colloidal mixer and injected by Moyno pump. A neat Type II mix with a w/c of 0.50 was used, providing 3-day crushing strengths greater than 25 MPa.

Throughout construction, the very adverse geological conditions posed major drilling problems. These were resolved, at length, by repeated pregrouting and redrilling. Figure 27 summarizes the actual total drilled lengths.

Regarding the anticipated caisson tip elevations, also shown in figure 27, these would have been in all cases shorter than subsequently proved necessary to found the micropiles safely. Poor or voided rock was consistently found below these anticipated elevations, further supporting the decision to use micropiles.

The total drilled length of 585 linear meters corresponded to the total foreseen quantity of 521 linear meters. Variations from 13.1 m less to 9.1 m more, with respect to foreseen lengths, were recorded on individual piles, highlighting the lateral variability of the rock. Overall, a volume of grout equivalent to four times the nominal hole volume drilled was injected, much of this was consumed in the zone above rock head during pregrouting operations. The level of maximum takes corresponded with groundwater level.

Testing and Performance. A separate test pile, 9.1 m long with only 1.62 m of bond, was load tested in accordance with ASTM D1143 to 1824 kN using rock anchors as reaction. This particular short bond length was selected because at test load, the average grout/rock and grout/steel bond values would be 2.1 and 1.7 MPa, respectively – both considered to be at or near ultimate values. An outer sleeve of PVC pipe extending to the top of the rock socket ensured load transfer only in the socket. A 152-mm-thick wooden plug was attached to the bottom of the steel pipe to ensure that no load could be transferred in end bearing.

Table 19. Borehole deviation data on micropile holes, Warren County, New Jersey (Bruce, 1988, 1989).

Pile #	Length (m)	Actual Drift (mm)	Ratio Actual to Allowable (based on 2% deviation) (%)	Direction of Drift
1	13.4	116.8	44	S 50° E
2	14.3	87.6	31	N 45° W
3	14.0	155.7	57	N 30° W
4	13.7	59.9	22	N 85° W
5	28.3	49.5	9	N 77° W
6	29.6	205.7	35	S 85° W
7	14.9	155.2	53	N 57° W
8	14.9	102.6	35	N 05° E
9	20.4	208.5	70	N 18° W
10	23.5	245.9	52	N 13° W
11	23.5	143.2	30	N 14° E
12	29.6	258.1	44	N 32° E
13	14.9	130.3	43	N 85° W
14	15.8	248.9	78	N 75° E
15	24.4	127.8	26	S 04° W
16	28.3	345.4	61	N 20° W
17	29.3	50.8	9	S 04° W
18	32.6	113.0	17	N 70° W
19	18.3	150.1	41	N 45° E
20	24.4	297.9	61	N 10° W
21	32.9	284.5	44	S 61° W
22	33.2	231.9	34	S 12° E
23	29.9	260.6	44	N 12° W
24	61.0	361.7	35	-(6.7 m above base)
			Average = 40%, i.e., average deviation of<1%	

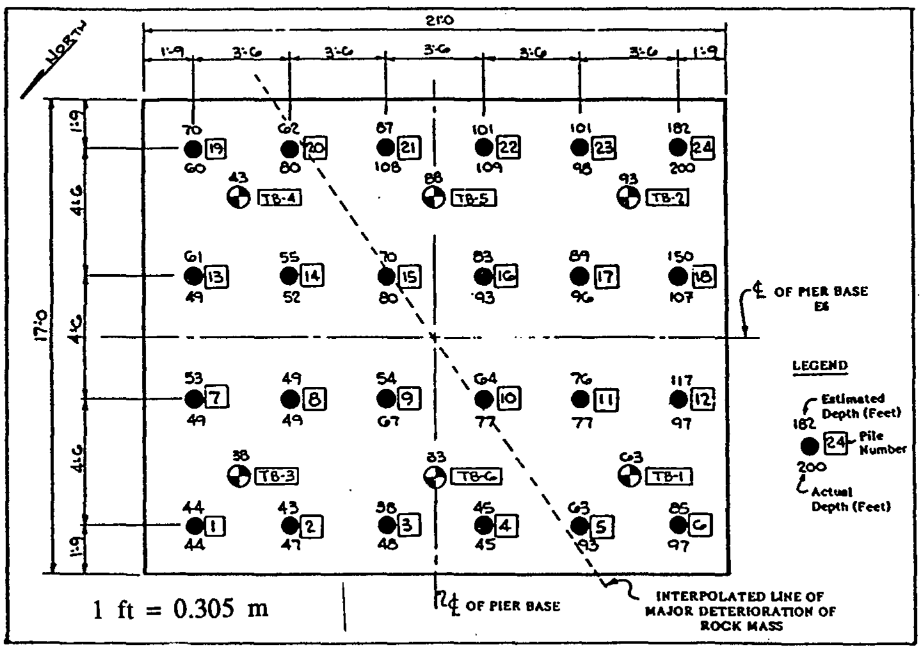


Figure 27. Actual micropile lengths, with anticipated caisson depths shown for comparison, Warren County, New Jersey (Bruce, 1988, 1989).

The data are presented in figure 28. The total movements at each successive cycle to test load were 9.32 and 9.47 mm, respectively. Creep of about 0.3 mm was recorded over 60 minutes hold at this load. The permanent movement after this operation was 1.8 mm.

The next day, testing was continued to higher levels. However, at 1990 kN, the material of the upper casing began to fail. Until that point, the pile was performing exactly as it had during the previous testing sequences. Total movement was 9.42 mm at 1913 kN and 11.5 mm at 1990 kN.

During installation of the reinforcing pipe in the last and deepest pile (No. 24), a threaded joint parted and a 40-m length of pipe fell into the 61-m-deep hole. Borehole TV revealed the casing to be further separated 9.1 m above the bottom of the hole, due to the impact. After various unsuccessful attempts at casing recovery, it was decided to grout the pile, having previously suspended a 6.1-m-long, 114-mm-diameter, 1034-MPa steel pin, with centralizers, from 19 to 25 m below the top. The intention of this pin was to ensure effective load transfer across the upper discontinuity. A very rigorous extended load test was then executed to 1512 kN. The performance of this rehabilitated pile proved excellent: the total movement was 4.75 mm with 0.25 mm of creep in 24 hours, and a permanent movement of 0.23 mm. It was judged to be capable of safely performing its function in service.

The bridge is now complete and pier E-6 has performed normally.

Environmentally Sensitive Areas

Brookgreen Gardens, South Carolina (Bruce, 1988, 1989)

Background. The 1.2-million-m^2 Brookgreen Gardens at Murrell's Inlet, South Carolina, was founded as an institution funded by private donations for the preservation of the flora and fauna of that particular part of the United States. A scheme was conceived to construct a large and almost invisible aviary in the cypress swamp -- a purely environmental structure enveloping the existing trees, but leaving nature untouched so that the public could view the local bird life in its natural habitat. The design envisaged a structure greater than 27 m high, octagonal in shape, with a diameter of 60 m (figure 29). It was designed to withstand hurricane winds, ice storms, and a corrosive marine atmosphere. It had to be constructed in a swamp without causing damage to the existing trees or changing the character of the swamp, within a tight budget and in a very short period of time.

The walls of the aviary were made of a special polyester safety netting, suspended off nine slender aluminum poles. Each pole was supported by two cable stays, and prestressed soil anchors were chosen to provide ground fixity for these. These poles also exerted compressive forces on the ground, while dynamic analyses indicated that the pole foundations would also be subjected to significant horizontal forces under certain circumstances. Conventional piling and spread footings were unacceptable because of the nature of the site and the potential disturbance to the existing flora. Micropiles were the logical choice, particularly since the equipment (and techniques) needed for the soil anchors could be used economically for the piles also.

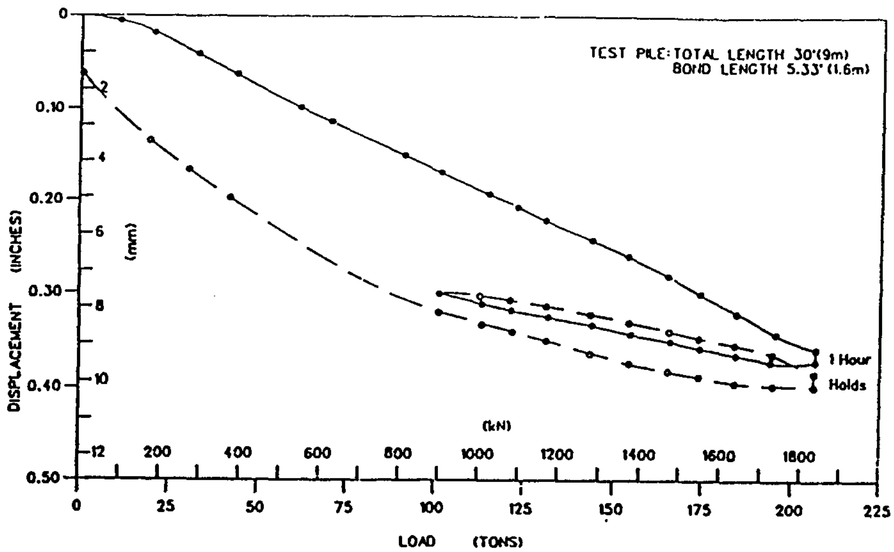


Figure 28. Load/movement data, test pile at Pier E-6, I-78 bridge, Warren County, New Jersey (Bruce, 1988, 1989).

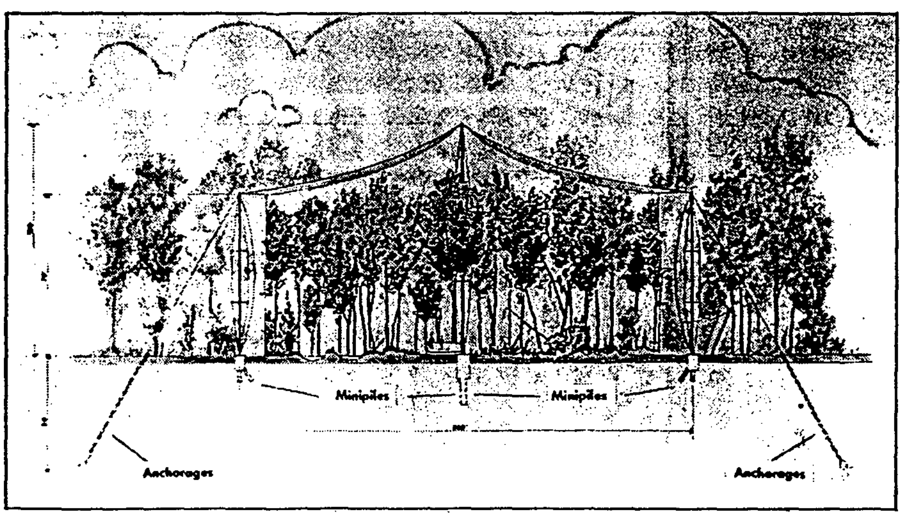


Figure 29. Artist's impression of the aviary in the cypress swamp, showing micropiles underpinning footings and ground anchors resisting tensile cable loads, Brookgreen Gardens, South Carolina (Bruce, 1988, 1989).

Site and ground conditions. Timber planking had to be set as temporary access into the swamp from the main road since the underfoot conditions were so soft. Site investigation holes showed grey/brown loose to fine sands under the cypress roots, overlying dense medium and coarse light-brown sands with shells. Water level coincided with the ground level.

Design. Cable tensile forces were reacted by bar anchors 18 m deep at 45 degrees to the horizontal. The maximum working load of each of these eight anchors was 400 kN. Eight 80-kN wall tie-down anchors were also constructed in similar fashion at 55 degrees to the horizontal.

Structural analyses determined the compressive/horizontal forces and their resolution. Each mast was supported on a pile cap bearing on three Type 1B micropiles. Two of these piles were at 45 degrees to the horizontal and were also aligned in plan towards the center and 45 degrees on either side of the radial line between the center mast and the peripheral mast. The third was vertical. This layout satisfied the maximum load conditions that could occur, defined as 490-kN vertical load at each mast pile, 190-kN horizontal load towards the center mast, and 100-kN horizontal load towards the adjacent peripheral mast. The external and internal load-carrying capacities of the piles were determined in the standard manner.

Construction. The micropiles were constructed in a similar manner to the tie-down anchors, except that a 10-m-long pressure-grouted zone was used for the angled piles, and a 3-m zone was used for the verticals. In each case, a 6-m length of 127-mm-diameter steel casing of 8-mm wall thickness was left in place for the length above the pressure-grouted zone. Pressures of up to 0.9 MPa were used with neat Type 1 cement grout with a w/c of 0.50. Each pile was further reinforced full length with one 28-mm rebar with centralizers. Four 13-mm hooked rebars were also set in the fresh grout at the top of each pile. The three micropiles were then connected into a pile cap that consisted of a piece of 1.1-m-diameter steel tubing set around the projecting steel pile casings and reinforcing bars. Further layers of reinforcing steel mesh were also placed, and the cap was filled with 28-MPa concrete. Bolts used to anchor the articulated mast bases were held by templates and concrete.

The 130-kN center mast pile consisted of a variation of the standard micropile in that the total length of the single pile required was 6 m, of which the top 3 m was a piece of 460-mm-diameter pile casing. This arrangement was chosen since the maximum vertical load calculated was much less than that for the peripheral piles and no horizontal forces were anticipated.

A wide-track, diesel-hydraulic drilling rig was used, equipped with a rotary duplex drilling system with water flush. The return flush and cuttings were ponded carefully and discarded. Special precautions were likewise taken with the mixer-pump unit and ancillary equipment.

At every stage of the operations, the curator of the gardens was consulted as to the impact of each construction step on the enclosed area.

Performance. No pile testing was conducted given the high degree of design conservatism. All the objectives of the project were achieved. The scheme – an engineering joint venture between the architect and the contractor – received first prize in that year's New York Association of Consulting Engineers Engineering Excellence Competition. Judgment was based on project significance, complexity, uniqueness, client needs, budget, originality, value to the profession, and timeliness.

Hong Kong Country Club (Bruce and Yeung, 1983)

Background. The prestigious Hong Kong Country Club overlooks Deep Water Bay on the south of Hong Kong island. To maintain the preeminence of its social and sporting facilities, substantial redevelopment was undertaken, a major part of which involved the extension of the main building (figure 30).

The existing structure sits on miscellaneous fills, founded on rock via 500-mm-diameter concrete piles. Originally, a cantilever extension from the existing structure was proposed to accommodate the addition. However, the additional loading from this extension threatened to overload the existing foundation, and its construction would have necessitated disruption to club operations. A piling solution was therefore determined, but which had to meet the following restrictions:

1. The construction technique had to cause the least nuisance and noise to the club membership and the adjacent Ocean Park entertainment facility containing an extensive open-air dolphinarium.

2. Equipment had to operate in very restricted access conditions, yet be capable of installing substantial piles to considerable depths.

3. Minimum movement was a technical requirement of the piling system.

4. A short program schedule was stipulated (a limit of 74 calendar days was set).

5. The piling system had to cause minimal disturbance to the ground and existing structures.

Hand-dug caissons, then widely used in this region, were rejected based on the above requirements and the potential dangers associated with water table drawdown. After careful consideration, a Type 1A micropiling system was adopted that met all the technical, logistical, scheduling, and economic constraints of the project. This was among the first applications of micropiling in the colony.

Ground Conditions. The major ground conditions are summarized in table 20. Rock head levels, defined as slightly weathered volcanics of a minimum 85 percent core recovery, were predicted as shown in figure 31(a) and subsequently served as the basis for pile-length estimates. For comparison, figure 31(b) shows the rock head levels recorded during piling. In general, the rock head dipped southwest, giving foreseen rock head depths of 11 to 23 m [figures 31(c) and 31(d)].

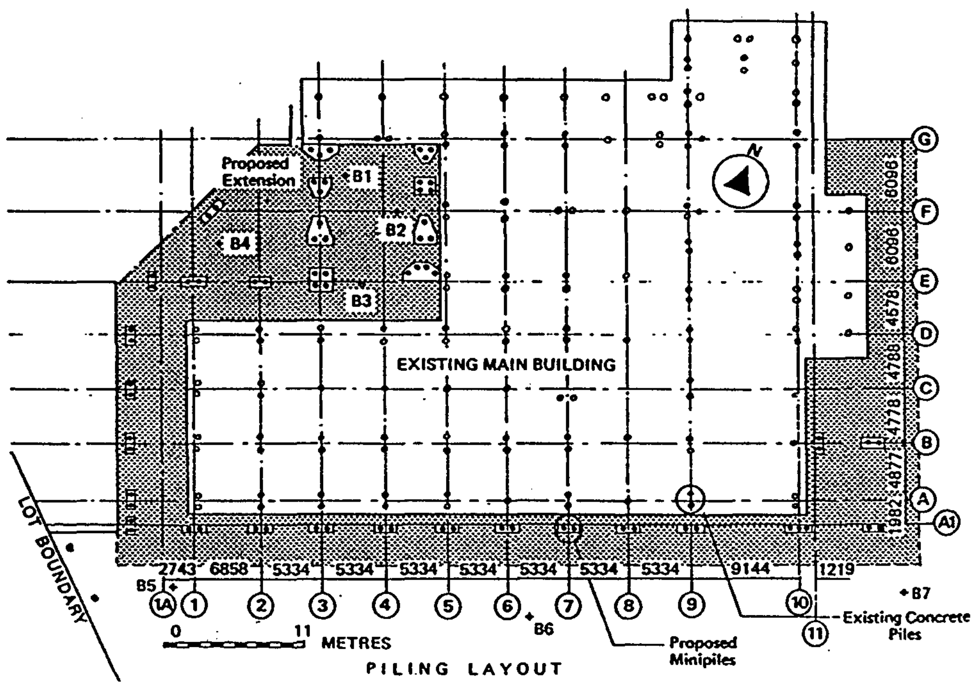


Figure 30. Plan showing the extension of the club's main building, Hong Kong Country Club (Bruce and Yeung, 1983).

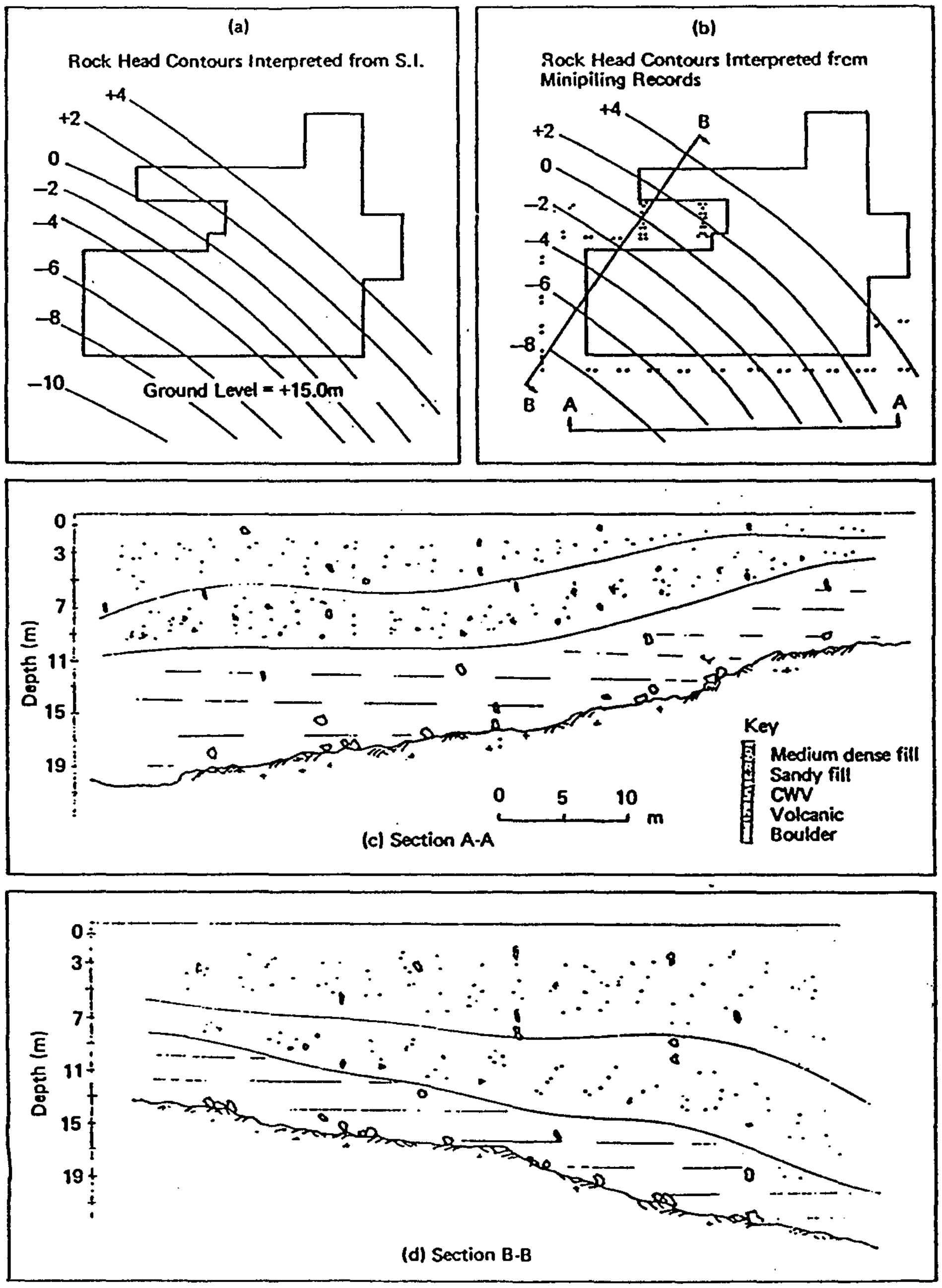


Figure 31. Site investigation interpretation, Hong Kong Country Club (Bruce and Yeung, 1983).

Design. Practical arrangements of the 34 new columns, and their associated pile groups, yielded a standard individual pile service load of 550 kN. Columns were supported on pile caps spanning from one to four micropiles. Contemporary Hong Kong building ordinances required that for internal design, the pile had to be considered either as a concrete pile or a steel pile: exploitation of composite action was not allowed. Since design on the basis of a concrete pile would have necessitated diameters much larger than standard for micropiles, the internal design assumed all the load being carried on the steel reinforcement.

For ease of handling, a single element, as opposed to a large number of bars, was considered. For the 350-MPa steel pipe of outside diameter 140 mm and a wall thickness of 9 mm, the theoretical permissible service load, using the agreed service stress of 42.5 percent f_y, was 561 kN.

Corresponding to this casing size, a borehole diameter of 220 mm was judged to be adequate to ensure a conservative annulus of grout cover to the steel.

Since project specifications required that structural movements be absolutely minimized, piles were designed to be founded in solid rock. Furthermore, since the density of the fill and residual soil was low, and the thickness of the weathered volcanics variable (table 20), contributions to load transfer by any of these overburden materials could not be relied upon, further supporting the decision to socket the piles firmly into rock.

In considering the ultimate bond between rock and grout (τ_{ult}), the following relationship was used from anchor practice (Littlejohn and Bruce, 1977):

$$\tau_{ult} = \frac{\text{Unconfined Compressive Strength (UCS) of rock}}{10}$$

(for values of UCS up to 42 MPa, and grout of comparative strength)

Assuming the volcanic bedrock to be at least this strong, the above relationship yielded a service bond value of 1.4 MPa at a factor of safety of 3. Thus, for a 561-kN service load, the required embedment length was calculated as 580 mm. The site investigation showed that boulders up to 1.0 m in diameter could be expected, particularly just above rock head. To safeguard against premature hole termination on a boulder, it was therefore established that drilling would proceed to a depth of 2.0 m into sound rock.

Construction. Drilling was conducted by diesel-hydraulic track rigs. Rotary duplex equipment with water flush was used to drill and case to rock head, while drilling in the hard rock was accomplished with down-the-hole hammers. Upon reaching full depth, the drill rods were extracted and the hole thoroughly flushed with water. Throughout, special care was exercised to observe the specified pile tolerances: $\pm$50 mm in plan and a verticality deviation not exceeding 1 in 75.

The welded pipe lengths were then placed. Peripheral spacer bars were used to ensure pipe verticality and concentricity. The chosen grout mix design comprised a 1:1 sand/cement mix of w/c ratio 0.55, with plasticizer/retarder to improve pumpability and ease the extraction of the drill casing. Grout was introduced into the hole from the base upwards via a tremie tube. Once good quality grout was observed to emerge at the top, the drill casing was withdrawn while maintaining the grout level at ground surface to avoid "necking" of the pile in the soft ground.

Overall, the 76 piles required a total of 1337 linear meters of drilling (average 17.6 m per hole; range 11.6 to 23.8 m). Grout consumptions ranged from 2.1 to 16.4 times the calculated hole volume, and averaged greater than 3.6, highlighting the relatively high porosity of the fill in particular.

Testing and Performance. In advance of production piling, a test pile was installed to demonstrate and test construction procedures, and to provide performance data as a check on design parameters. In addition, at the conclusion of the piling works, one of the production piles was selected for proof testing to twice service load.

Pre-Production Test Pile. The test pile was installed, by the method described above, at a location southwest of the main building. The summary piling record is shown in table 21. At the time of installation, the special high-yield steel of the production piles was not available, so the following alternate was used: steel casing of outside diameter = 141.3 mm, wall thickness = 9.6 mm, and f_y = 241 MPa.

Applying the specified steel service stress limit of 42.5 percent, the pile was therefore rated as having a service load of 403 kN.

Due to restricted access, reaction for the applied load was supplied by two inclined rock anchors. Load was applied in increments of 61 kN and held at each increment for 10 minutes to record any creep. The loading proceeded as follows:

- Load to service load and lock-off for 24 hours.
- Unload, reload to 1.5 times service load, and lock-off for 24 hours.
- Unload, reload to 2 times service load, and lock-off for 24 hours.
- Unload, reloading continued to 2.3 times service load when a temporary problem with loading system occurred, requiring locking off at this load for 24 hours.
- Unload, then reloading continued to 2.7 times service load at which point the capacity of the loading system was reached.

The load-movement curve indicated satisfactory performance and linear behavior, with the final test load of 1090 kN corresponding to 115 percent f_y. Table 22 summarizes the major features. A creep of 0.47 mm was recorded in the 48 hours during which this load was held. Approximately 80 percent of this amount of creep was recorded within the first 12 hours of load hold.

Production Pile Test. Pile NC 7b (table 23) was selected for testing. Load was applied in increments of 50 percent service load (280 kN), with 2-minute intervals. The pile was loaded cyclically, but continuously, in this manner before being held at twice service load (1122 kN) for 72 hours. Load-movement behavior is shown in figure 32 and creep records are shown in figure 33.

Table 20. Summary of site investigation borehole logs B1 to B7, Hong Kong Country Club (see figure 30 for borehole locations) (Bruce and Yeung, 1983).

Ground	Description	Notes
5 to 14m Overlying	Medium dense grey-yellow silty to coarse sand with bricks, gravel, and cobbles (FILL) Occasional fresh rock boulders	N = 6 to 70, variable, but often below 20
0 to 2.4m Overlying	Firm dense yellow-brown clayey SILT/SAND (RESIDUAL SOIL)	N = 16 to 94 Present in B5 & B6
1.4 to 11.7m Overlying	HIGHLY WEATHERED dense grey-yellow VOLCANICS with rock fragments: silty sand and gravel	N = 35 to 200 but usually 200
Bedrock	Moderately strong– Very strong green-grey fine-grained moderately – slightly weathered VOLCANIC ROCK with closely spaced stained joints	Core recovery 87-100% RQD typically 20-50%, but often 0%

Table 21. Preproduction test pile construction summary, Hong Kong Country Club (Bruce and Yeung, 1983).

Summary Drill Log

Depth (m)	Strata	Average Penetration (min/m)	Notes
0-15.5	Fill	3.5	Soft sandy fill, occasional small boulder (< 1m). No flush return.
15.5-18.9	Completely Weathered Volcanics	8.3	Firmer ground. No flush return.
18.9-21.9	Fresh Volcanics	13	Very hard rock. No flush return.

Installation

Pipe O.D. = 141.3mm, I.D. = 122.2mm, Length = 21.9m
fy = 241N/mm² (Test Pile only)

Grouting

OPC:	2,430kg	Sand 2,430kg
Colplus:	7.49 litres	w/c = 0.55
Av. UCS:	40.3N/mm² (21 days)	
	45.3N/mm² (28 days)	

Table 22. Test pile performance summary, Hong Kong Country Club (Bruce and Yeung, 1983).

Load	Total Settlement (mm)	Permanent Settlement (mm)
WL (403kN)	2.75	0.25
2WL (806kN)	6.00	0.75
2.7WL (1,090kN)	14.50	5.45

Table 23. Production pile NC 7b construction summary, Hong Kong Country Club (Bruce and Yeung, 1983).

Summary Drill Log Depth (m)	Strata	Average Penetration (min/m)	Notes
0-3.0	Fill	9	Soft, sandy. No flush return.
3.0-3.5	Boulder	65	Very hard erratic.
3.5-11.5	Fill	10	Soft and sandy with occasional cobbles. Little flush return.
11.5-11.9	Completely Weathered Volcanics	10	Firmer. More flush return.
11.9-13.9	Fresh Volcanics	70	Very hard rock.

Installation

Pipe O.D. = 139.7, I.D. = 121.3mm, Length = 14.7m
fy > 350N/mm² (Typical of production piles)

Grouting

OPC: 3,420kg Sand: 3,420kg
Admixture: 4.86 litres w/c: 0.55
Typical UCS: 20N/mm² (7 days)
35N/mm² (28 days)

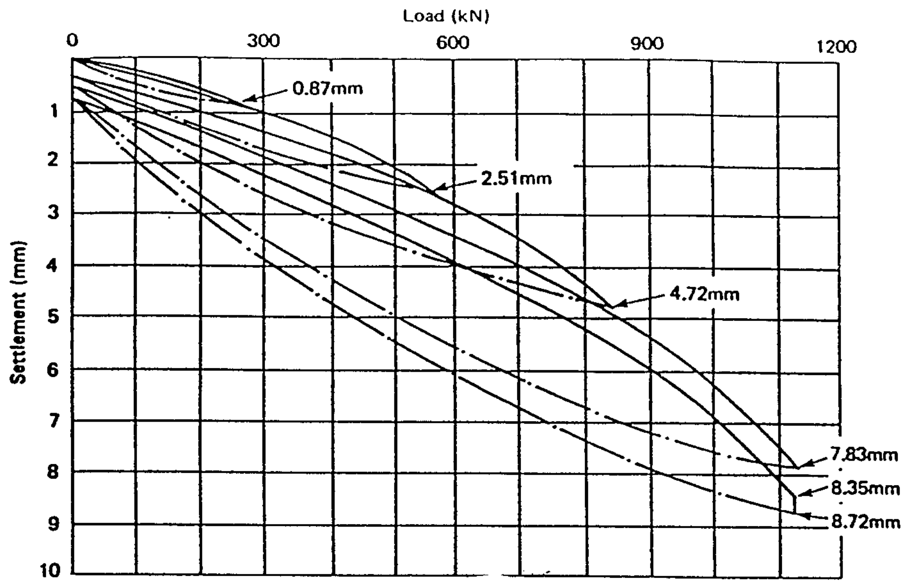


Figure 32a. Load/movement data, Production Pile NC 7b, Hong Kong Country Club (Bruce and Yeung, 1983).

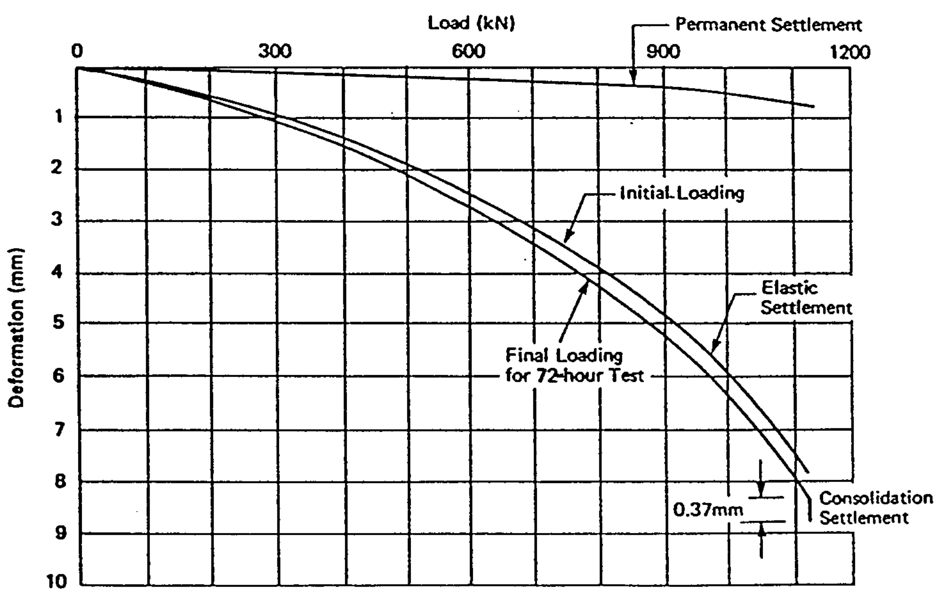


Figure 32 b. Load/movement resolution curves, Production Pile NC 7b, Hong Kong Country Club (Bruce and Yeung, 1983).

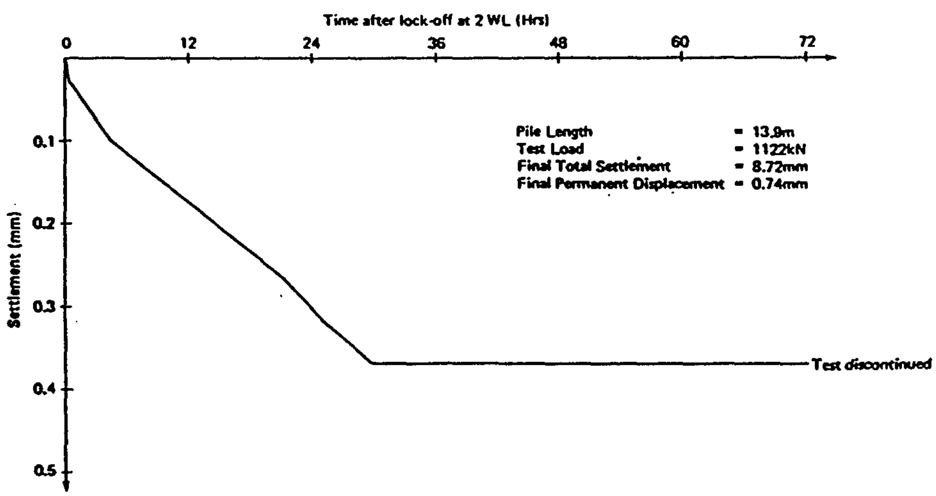


Figure 33. Creep test data, production pile NC 7b, Hong Kong Country Club (Bruce and Yeung, 1983).

The target performance of the production pile was designated to be based on composite action within the pile. Calculations gave a total theoretical elastic deformation of 8.7 to 15.0 mm (allowing for possible variations between modular ratios and E-values for the grout) for this pile length (14.7 m). A permanent movement target of 4 mm was also set. This analysis was therefore consistent with the total settlement criterion of 15 mm specified in the building regulations. As shown in figure 33, the elastic and permanent movements at test load were 7.98 mm and 0.74 mm, respectively (the corresponding figures were 2.15 mm and 0.36 mm, respectively, at service load). Among other conclusions, it was therefore obvious that considerable load was being transferred in the fill, above the bond zone, underlining the high degree of design conservatism.

These small movements, together with the very low creep values recorded, permitted the engineering conclusion that the piles, as installed, would readily and safely fulfill their service requirements. The extended structure has since been in service for more than 15 years and has performed perfectly.

<u>Industrial Facility, Mobile, Alabama (Bruce et al., 1992)</u>

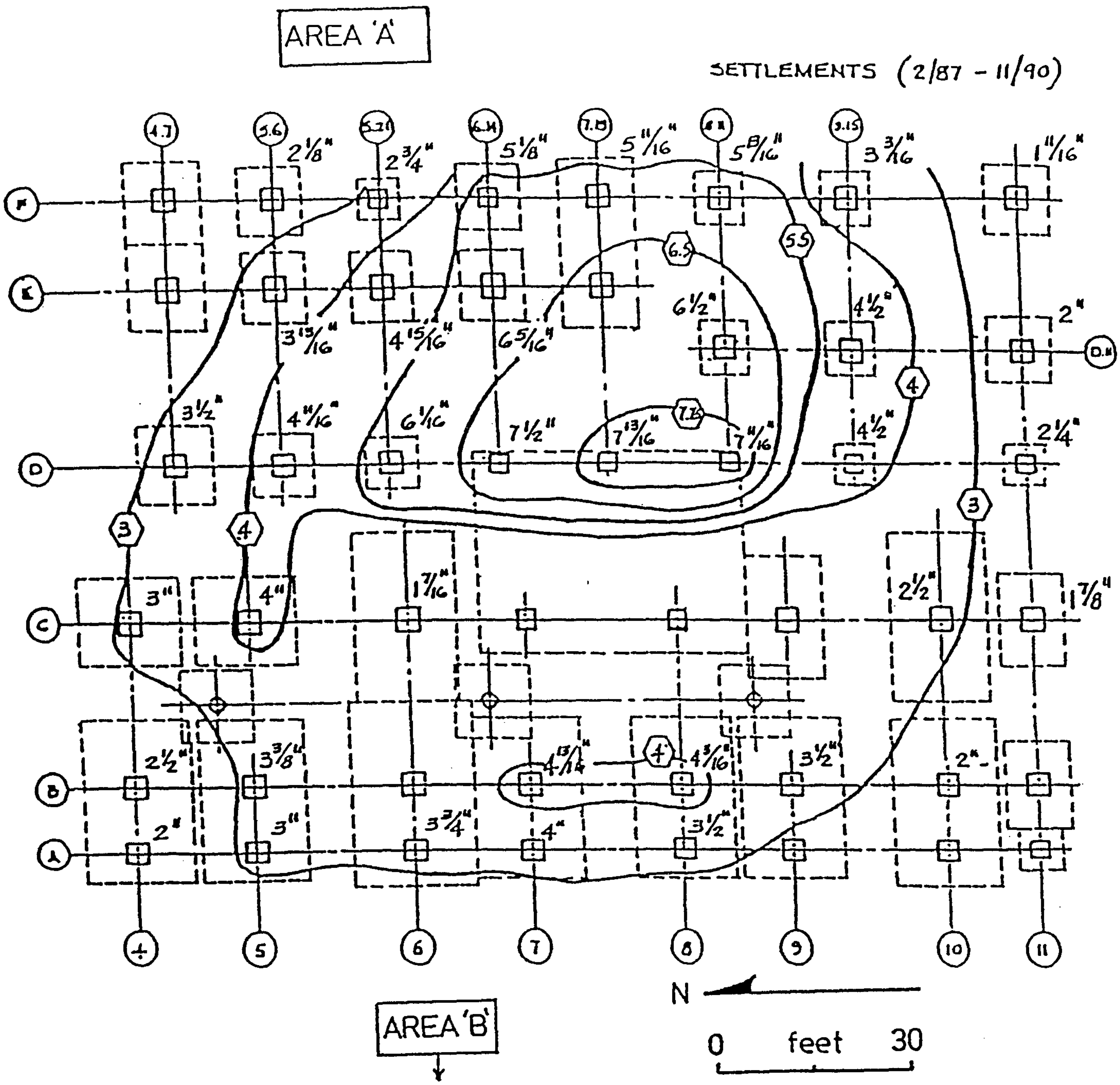
Background. In order to prevent continuing settlement of a caustic evaporator structure within their major chemical producing plant, the owners called for a contractor-designed underpinning system. The alternatives considered were:

1. Jet-grouted columns extending from the underside of the existing footings into the dense sands and gravels of an underlying bearing stratum.

2. Micropile support between the same limits.

3. Micropile support from the bearing stratum, but passing through the existing footings (approximately 3 m below the surface) with at-grade pile caps and welded steel connections. The existing footings would then be detached and isolated from the new pile cap and its supporting micropiles.

Careful analysis of the structural settlement data was a major determinant of the method. Average settlement rates of 0.1 to 4.5 mm per month per column had been recorded over the immediate 44-month period (figure 34), and this demanded that special attention be paid to the connections between the piles and the structure. In effect, damage to the new underpinning system could potentially have occurred if their attachment to the structure was made before they achieved full design strength. This concern ruled out Option 1 (jet grouting), despite its possible economic attractions. It was felt that the rate and total magnitude of the ongoing settlements would not permit proper, undisturbed setting of the "soilcrete," especially in its upper part immediately under the footing. In addition, it was also believed that the disturbance to the soil and its short-term removal, inherent in the jet-grout method, would likely increase the local settlement rates.



1 in = 25.4 mm
1 ft = 0.305 m

Figure 34. Cumulative structural settlements in period from January 1987 to November 1990, Mobile, Alabama (Bruce et al., 1992). [Test Pile areas shown.]

These fears were supported by field observations on remedial concrete works recently completed: a concrete dampener installed at one group of columns, although well reinforced and constructed, had experienced significant distress due to settlement before its concrete had reached full strength. In another case, new concrete placed at another column had to be repaired by epoxy resin injection for similar reasons.

The second option was also discounted. To achieve the foundation connection, difficult and costly excavation and shoring was anticipated to ensure safe access to the footings. In addition, groundwater or process water in-flows would have required continuous dewatering.

Option 3 – micropiles with new pile caps and welded connections between pile cap and structure column – was therefore chosen and developed.

Site and Ground Conditions. The working conditions were extremely awkward, due to restricted headroom (1.8 m typical minimum, occasionally as little as 1.4 m), tight and difficult access, and very strict safety/procedural restrictions and regulations. The ambient weather conditions were typically very hot and humid.

A typical section through the site is shown in figure 35. The general stratigraphy was as follows:

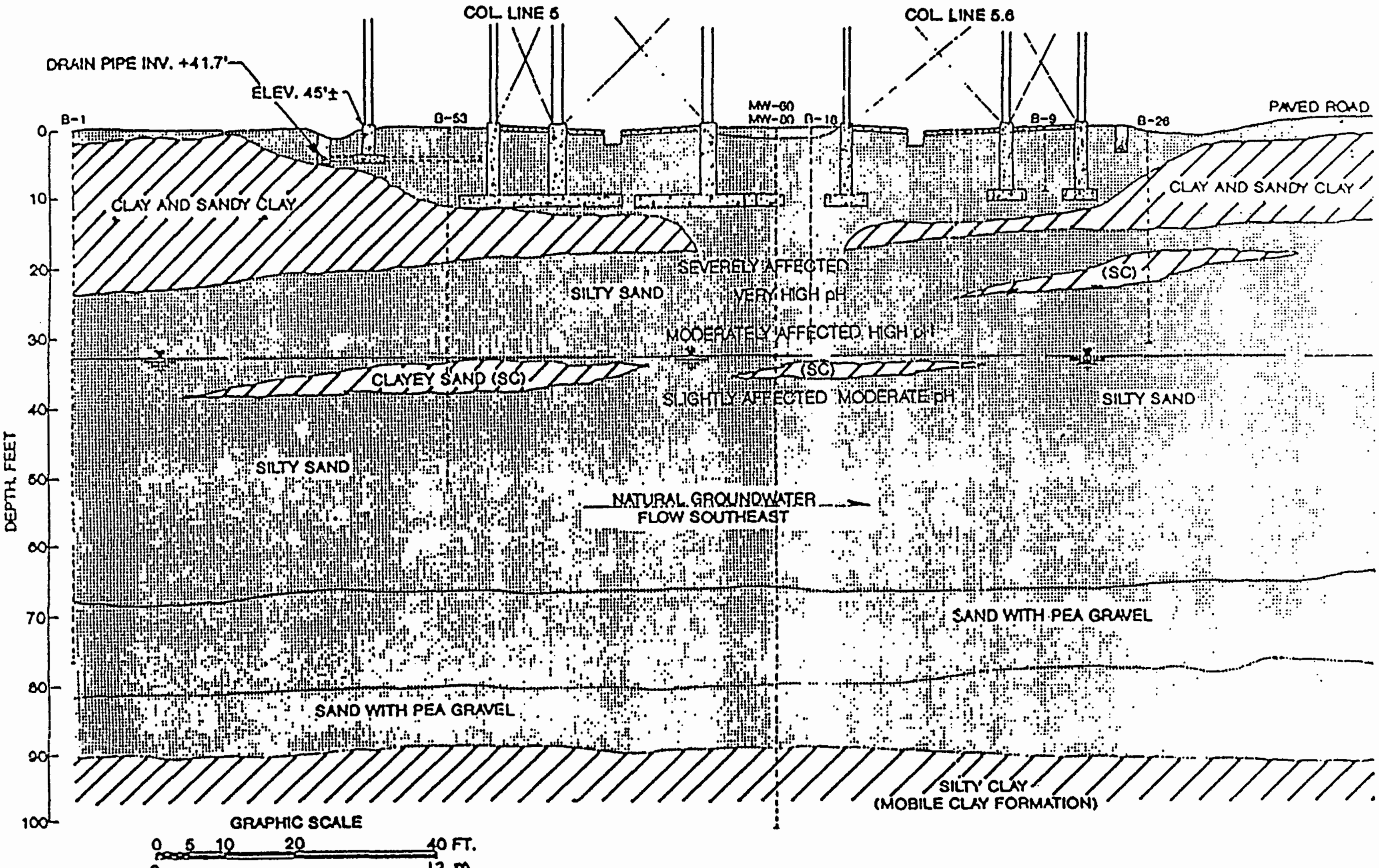
- From 2.4 m to 5.4 m: Fill – fine to medium, tan-orange silty sand beneath shell cover material. From 1 to 26 blows per 0.3 m.

- To 7 m depth: Gray clay, gray and orange silty clays, light brown sandy clay over fine to medium tan-orange and gray silty sands, from 2 to 30 blows, increasing with depth.

- From 7 m to 27 m depth: Fine to medium to coarse tan, brown sands, occasional clay lenses, and rounded quartz gravel beds. Typically single-sized in any given bed. Dense to very dense (12 to 100+ blows, typically more than 40 below 18 m). Indication of more angular gravels below 19 m.

The sands and gravels of this founding horizon were extremely permeable and the groundwater was within 1.5 m of the surface, although this may have been perched, with the main table 8 to 9 m down. The water was chemically contaminated, with pH values being very high in the upper 6 m.

Design. The loads exerted by the 60 existing columns varied from 108 to 3630 kN. As described below, a standard micropile of 900-kN service load was selected throughout. When allowing for the geometries and structural symmetries of the new footings, each new footing was supported on between 1 and 6 piles, with all but 16 of the footings having only 1 or 2 piles. Typical arrangements are illustrated in figure 36, showing a total of 123 individual micropiles.



1 ft = 0.305 m

Figure 35. Typical geological section, looking north, Mobile, Alabama (Bruce et al., 1992).

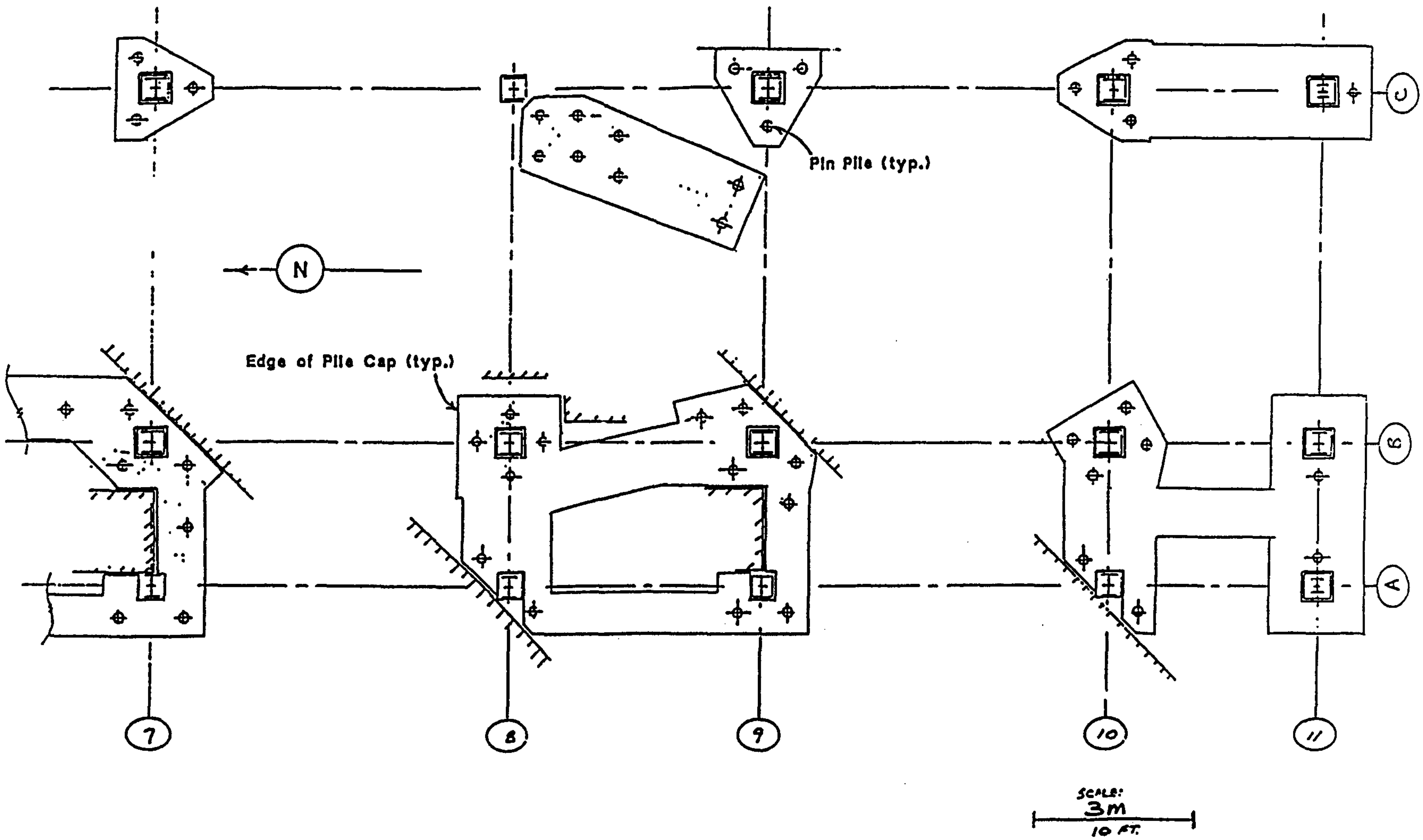
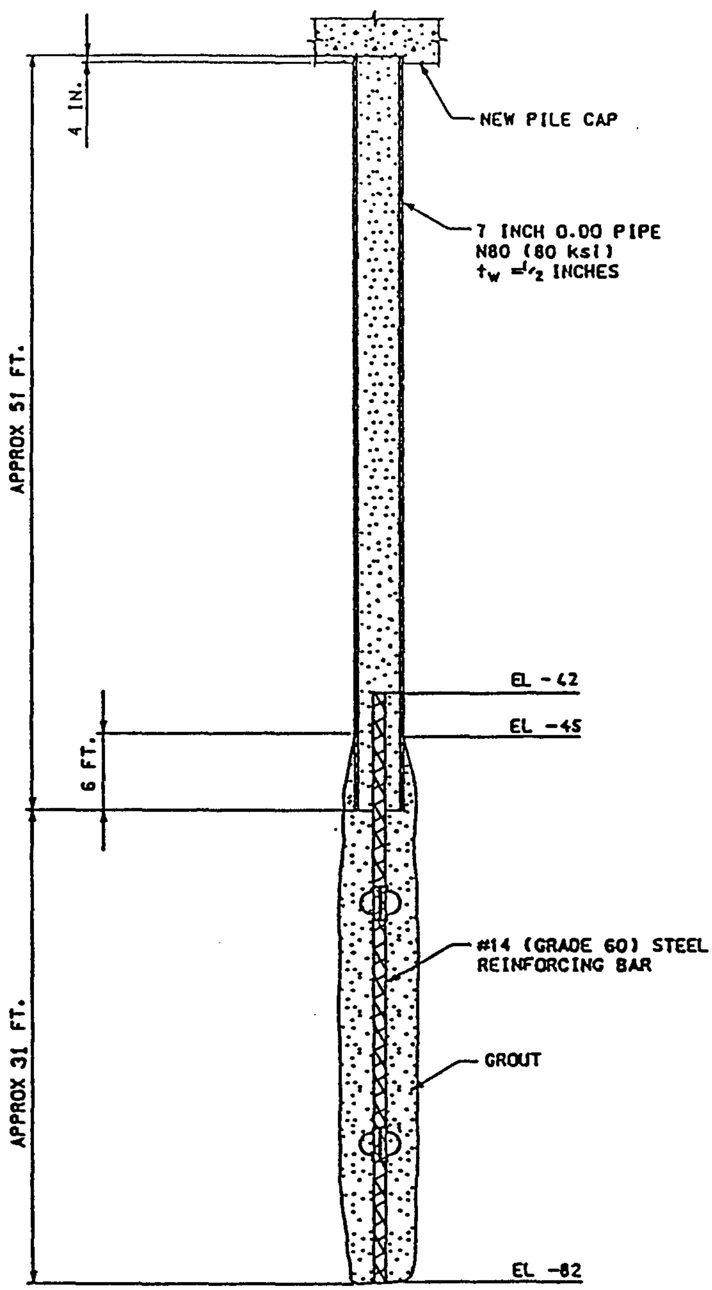


Figure 36. Example of typical micropile and new cap arrangement, Mobile, Alabama (Bruce et al., 1992).



1 in = 25.4 mm
1 ft = 0.305 m

Figure 37. Standard micropile configuration, Mobile, Alabama (Bruce et al., 1992).

The standard pile configuration is shown in figure 37, and the more significant design details and assumptions were as follows:

- Casing: 13-mm wall, 550-MPa yield strength, with allowable stress of 40 percent f_y.
- Rebar (bond zone only). Single 44-mm-diameter bar, 410 MPa, allowable stress of 50 percent f_y.
- Grout: 28-MPa UCS, allowable stress of 33 percent f_c.
- For the bond length calculation, the allowable bond stress at service load was (by convention) calculated as the average grouting pressure times tan ø (soil). Assuming an average grouting pressure of 0.45 MPa, an effective bond zone diameter of 300 mm, ø = 32 degrees for the sands and gravels, and a factor of safety of 2, a bond zone length of 7.5 m was chosen.

At design load of 900 kN, the estimated elastic movement was 8 mm and estimated total movement was 13 mm. From data on the historical rate of settlement, it was estimated that these Type 1B micropiles would be fully loaded to service load within 1 month of being connected to the structure.

It was also necessary to accommodate the possible effects of negative skin friction (or downdrag), and thus the upper 14 m of the casing was coated with a low-friction polyurethane resin. This coating also provided corrosion protection to the casing against the caustic environment in the upper strata. Calculations indicated that the maximum possible downdrag would be 130 kN per pile. Consideration of the redundancy inherent at each pile cap as a result of "rounding up" the numbers of piles provided, confirmed that this extra load could be safely accommodated without danger of overstress or excessive movement.

Regarding other design implications, it was also decided to install a PVC pipe into the existing footing after penetration to act as a bond breaker between it and the micropile.

As shown in figure 37, the pile head was to be encased in the new reinforced concrete pile caps (1.2 m thick, and poured around the existing pedestals to the same elevation as the existing slabs). Then, the pile caps were to be attached to the existing columns with welded steel connections (figure 38). Due to the high structural settlement rates, this connection was to be made after the concrete reached its design strength (28 MPa).

Construction. The existing concrete slabs adjacent to each column were sawn and removed, and in areas of minimum headroom, shallow excavations were prepared and shored to allow the drill rig mast to be placed. Where the pile had to be drilled through an existing footing, an oversized hole was first cored or hammered through it to accommodate the PVC bond breaker. Special electro-hydraulic track rigs with "mast-off" capability were then used to

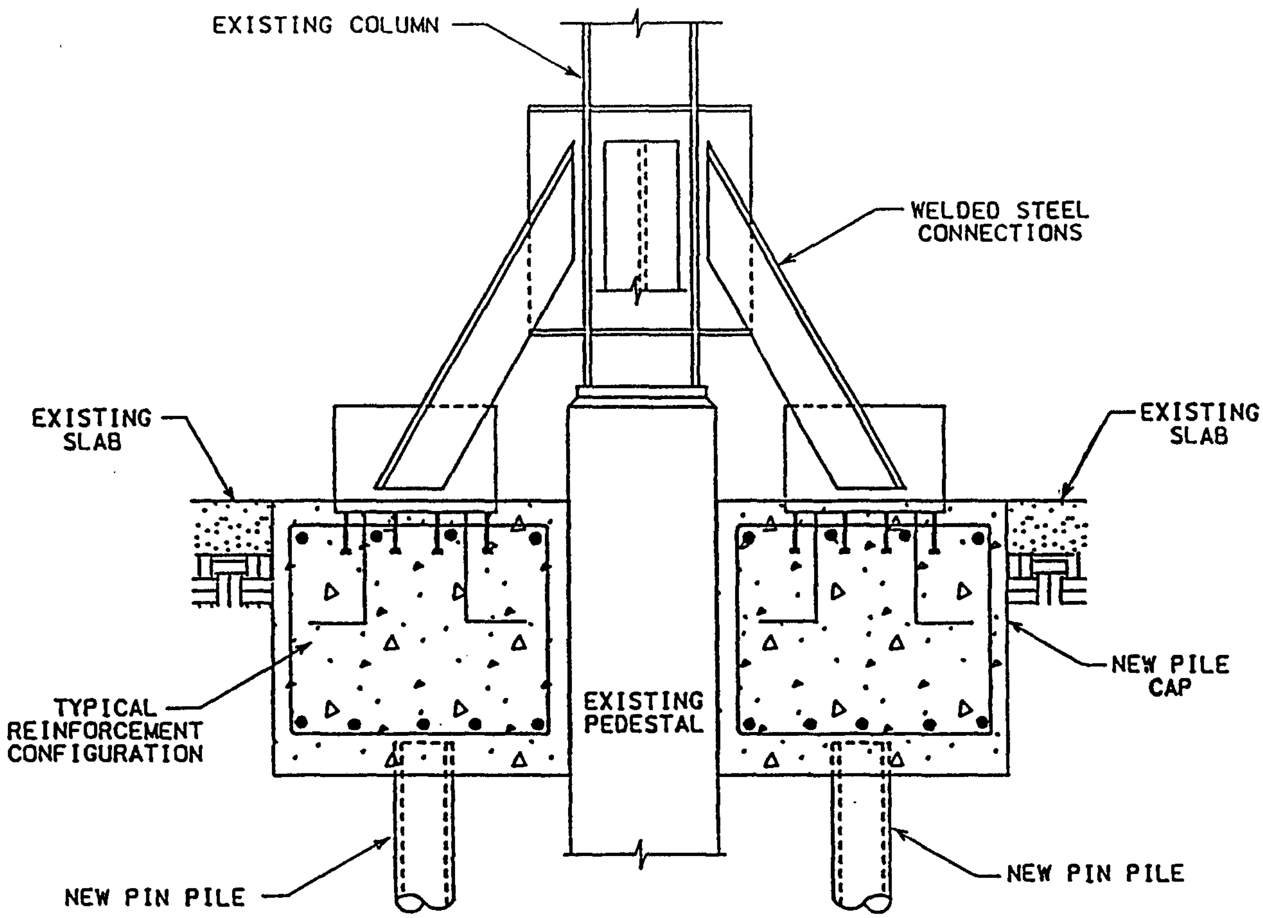


Figure 38. Typical connection between new cap and existing column, Mobile, Alabama (Bruce et al., 1992).

rotate the 178-mm-diameter casings to the target depth of 25 m. The drilling of the very dense and permeable sands with such relatively small equipment was greatly facilitated by using biodegradable polymer drilling slurry. This material was completely displaced from the hole with water flush before tremie placement of the grout into the casing. The grout was prepared from Type I/II cement in a colloidal mixer, using a w/c ratio of 0.45.

Following placement of the 12-m-long reinforcing bar in coupled lengths, the casing was then rotated and withdrawn while grout was continually pumped through it at a pressure of about 0.4 MPa. This phase of pressure grouting was conducted for 11 m of withdrawal. Thereafter, the casing was plunged 1.8 m down into the pressure-grouted zone (for a total of 16 m in the pile), and further pressure grouting commenced until either grout leakage occurred up the outside of the casing, or a pressure of 1.7 MPa had been recorded, or nine 42-kg bags of grout had been injected. The drill head was then disconnected and the rig was moved to the next location.

No pile installation was permitted within 9 m or 1 day of a new pile to prevent possible disturbance of the setting grout.

Testing and Performance. An expanded test pile research program was undertaken, with six piles tested to failure before installation of the production piles, and an additional three tested in an adjacent area after the work was complete. A test arrangement with a safe structural capacity of 2800 kN was provided, compared with the pile service load of 900 kN.

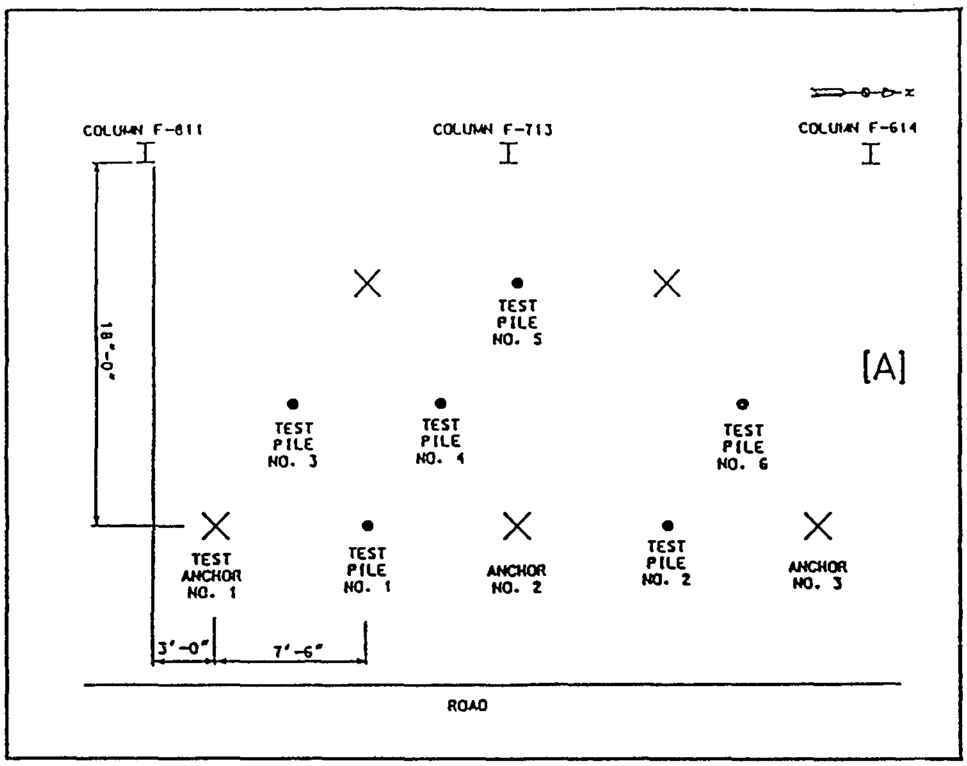
Test Site A [test piles 1 through 6, figure 39 (a)] was approximately 60 m east of Test Site B [TP's 7 through 9, figure 39 (b)]. At the former site, the dense sands and gravels were encountered 4 to 6 m below the surface, whereas at the latter site they commenced about 9 m down. Construction was generally the same as for the production piles, with specified details summarized in table 24.

The change from drill flush polymer X to Y was made to aid preparation and handling. The oversize casing used to isolate the pile casing in TP1 and TP2 was not used elsewhere. The changes in the nature of the bond zone reinforcement in the latter group of piles was for experimental reasons.

Each pile was tested basically in accordance with ASTM D1143 provisions except that intermediate cycles were inserted into the sequence. Table 25 summarizes the test results, from which it is clear that two modes of failure were recorded: a gradual plunging, associated with grout-soil bond failure, and a sudden, explosive release of energy, related to an internal materials failure. Bruce et al. (1992) conducted an intensive analysis of these data, and introduced the concept of Elastic Ratio as both an analytical tool and a predictor of pile performance. This is discussed in volume II. They concluded:

- In appropriate soil conditions, and with adequate bond zone reinforcement, compressive loads at least as high as 2460 kN can be resisted with Type 1B micropiles.

- Progressive debonding occurs between the casing and the surrounding grout, permitting load to be transferred ever deeper down the pile.



1 in = 25.4 mm
1 ft = 0.305m

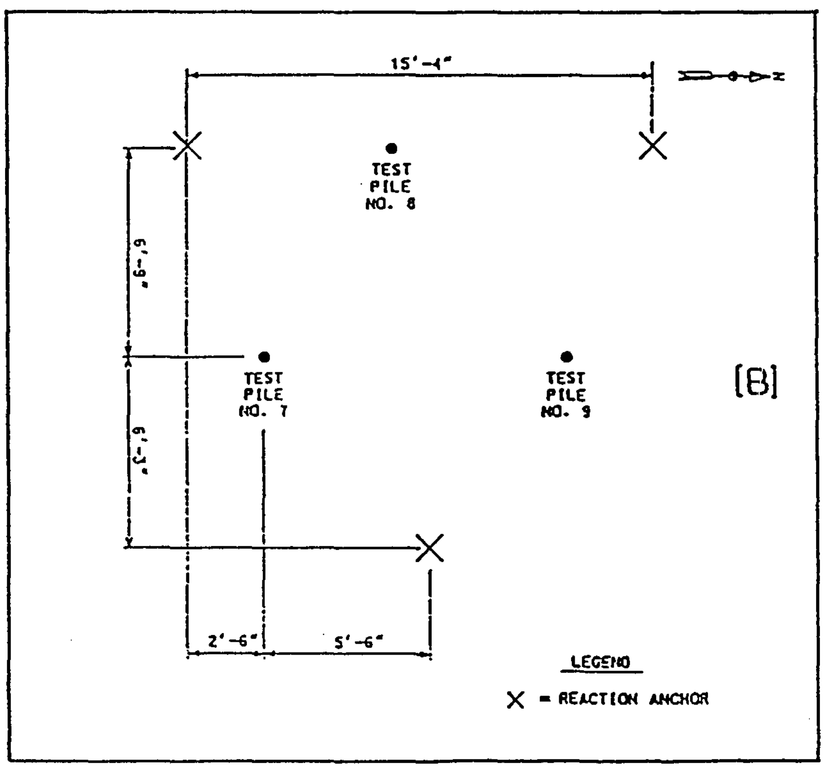


Figure 39. Plan of test pile, Area A (test piles 1 through 6) and Area B (test piles 7 through 9), Mobile, Alabama (Bruce et al., 1992).

Table 24. Summary of test pile construction data, Mobile, Alabama (Bruce et al., 1992).

Test Pile #	Total Depth Drilled (m)	Grouting (1 bag weighs 43 kg)	Total Depth Pressure Grouted (m)	Total 178-mm Casing Left in Place (m)	Rein-forcement (m)	Notes
TP1	25.0 (Drill fluid "A")	17 bags tremie + 10 bags pressure	25.0 to 16.5 i.e. 8.5 m	18	25.0 to 15.8 i.e. 9.2 m of 44-mm rebar	•Upper 12 m surrounded by 244-mm casing (open annulus)
TP2	25.0 ("A")	20 bags tremie + 13 bags pressure	25.0 to 16.5 i.e. 8.5 m	18	25.0 to 14.6 i.e. 10.4 m of 44-mm rebar	•Ditto except pea gravel placed in annulus for lateral support
TP3	25.0 ("A")	18 bags tremie + 6 bags pressure	25.0 to 15.8 i.e. 9.2 m	17 (12 m debonded)	25.0 to 15.8 i.e. 9.2 m of 44-mm rebar	•Problem during grouting •Annulus clear for minimum 3.5 m. •Very low grout strengths
TP4	25.0 ("A")	27 bags tremie + 15 bags pressure	25.0 to 15.5 i.e. 9.5 m	17 (12 m debonded)	25.0 to 14.3 i.e. 10.7 m of 44-mm rebar	•Flush connected to TP3 from depth of 9 m (Age 10 days at time)
TP5	25.0 ("B")	30 bags tremie + 20 bags pressure	25.0 to 13.7 i.e. 11.3 m	15	25.0 to 12.8 i.e. 12.2 m of 44-mm rebar	•High grout pressures •Gravel near tip
TP6	25.0 ("B")	20 bags tremie + 16 bags pressure	25.0 to 13.7 i.e. 11.3 m	15	25.0 to 12.8 i.e. 12.2 m of 44-mm rebar	•Ditto as for TP5
TP7	25.0 ("B")	18 bags tremie + 18 bags pressure	25.0 to 13.7 i.e. 11.3 m	15	None	•Relatively low grout pressure
TP8	25.0 ("B")	18 bags tremie + 27 bags pressure	25.0 to 13.7 i.e. 11.3 m	15	25.0 to 12.8 i.e. 12.2 m of 57-mm rebar	•Slightly oversize bit (229 mm) •Relatively low grout pressure
TP9	25.0 ("B")	18 bags tremie + 14 bags pressure	25.0 to 13.7 i.e. 11.3 m	15	11 strands each 15-mm diameter, full length	•Lower grout pressure

Table 25. Summary of test pile performance, Mobile, Alabama (Bruce et al., 1992).

Test Pile	Maximum Recorded Load (kN)	Pressure-Grouted Length (m)	Mode of Failure	Average Grout/Soil Bond at Maximum Recorded Load (kN/m)	Comment
TP1	1500	8.5	Grout-Soil	176	•Loaded in 220-kN cycles to plunging failure at 1500 kN •Reached maximum of 1330 kN on next load cycle
TP2	2624	8.5	Grout-Soil	309, apparently, but realistically 191	•Loaded in 220-kN cycles/ 130-kN increments to 2625 kN (no failure) •Failed at 2535 kN next cycle •Estimate 40% load resisted permanently in free length
TP3	1334	9.1	Grout-Soil	147	•Loaded in 220-kN cycles to 1330-kN failure •Possibly poor grout •Failed on first cycle
TP4	1780	9.4	Grout-Soil	191	•Loaded in 220-kN cycles to 1780-kN failure •Failed on first cycle
TP5	2002	11.2	Internal	176 OK	•Loaded in 220- and 440-kN increments to 2000-kN explosive failure •Failed 15 minutes into load hold period
TP6	2224	11.2	Internal	206 OK	•Loaded in 220- and 440-kN increments to 2225-kN explosive failure •Failed 5 minutes into load hold period
TP7	1780	11.2	Internal	162 OK	•Loaded cyclically in 220-kN increments to 1780-kN explosive failure
TP8	2446	11.2	Internal	221 OK	•Loaded cyclically in 220-kN increments to 2450-kN explosive failure
TP9	2180	11.2	Grout-Soil	191	•Loaded cyclically in 220-kN increments to 2180 kN •Failed on reloading at 2000 kN

- Interpreted failure load of the pile strongly reflects the proportion of the load that can be accepted in this free length.

- When debonding reaches the bottom of the casing, load is then increasingly transferred into the bond zone. The subsequent pile capacity then depends on the composition of the bond zone. If insufficiently reinforced, this bond zone will quickly fail explosively. If better reinforced, it will sustain more loads, perhaps even high enough to cause failure at the grout-soil interface.

- Debonding is irreversible in its effect on load-transfer length.

- Within 20 percent or so of the failure load, permanent movements and creep rates will become quickly excessive, reflecting the fact that a significantly higher proportion of the total applied load is then having to be resisted by the bond length.

Regarding the performance of the structure, each column was surveyed at weekly intervals throughout the 7 months of installation of the micropiles and the new concrete caps. This confirmed that settlements were occurring at the same rate as had been recorded over the previous 44-month period. This ranged up to 4.5 mm per month. At the end of pile and cap construction, the columns were then connected (by welding) to these new caps at the rate of 20 per month, and column settlement was arrested.

SEISMIC RETROFITTING

I-110, Los Angeles, California (Pearlman et al., 1993)

Background. As noted elsewhere in this report, recent destructive seismic events have enforced upon Caltrans a massive program of seismic retrofitting. The bridge failures from the 1971 Sylmar Earthquake were primarily linked to separations at bridge deck expansion joints and lack of ductility in the supporting columns. Approximately 1250 State bridges were retrofitted to provide deck continuity from 1971 to 1989. Column ductility improvements, delayed until 1986 due to budget constraints, result in an increase in load demand on both superstructure and foundation. Investigations into various bridge foundation problems and solutions were intensified in the years following the 1989 Loma Prieta Earthquake.

A standard approach often utilized by Caltrans designers to resolve bridge foundation problems associated with seismic activity is to add tension/compression pile elements around the perimeter of the existing bridge footings. Caltrans standard piles (driven precast concrete and steel piles, Cast-in-Drilled-Hole (CIDH) piles, and concrete-filled steel pipe piles) were typically used for the foundation support. However, due to constraints on allowable noise and vibration levels, installation difficulties presented by low overhead conditions, difficult drilling conditions, limited right-of-way access, and higher tension capacity requirements, alternatives to the standard driven piles are becoming more desirable and are being incorporated into the design of many retrofit projects.

Caltrans earthquake retrofit project 07-118424, "North Connector Overcrossing" at the intersections of I- 5 and I-110 in Los Angeles, included the retrofitting of Bents 2, 3, 5, and 6 by strengthening the existing footings. The design called for sixteen 600-mm-diameter CIDH piles placed around the existing footing at each bent. However, due to difficult drilling conditions, including buried concrete obstructions and water bearing, flowing sand layers, low overhead conditions, and limited right-of-way access, attempts to install the CIDH piles were unsuccessful. Micropiles had proved to be an acceptable alternative pile method during the major test program conducted in 1992-1993 in South San Francisco (Mason, 1993) and a change order was issued to the general contractor to permit their use.

Site and Geology. The site was in Los Angeles near Figueroa Street and the southbound on-ramp to I-5. The soils consisted of 7.5 m of loose to slightly compact fills overlying dense to very dense sands and gravels. Groundwater level was at the base of the fill. The site had been a dump location for a ready-mix concrete plant, and the upper fill material contained many large chunks of concrete and cobbles.

Three of the retrofitted footings were located adjacent to the Arroyo Seco drainage channel and were accessed from the shoulder of the Pasadena Freeway. The fourth footing was located in the middle of the Pasadena Freeway, creating very difficult access conditions. Overhead clearance under the freeway deck was approximately 6 m.

Design. A total of 64 Type 1B micropiles, each 18 m long, were proposed. They were each designed to support a maximum tensile load of 1350 kN at a maximum head movement of 17 mm, and a maximum compressive load of 2250 kN at a movement of 14 mm.

Construction. A short-mast diesel-hydraulic drill rig was used to rotate the 178-mm-diameter casing to full depth with water flush. Standard construction techniques were used, and 11 m of the casing was left in place through the upper zone. Central, full-length reinforcement was also added, comprising two 36-mm-diameter high-strength bars. Pressure grouting (at 0.7 to 1.0 MPa) was conducted from 9 to 18 m below ground level.

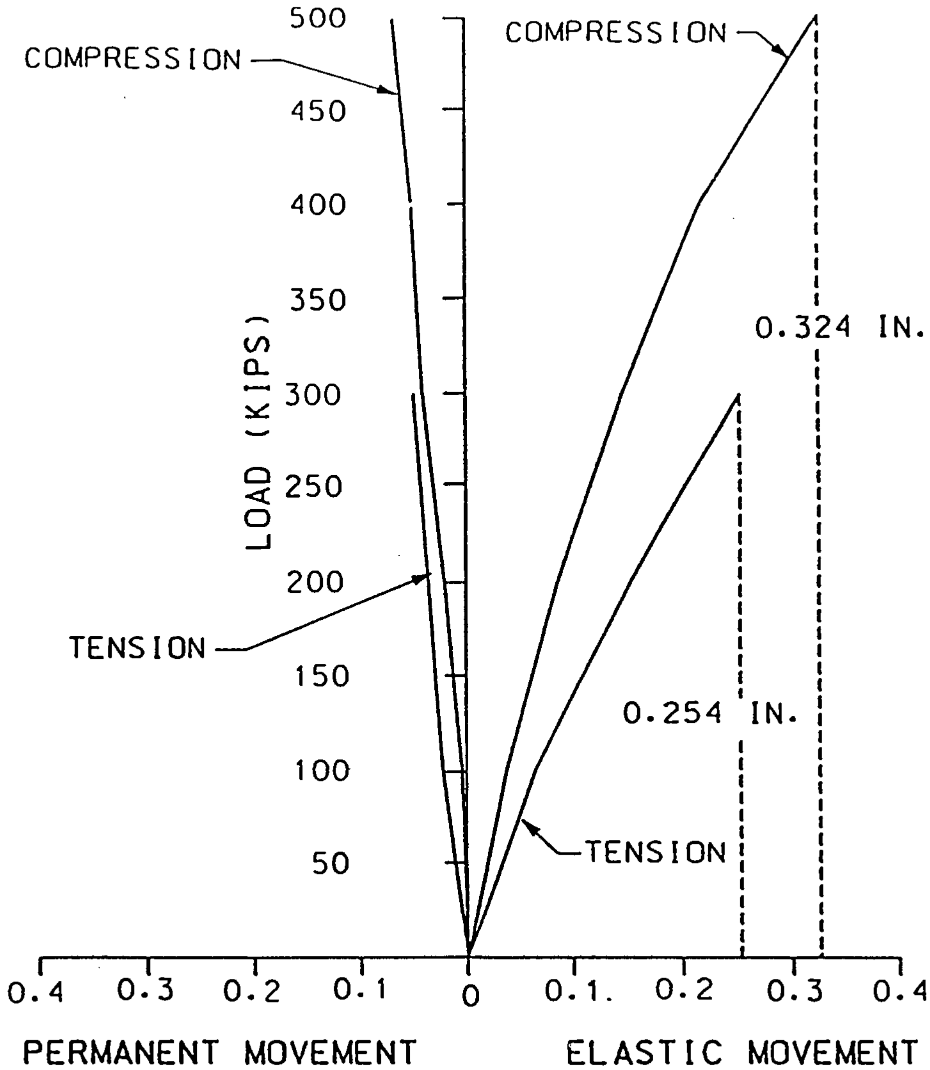
Testing and Performance. A production test pile (Bent No. 3, Pile No. 3), selected by Caltrans, was installed from existing grade, allowing testing to be performed before footing excavation. The pile test was conducted by representatives from Caltrans to verify design assumptions and pile performance. The pile successfully supported the required maximum tension load with a total movement at maximum load of 7.7 mm and a permanent movement of 1.3 mm. The pile then successfully resisted the required maximum compression load with a total movement at maximum load of 10 mm and a permanent movement of 1.7 mm (figure 40). Creep at the tensile maximum was 0.15 mm (5 minutes) and at the compressive maximum creep was 0.18 mm (5 minutes). Total pile head movements relative to the initial position were approximately 50 percent of the predicted and allowable movements.

Britannia School, Vancouver, BC, Canada (DSI, 1994)

Although fewer details were published on this particular case history, it is typical of seismic retrofitting being conducted on both coasts of North America.

The school was built of brickwork and concrete in the early 1920's, and recently had to be retrofitted to comply with current seismic codes. A total of 150 Type 1A micropiles were designed, each reinforced by a single 57-mm-diameter, 520-MPa GEWI bar. Each extended 15 m through the existing footings into dense sands with plastic silt beds. Drilling was conducted with a special electrohydraulic track rig, operating within the 2.7-m-high foundation space. Hole diameter was 140 mm, and the bars were protected by a grouted corrugated sheath. Pile inclinations varied from vertical to 40 degrees off vertical.

In advance, a preproduction test pile was installed and tested to verify that the service load of 1100 kN, in both tension and compression, could be resisted. Testing was conducted alternately in compression and tension to simulate actual earthquake loadings, and was terminated when loads of 1250 kN were successfully resisted.



1 in = 25.4 mm
1 kip = 4.45 kN

Figure 40. Permanent/elastic movements of test pile North Connector Overcrossing (I-110), Los Angeles, California (Pearlman et al., 1993).

CHAPTER 4. MICROPILES AS IN SITU REINFORCEMENT: SELECTED CASE HISTORIES

EMBANKMENT, SLOPE, AND LANDSLIDE STABILIZATION

State Route 4023, Armstrong Co., Pennsylvania (Pearlman et al., 1992)

Portions of this two-lane roadway were constructed on a slope adjacent to the Allegheny River. A 75-m-long section of the road, and the railroad tracks located upslope, were experiencing damage caused by slope movements toward the river. A monitoring program initiated by the Pennsylvania Department of Transportation (PADOT) indicated a slip plane was located about 8 to 10 m below the road and that the slope was moving at a rate of up to 18 mm/month.

PADOT designed a repair using prestressed rock anchors and tangent caissons extending into competent rock. The earth pressures used for the design were based on the results of stability analyses, for which the soil along the slip plane was assigned a residual angle of friction. This design provided a minimum factor of safety with regard to the overall slope stability equal to 1.5 and 1.2 for the normal and rapid drawdown conditions, respectively. A post-bid alternative based on inclined CASE 1 micropiles was accepted by PADOT with a resultant savings of about $1 million, compared to the lowest bid for the anchored caisson wall design.

In general, the wall consisted of four rows of Type 1A micropiles extending across the slip plane and into competent rock. The wall comprised two equal-length sections designated as Wall A and Wall B. Wall A contained a higher density of piles than Wall B because the top of a weathered rock dipped to a lower elevation in the area of Wall A. Construction details are provided in table 7.

Performance monitoring consisted of: (1) strain gauges on the reinforcement of the outermost pile rows; (2) inclinometers through and upslope from the wall; (3) fixed-end extensometers to measure average strain along the reinforcements; and (4) surface survey monuments. Due to space limitations, only the results of the slope inclinometer data for Wall A are described.

The maximum movement at select inclinometer locations along the length of the wall is shown in figure 41. The data for inclinometers located relatively close to and within the wall indicated that up to 38 mm of horizontal movement occurred during construction, and as much as 10 mm of additional movement occurred in the year following completion of construction. Inclinometer S11W, located approximately 18 m upslope from the wall, exhibited approximately 30 mm of additional movement in the first year after the completion of construction and 8 mm of movement in the second year following the end of construction.

Overall, the inclinometer data showed that the wall significantly slowed the slope movement at the site from a rate of approximately 200 mm/year to a rate of 8 mm/year. Furthermore, the rate of movement decreased with time. The stabilizing effects of the relatively flexible pile elements appeared to become greater with the increasing movement of the wall. If movements had

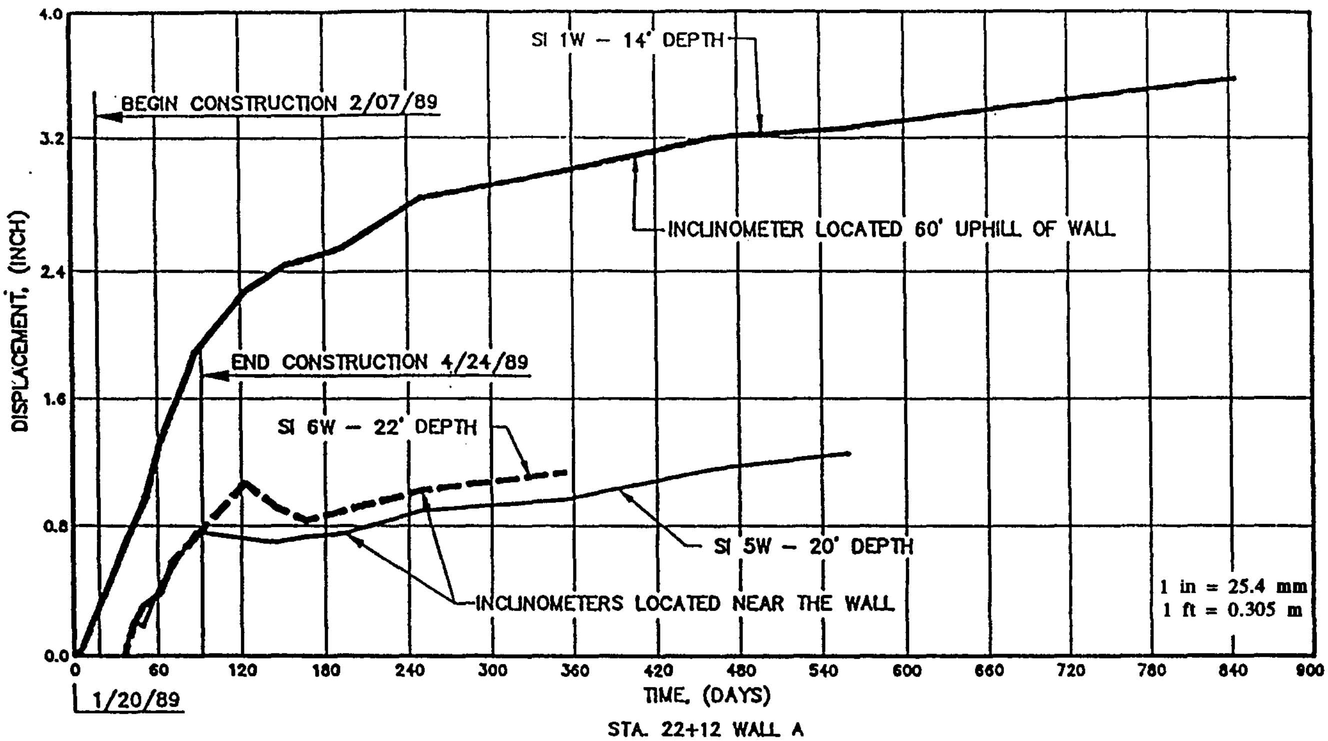


Figure 41. Maximum movement at select inclinometer locations with time, State Route 3023, Armstrong County, Pennsylvania (Pearlman et al., 1992).

continued and eventually reached an unacceptable magnitude, additional piles could have been installed.

The performance of the wall was back-analyzed with a view to checking the design approach. The location of the slip plane was estimated based on inclinometer data, and the driving forces of the slide were calculated to be approximately 2000 kN/m.

Using equations developed by Leinenkugel (1976) and considered by Winter et al. (1983) for the stabilization of creeping clay slopes, the loads acting on the wall were estimated. The basic assumption was that the mobilized shear stress along the failure plane equals the shear strength associated with an initial strain rate. The placement of the micropiles across the zone of movement reduced the stresses in the soil along the slide plane, causing the slope to move at a slower rate, thereby reducing the mobilized soil shear resistance along the slide plane. Consequently, equilibrium dictated that a decrease in the mobilized shear resistance in the soil along the slide plane resulted in an increase of the resisting forces provided by the piles. Using this approach and the actual measured rates of movement, the resisting force along the slide plane provided by the wall was calculated as equal to about 13 percent of the driving load (260 kN/m).

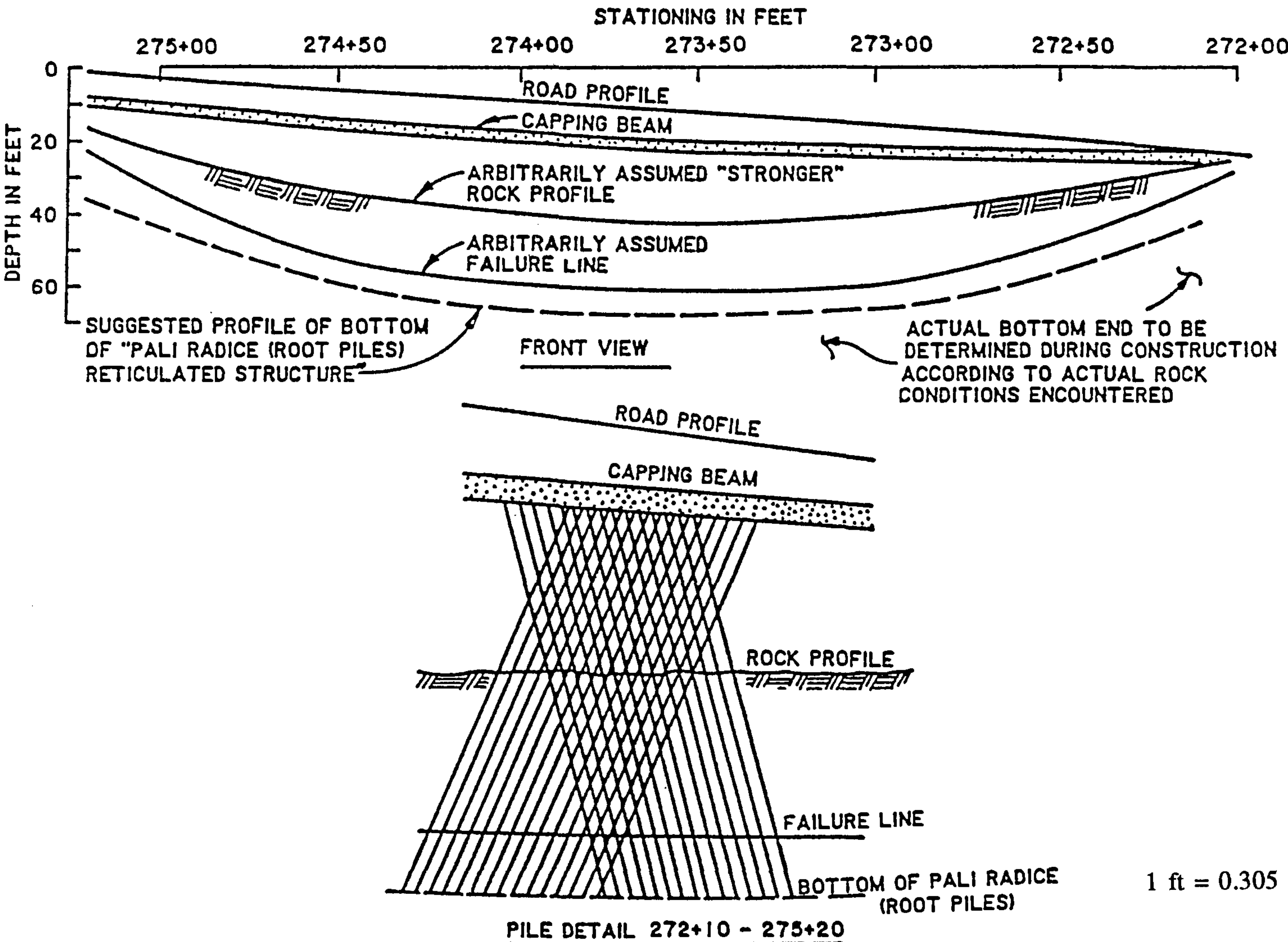
Using the procedure for preliminary design, the maximum resistance provided by the wall was calculated to be approximately 150 kN/m (5.7 percent of the driving load). A more detailed analysis using the Group I program (Reese et al., 1987), and assuming that the rock located below the slip plane acted as a pile cap, confirmed that a maximum resistance of 13 percent of the driving load could be mobilized. This load was similar to the back-calculated load imposed on the wall based on the measured decrease in the strain rate.

The field data suggested that the piles were at the limit of resistance as given in the procedure for preliminary design (i.e., the grout surrounding the reinforcement was beginning to crush). Further slope movements would have resulted in the piles acting in tension, which would have provided additional capacity and ultimately led to cessation of slope movement. This, in fact occurred within a few months.

Mendocino County, California (Palmerton, 1984)

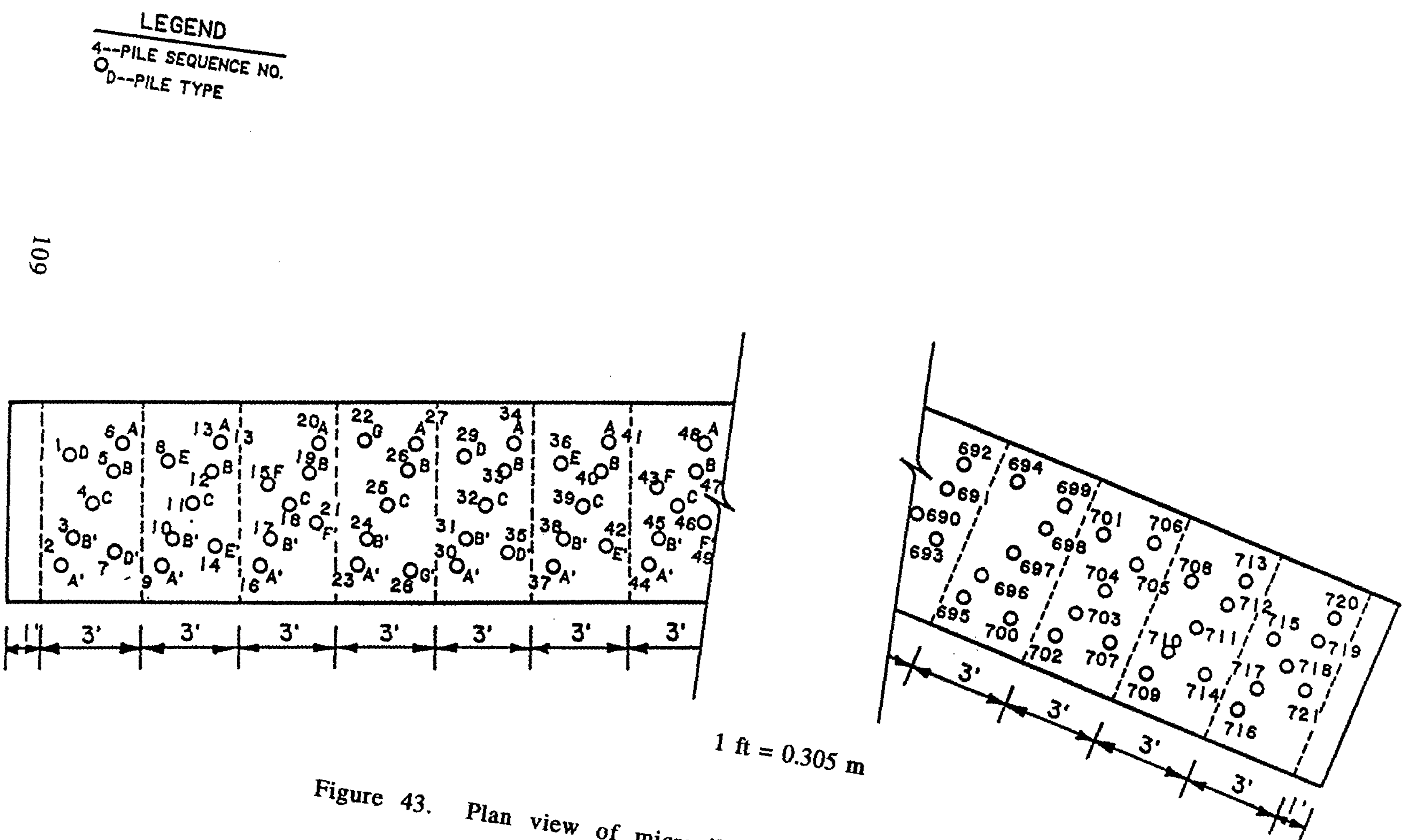
Background. This project is located on Forest Highway 7 in Mendocino National Forest, California. The principal use of the road was to facilitate logging operations. Although the portion of the road was then only 13 years old, slide movements had occurred at numerous locations. A 94-m-long section was selected for repair.

Site and Geology. The site comprises metasedimentary rocks, mainly phyllite with secondary mica quartzschist, and slate. The slide mass itself comprised a mixture of red-brown clayey soil and rock fragments of the parent. The road was built across the old landslide and subsequent movements were attributed to excessive rainfall causing saturation of old loose slide debris forming the 6- to 9-m-high cut slope and embankment foundation, and removal of lateral support by cutting.



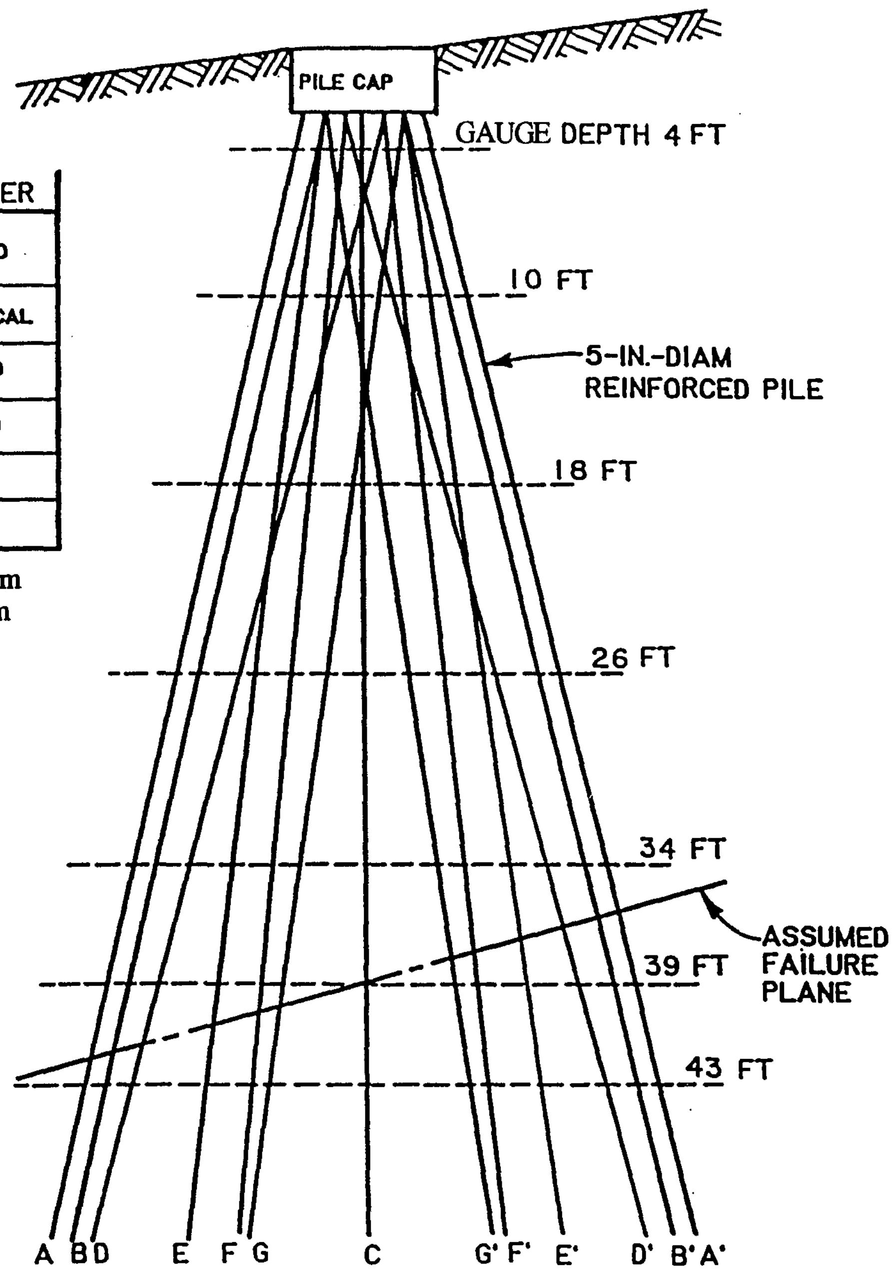
1 ft = 0.305 m

Figure 42. Front view of micropile network, Mendocino, California (Palmerton, 1984).



1 ft = 0.305 m

Figure 43. Plan view of micropile network, Mendocino, California (Palmerton, 1984).



PILE	BATTER
A,A',B,B'	3.4:10
C	VERTICAL
D,D'	2.6:10
E,E'	1.3:10
F,F'	1.0:10
G,G'	1.4:10

1 in = 25.4 mm
1 ft = 0.305 m

Figure 44. Cross-sectional view of micropile network, Mendocino, California (Palmerton, 1984).

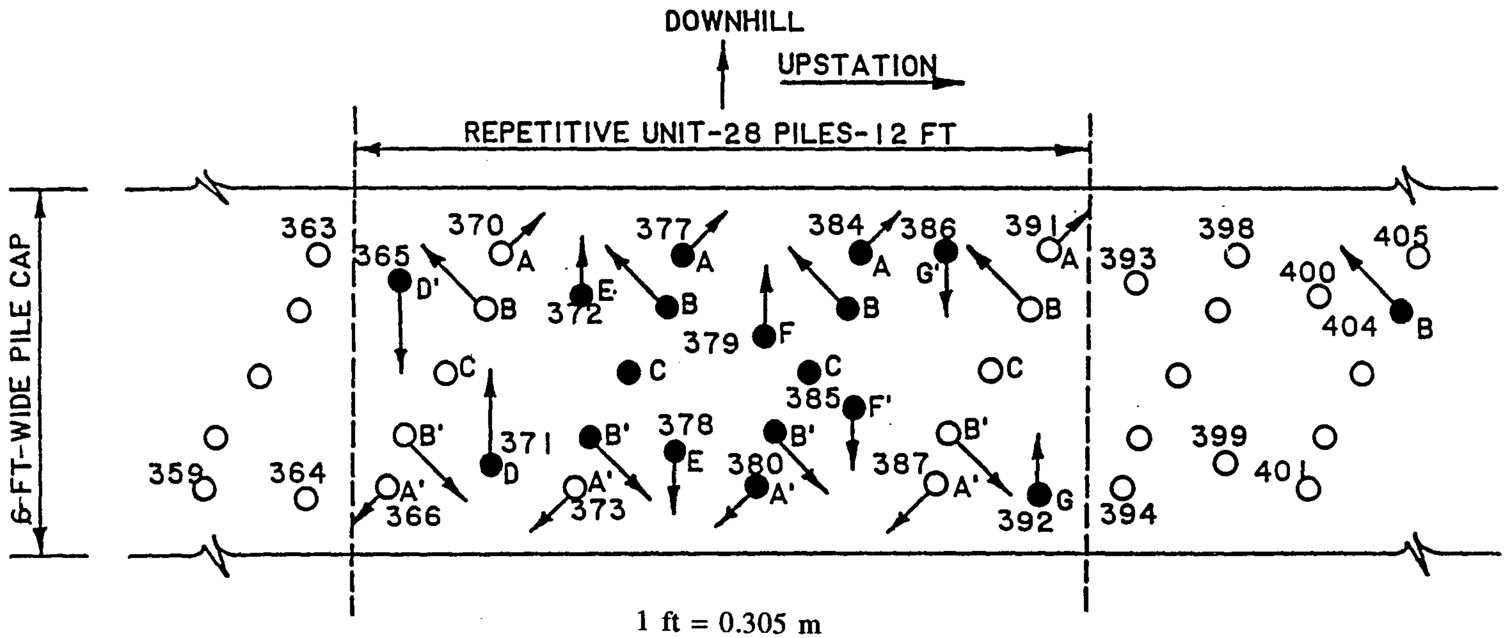


Figure 45. Plan view of instrumented section of wall, Mendocino, California (Palmerton, 1984).

Design. The density of the piles was 8.25 per linear meter of retaining structure (figures 42 through 44), arranged in repetitive units 3.6 m long (figure 45). The repair was visualized as a buried retaining wall, the concept being to provide additional sheer capacity to increase the factor of safety to an acceptable value. The wall was designed as a CASE 2 structure.

Construction. A total of 721 127-mm-diameter Type 2A micropiles were placed. The concrete cap, 0.9 m thick by 1.8 m wide, was cast first. Drilling was conducted by open-hole methods and air flush to depths of 15 to 24 m. The 6-m lengths of rebar were welded together and placed prior to grouting.

Performance and Testing. Seventeen piles of different orientations were instrumented with strain gauges. With few exceptions, the bars appeared to all be acting in compression. The trend of the data showed that following installation, the strains built up quite rapidly for 1 to 2 months (the construction season) and then either remained constant or relaxed slightly. Analysis of the data indicated that the maximum compressive force generated in a pile was less than 6 kN, indicating the conservatism of the design.

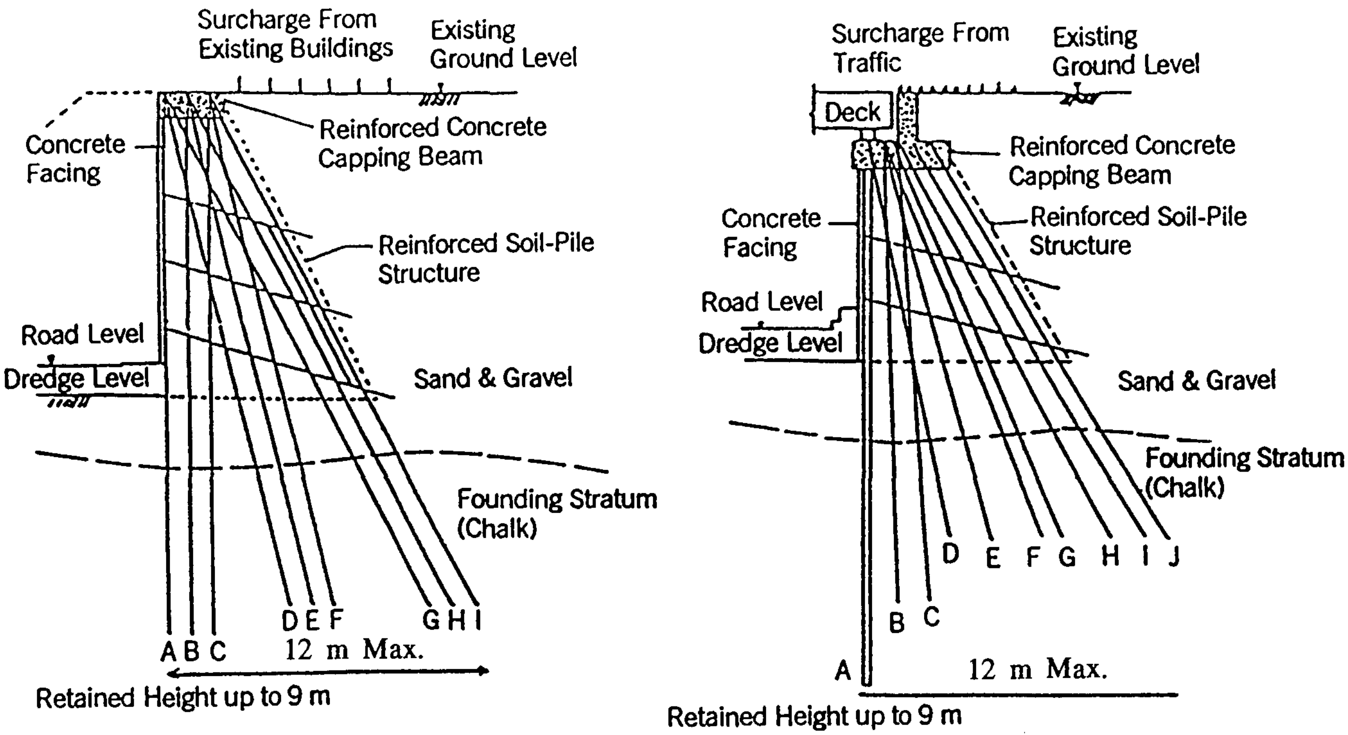
The completed structure performed well during the rainy season. No cracks or movements were noted in the reconstructed roadway. No large strains or strain buildups within the instrumented section had built up through the time of the last instrument readings (18 months after construction).

Dartford Tunnel Southern Approach Widening, England (Attwood, 1987)

A widening of the southern approach road to the Dartford Tunnel was proposed to improve traffic flow to and from the tunnel's southern end. The existing road lay in a cutting in the area between the Watling Street and Brent Street Bridges. Accommodating the increased width necessitated the construction of retaining walls up to 9 m high and approximately 300 m long, along both sides of the approach. The western side of the road generally had sufficient open ground to allow installation of an anchored retaining wall. On the eastern side, however, the close proximity of the boundary load-bearing walls of an adjacent garage complex precluded the use of such conventional techniques.

An alternate design scheme utilizing a CASE 2 reticulated micropile system was adopted (figure 46). As an added benefit, the reticulated micropile system was also able to form the new abutments for both the Watling Street and Brent Street bridges, and thus, in these locations, also acted as a CASE 1 structure.

The reticulated micropile system was well suited to the difficult site conditions. Only 1.75 m of working room existed between the top of the cutting and the adjacent garage complex. To accommodate installation under these conditions, a piling platform was erected on the existing slope to support the 3-tonne, skid-mounted, hydraulically operated drilling rig. The design of the platform and light weight of the drill rig ensured that the cutting remained stable throughout construction, and that the safety of the adjacent traffic flow was not compromised. The installation of the piles for the new cast-in-place abutments was accommodated during partial road closures on both bridges.



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

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 46. Schematic illustration of the reticulated micropile structure for the eastern retaining wall and as underpinning for the bridge deck, Dartford Tunnel, England (Attwood, 1987).

The CASE 2 structure was designed to resist loadings from earth pressure forces, self-weight, and adjacent structures. "At rest" earth pressures were used to limit deformations under service-loading conditions. All piles were connected at their tops by a continuous, reinforced concrete capping beam. The selection of pile geometry, pile inclination, and number of elements per row per unit length of wall involved such considerations as:

- Providing adequate stiffness to the retained soil/pile network.
- Inducing an efficient distribution of pile loading throughout the network.
- Ensuring that a shear key was provided across any possible critical surface.

Other influential factors included the ground conditions and the retained height. In addition, the experience gained by the contractor during the construction of many similar structures throughout the world over a 30-year period also provided valuable input.

As a further enhancement to the design, but more in response to competitive pressure than to technical necessity, short soil nails were required to intensify the network and provide restraint to the exposed face. These generally were installed on a 2-m grid during staged excavation.

At both the Watling Street and Brent Street bridges, the retaining wall was required to form the new east abutments. This was achieved by incorporating higher capacity cased Type 1B micropiles into the vertical front face of the network, coincident with the applied bridge deck loading locations. Figure 46 illustrates the use of the reticulated micropile structure in this extended application. Typical ranges of pile loadings are also given in this figure for each application.

The techniques used in the construction of the typical 140-mm-diameter micropiles, and of the higher capacity 220-mm-diameter tubular micropiles, were as follows. Rotary drilling techniques were used throughout to minimize noise and vibration.

1. The drilling rig was set up at the required inclination at the top of capping beam level.

2. A 140-mm-diameter temporary casing with cutting crown was rotated and inserted using water flush.

3. Drilling then continued through sand and gravel deposits, adding temporary casings in short lengths until the required penetration into the underlying chalk was achieved. (Typical overall pile length for 8 m of retained height was 15 m.)

4. Flushing water and spoil were contained within sandbag bunds, excess water being pumped to settlement tanks prior to discharge in the main drainage system. Great care was taken to avoid discharge down the embankment into the flow of traffic on the main carriageway beneath.

5. On completion of drilling, a 1.5:1 sand/cement grout having a characteristic strength of 35 MPa was injected into the pile bore via a

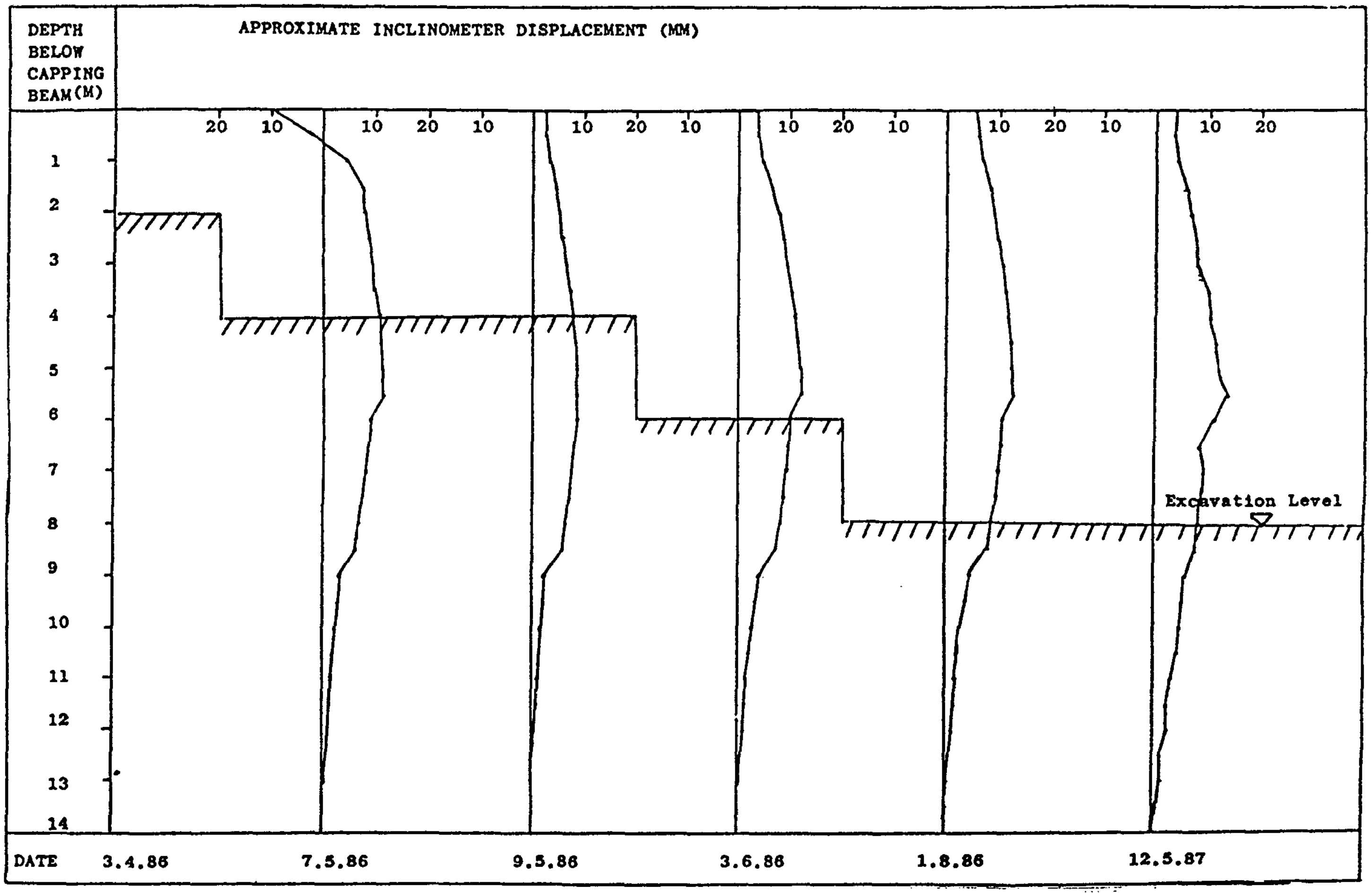


Figure 47. Displacement of the reticulated micropile structure at various phases of excavation and dates, Dartford Tunnel, England (Attwood, 1987).

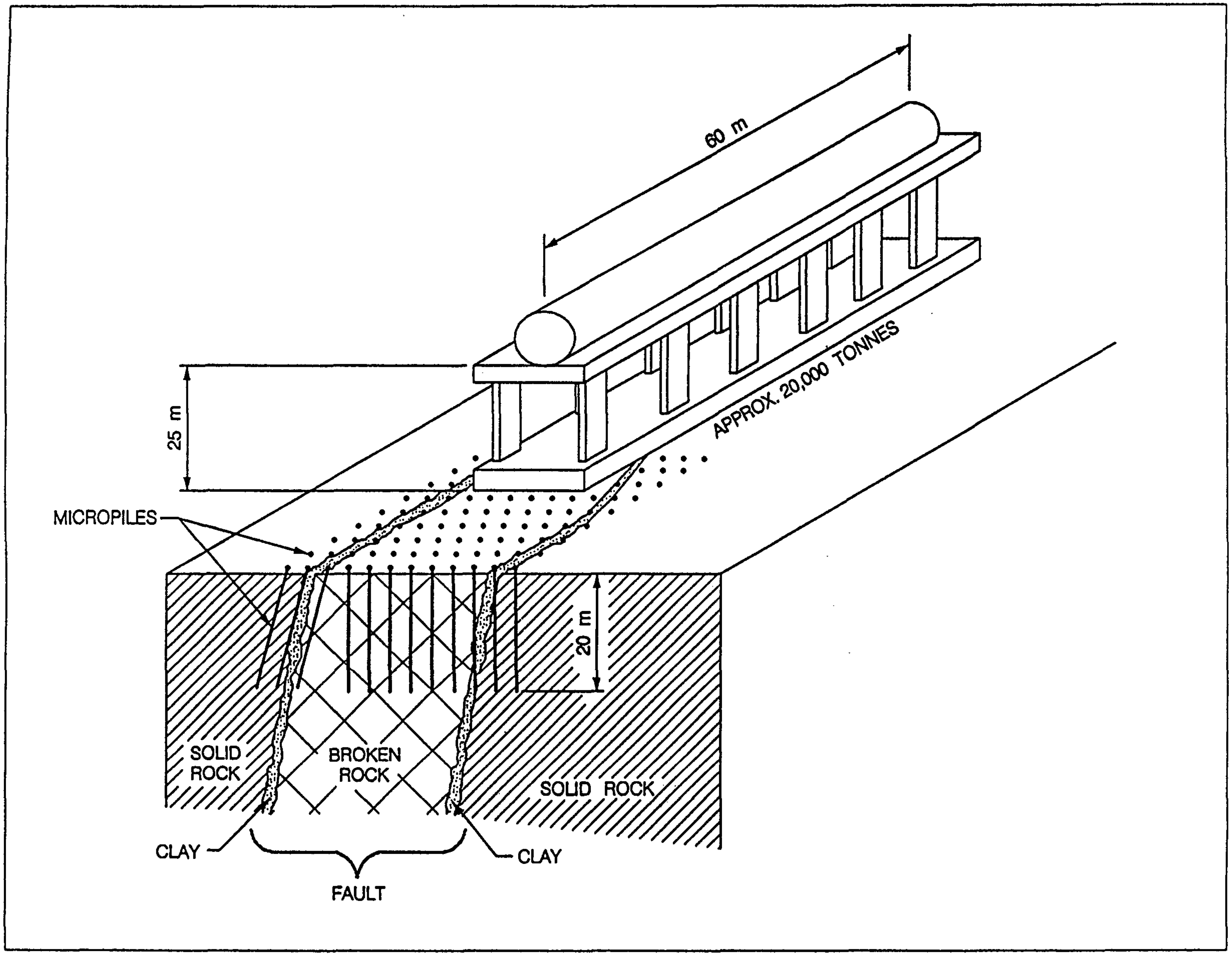


Figure 48. Micropile foundations at Uljin Nuclear Power Plant, South Korea (Blondeau et al., 1987).

tremie tube placed to the toe. Injection continued until all water and contaminated grout was displaced to the surface.

6. The single deformed reinforcing bar, complete with expanding plastic centralizers, was then placed into the grouted borehole jointed by means of full-strength threaded compression couplings. Each pile was therefore reinforced for its full length.

7. The temporary casing was then progressively extracted in short lengths, topping off the borehole with additional fresh grout as appropriate.

The construction of the 220-mm-diameter micropiles was similar to that of the 140-mm micropiles, except that:

1. The 220-mm-diameter temporary casings were installed vertically to a depth of 18 m.

2. Reinforcement was comprised of a 140-mm outside diameter, 8-mm-thick steel tube installed in one length with full-strength welded joints.

A number of 20-m-deep vertical inclinometer tubes were installed through the pile network at selected locations. Horizontal wall deflections were monitored before, during, and after excavation. The results obtained at the maximum retained height are summarized in figure 47.

The data showed that the deflection profile displayed the typical shape of a cantilever with increasing, uniformly distributed, applied loading. The actual magnitude of deflection and its profile were similar to those predicted by theoretical analysis. Longer term monitoring indicated negligible additional deflection of the structure.

SOIL STRENGTHENING AND PROTECTION

Korean Nuclear Unit 10, Uljin, Korea (Blondeau et al., 1987)

Background. During the investigation phase of the rock foundations for this new nuclear power station, a major "weak zone" was found. It was approximately 23 m wide and 50 m long, striking approximately NNW. As described in detail below, it was of very variable, weathered, and sheared lithologies and underlaid the footprint of the new turbine hall. The challenge was to provide this zone with the same overall modulus of subgrade reaction as the rock mass upon which the adjacent structures were to be founded (figure 48).

A micropile solution was developed in which the concept was to provide a "plug bridging the two edges of the weak zone" and to provide conceptually an anisotropic rock mass, the static Young's Modulus of which was intended to be:

E = 500 MPa, horizontally
E = 4000 MPa, vertically

These parameters, plus all other numerical values determined for this project were arrived at following detailed and sophisticated mathematical modeling, the basics of which are beyond the scope of this synopsis. In essence, however, the micropiles were designed as vertical elements, except for slightly inclined piles at the edges, and reached 20 m in depth. Throughout, emphasis was given to the load-holding capacity of single piles and group effects were ignored. Therefore, it is concluded that these were designed as CASE 1 elements, although, in essence, a CASE 2 structure may well have been the truer philosophical concept.

Site and Ground Conditions. Three types of rock or soil conditions were identified from a mechanical viewpoint:

1. Slightly Weathered rock on both sides of the weak zone in which:
   $E_{Menard} > 660$ MPa
   $E_{Young} > 2000$ MPa

2. Highly Weathered and Medium Weathered rock, constituting the main part of the weak zone, is characterized by:
   $100 < E_{Menard} < 660$ MPa
   and average figures of $E_{Menard} = 275$ MPa
   $E_{Young} = 825$ MPa

   Lithologically this was a granite gneiss, occasionally pegmatitic with occasional quartz veins.

3. Clay filling in the faults on the edges of the zone and in some of the cracks in the central part of the zone:
   $14 < E_{Menard} < 76$ MPa
   and average figures of $E_{Menard} = 44$ MPa
   $E_{Young} = 131$ MPa

This clay probably coincided with the totally weathered ultrabasic intrusions of a mafic dike.

The designers took a (conservative) mean value for the weak zone of 500 MPa, corresponding to a fissured mass of rock, highly to medium weathered, and containing 10 to 15 percent clay-filled fissures. The dynamic Young's Modulus was taken as 6.5 times the static value of 500 MPa, namely 3250 MPa.

The site was "greenfield" and work was conducted in the open air from a stripped rock surface.

Design. The piles were designed according to the French code, and key geotechnical parameters were drawn from the results of a previous test program, described below.

For the piles, the following were observed:

Reinforcement to ASTM A615 (Grade 60)
$F_y = 414$ MPa, $E = 210{,}000$ MPa
Grout $E = 15{,}000$ MPa

Table 26. Two basic types of production micropiles, Korean Nuclear Unit 10, South Korea (Terrasol, 1984).

	TYPE I	TYPE II
Diameter of borehole	∿ 170 mm	∿ 170 mm
Diameter of rebars	35 mm	35 mm
Number of rebars	6	3
Allowable load:		
static	1200 kN	600 kN
dynamic	1600 kN	800 kN
Total length of the rebars	20.50 m	20.50 m
Embedded length	20.00 m	20.00 m

Soil E_{static} = 2000 MPa
$E_{dynamic}$ = 13,000 MPa

Two types of piles were designed, each in a nominal 170-mm-diameter hole, 20 m deep: Type I with six 35-mm-diameter rebars and Type II with three 35-mm-diameter rebars (table 26). (Due to supply restrictions, the maximum diameter that could be provided in 20.5-m continuous lengths was 35 mm.) The total maximum surface pressure (including seismic) was calculated as 0.93 MPa, and distributed as shown in figure 49.

The interpile distance of about 10 diameters was selected in the knowledge that "there is no profitable group effect for such a mesh grid." Three detailed spacings were designed:

- 1.65 x 1.65 m: Under the table group and the edges of the weak zone (Areas 1, 3, 6 of figure 49).
- 1.85 x 1.85 m: Under the central part of the building raft, more lightly loaded (Areas 2, 5).
- 1.45 x 1.45 m: Under the corner of the northern wall (Area 4).

The piles were generally vertical, except for the peripheral three rows at each edge, where a 15-degree inclination (the maximum practical) was selected "to best transfer the shear stresses to the fresh rock."

Further detailed analyses showed that for the chosen scheme, the reinforced ground could not perform identically to the isotropic rock in both static and dynamic load combinations. Knowing that the most severe static loading combination would certainly occur, the designers gave priority to the static modulus requirement, with the result that the dynamic modulus would be 75 percent lower than that required, although this was within the range of accuracy acceptable in the analytical method.

Construction. The working surface was prepared by removing superficial rock, deeper in the case of the weathered intrusion (figure 50). The Type 1D micropiles were then installed to the following tolerances:

Location in plan: ± 50 mm
Inclination: ± 1 degree
Level of top of rebars: ± 50 mm
Free space between bars: 10 mm (figure 51)
Spacers: at 1.5-m centers
Centralizers: at 2-m centers

Casing of 167-mm-diameter and 8-mm wall thickness was rotated down using water flush (or, in special clay seams, bentonite slurry). About 5 m of casing was used for vertical piles, but full-length casing was needed for inclined holes. After drilling, further water flush was used to clean the holes. Before placing the reinforcement, the primary grout was placed by tremie. This was a cement/bentonite/water mix with a w/c ratio of 0.5, and a bentonite-to-cement (b/c) ratio of 0.25 percent. The casing was fully extracted. Post-

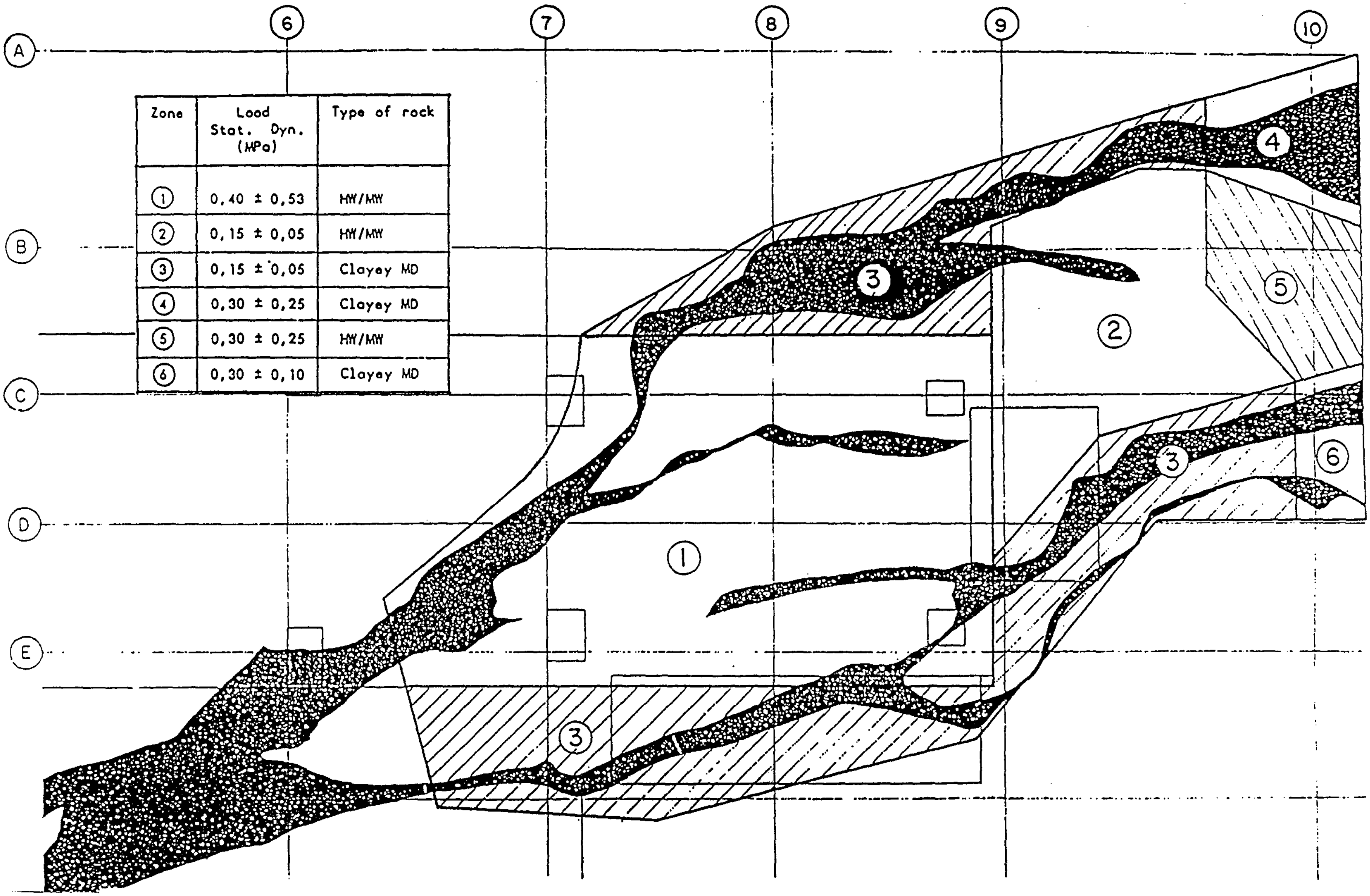


Figure 49. Map of the applied stresses, Korean Nuclear Unit 10, South Korea (Terrasol, 1984).

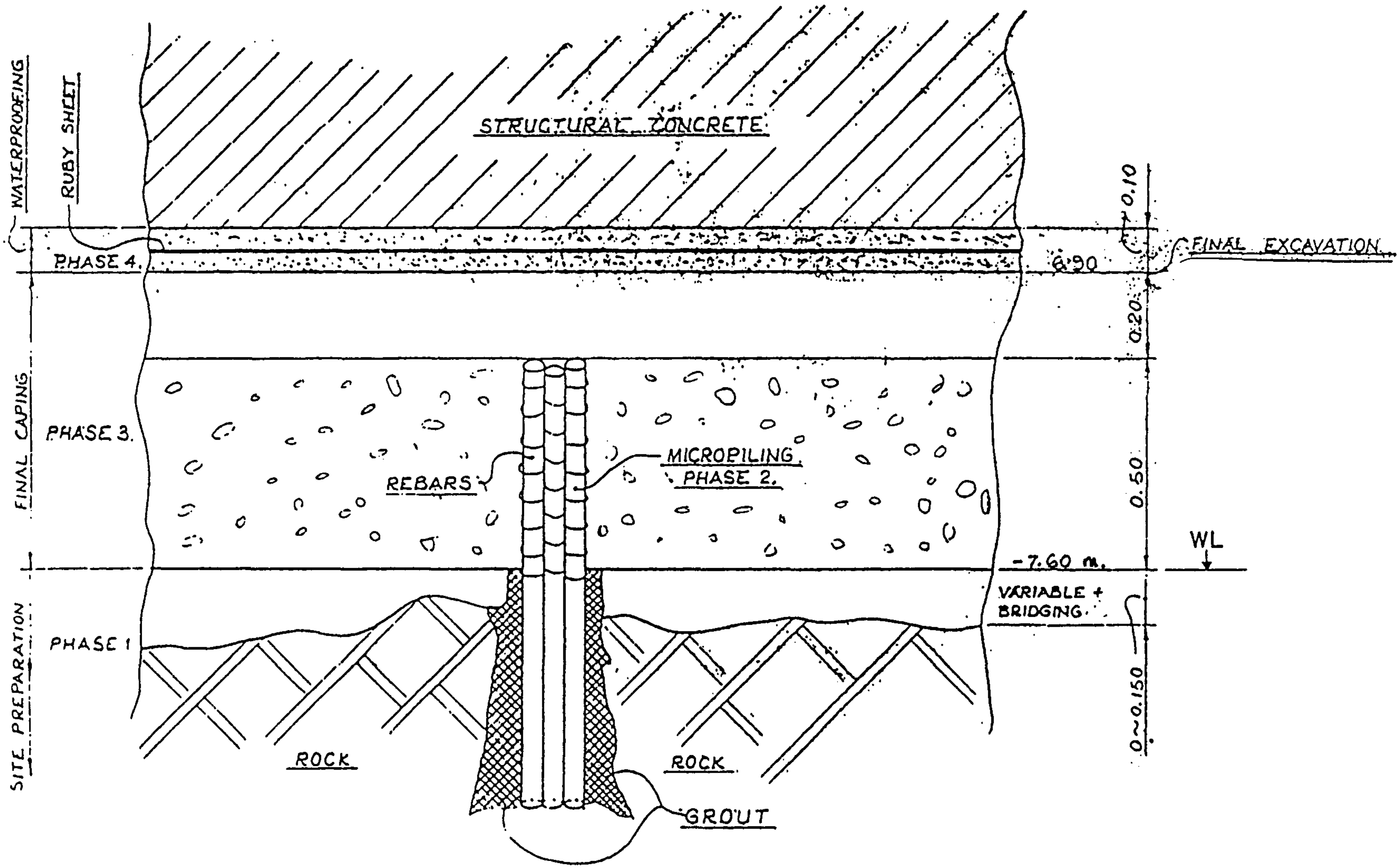
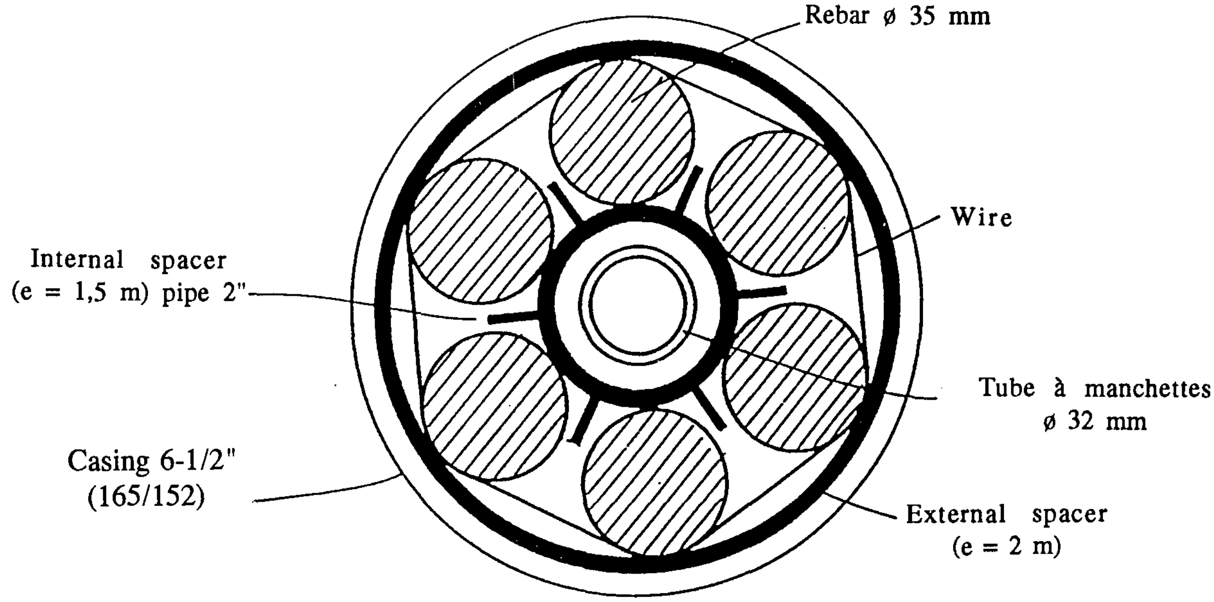


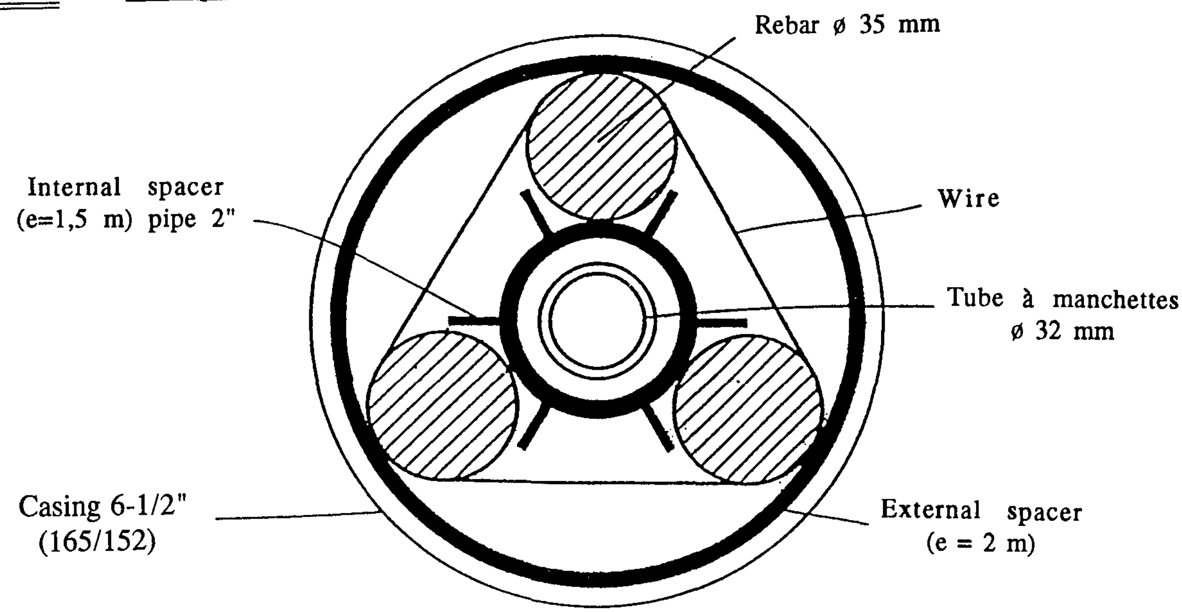
Figure 50. Typical section of micropiling, Korean Nuclear Unit 10, South Korea (Terrasol, 1984).

TYPE I   Micropile 6-1/2" with 6 ϕ 35 mm



1 inch = 25.4 mm

TYPE II   Micropile 6-1/2" with 3 ϕ 35 mm



1 in = 25.4 mm

Figure 51. Specifications for micropiling, Korean Nuclear Unit 10, South Korea (Terrasol, 1984).

grouting was conducted 24 to 48 hours later via tubes à manchette with a similar mix, but with w/c ratios from 1.0 to 0.5, and b/c ratios of 0.25 to 2 percent. Each sleeve, at 500-mm centers, was injected to the following acceptance criteria:

- Breakout pressure of 7 MPa or more.
- Final flow rate of 2 MPa with less than 100 kg of solids injection. Repeated injections were conducted at 24-hour intervals until these criteria were met.

This incremental construction procedure provided the basis for pile acceptability: the designers felt that any "global method" of pressure grouting (Types B or C) would have needed a later load test to verify acceptable installation. Table 27 summarizes the types of piles installed.

Testing and Performance. Six single piles were installed and tested in different soils with different grouting methods (table 28). The average borehole diameter was 180 mm. An 8-mm-thick steel tube, 1.7 m long, was placed in the upper part to guard against buckling. Prestressed anchors (also installed with the sleeved pipe method) were used as reaction, providing a maximum test-load capability of 1600 kN. As shown in table 29, all except one pile reached this load: G3 failed when a rebar connection failed at 1200 kN. The critical design values for k_p then followed (table 30). The factor k_p gives the movement, Y, at the top of the micropile for the applied load, P.

This work was undertaken in the middle 1980's and since then the facility has performed without problems related to the problem zone.

Table 27. Summary of production piles, Korean Nuclear Unit 10, South Korea (Terrasol, 1984).

	Vertical	Inclined	Total	Number of rebars
Type I (6 rebars)	96	207	303	1818
Type II (3 rebars)	41	6	47	141
Total	137	213	350	1959

Table 28. Details of test piles, Korean Nuclear Unit 10, South Korea (Terrasol, 1984).

	G2	G3	G4'	G5	G6	G7
Type of injection *	T.M.	T.M.	Glob	Glob	T.M.	T.M.
Type of rock or soil	H/MW	Clay + HW	HW	Clay + H/MW	Clay +HW	HW
Ultimate load (kN)	> 1600	1400	> 1600	> 1600	> 1600	> 1600
Critical load (kN)	> 1600	1400	> 1600	> 1600	> 1600	> 1600
K_p (MN/m)	330	200	420	280	280	340
$K_p \times B/A$ (kN)	70	80	80	80	80	80

HW - Highly weathered
M - Medium weathered

Table 29. Test pile performance summary, Korean Nuclear Unit 10, South Korea (Terrasol, 1984).

Micropile No	Critical load (kN)	Ultimate load (kN)	ΔH (800 kN) (mm)	ΔH (1600 kN) (mm)	K_p (MN/m)	K_p B/A (MN)
G2	>1600	>1600	2.93	8.07*	320	0.07
G3	1400**	1400**	3.69	——	200	0.08
G4.1	>1600	>1600	1.50	4.16	420	0.08
G5	>1600	>1600	2.94	7.43	280	0.08
G6	>1600	>1600	2.93	9.22	280	0.08
G7	>1600	>1600	2.21	5.96*	300	0.12

Table 30. Average K_p values for piles using tubes à manchette, Korean Nuclear Unit 10, South Korea (Terrasol, 1984).

	HW or H/MW (G2 - G7)	Clay + HW or Clay + H/MW (G3 - G6)
K_p (MN/m)	335	240
K_p x B/A (kN)	80	80

REFERENCES

American Society for Testing and Materials ASTM D1143-81. 1981. "Standard Method of Testing Piles Under Static Axial Compressive Load." Section 04. Vol. 04.08.

Attwood, S. 1987. "Pali Radice: Their Uses in Stabilizing Existing Retaining Walls and Creating Case In Situ Retaining Structures." *Ground Engineering*. Vol. 20. No. 7. pp. 23-27.

Barley, A.D. and M.A. Woodward. 1992. "High Loading of Long Slender Minipiles." *Proceedings ICE Conference, Piling: European Practice and Worldwide Trends*. Thomas Telford, London. pp. 131-136.

Blondeau, F., F. Schlosser, and T.V. Nhiem. 1987. "Reinforcement of a Large Fault by Micropiles, Beneath a Nuclear Plant Group Table Footing." ENPC Symposium, Soil Structure Interactions. May.

Bruce, D.A. 1988. "Developments in Geotechnical Construction Processes for Urban Engineering." *Civil Engineering Practice*. Vol. 3. No. 1. Spring. pp. 49-97.

Bruce, D.A. 1988, 1989. "Aspects of Minipiling Practice in the United States." *Ground Engineering*. Vol. 21. No. 8. pp. 20-33 and Vol. 22. No. 1. pp. 35-39.

Bruce, D.A. 1992. "Recent Progress in American Pin Pile Technology." *Proceedings ASCE Conference: Grouting, Soil Improvement, and Geosynthetics*. New Orleans, Louisiana. Feb. 25-28. pp. 765-777.

Bruce, D.A. 1992a. "Recent Advances in American Pinpiling Practice." *Proceedings ICE Conference, Piling: European Practice and Worldwide Trends*. London. April 7-9. pp. 123-130.

Bruce, D.A. 1994. "Small-Diameter Cast-in-Place Elements for Load Bearing and In Situ Earth Reinforcement." In *Ground Control and Improvement* by P.P. Xanthakos, L.W. Abramson, and D.A. Bruce. John Wiley and Sons. Chapter 6. 87 pp.

Bruce, D.A., R. Bjorhovde, and J.R. Kenny. 1993. "Fundamental Tests on the Performance of High Capacity Pin Piles^{sm}." *Proceedings 18th Annual DFI Conference*. Pittsburgh, Pennsylvania. October 18-20. 33 pp.

Bruce, D.A. and Gemme, R. 1992. "Current Practice in Structural Underpinning Using Pinpiles." *Proceedings New York Met. Section ASCE Seminar*. New York. April 21-22. 46 pp.

Bruce, D.A., C.H. Hall, and R.E. Triplett. 1992. "Structural Underpinning by Pinpiles." *Proceedings 17th DFI Annual Meeting*. New Orleans, Louisiana. October 21-23. pp. 49-80.

Bruce, D.A., J.D. Ingle, and M.R. Jones, 1985. "Recent Examples of Underpinning Using Mini Piles." Second International Conference on Structural Faults and Repairs. London. April-May. 11 pp.

Bruce, D.A., S.L. Pearlman, and J.H. Clark. 1990. "Foundation Rehabilitation of the Pocomoke River Bridge, Maryland, Using High Capacity Preloaded Pinpiles." *Proceedings 7th Annual International Bridge Conference.* Pittsburgh, Pennsylvania. June 18-20. Paper IBC-90-42. 9 pp.

Bruce, D.A., S.L. Pearlman, and J.R. Wolosick. 1993. "Support of Bridge Piers by High-Capacity Pin Piles^{sm}." *Proceedings National Earthquake Conference.* Memphis, Tennessee. May 3-5. Vol. II. pp. 563-573.

Bruce, D.A. and C.K. Yeung. 1983. "A Review of Minipiling, With Particular Regard to Hong Kong Applications." *Hong Kong Engineer.* June 1984. pp. 31-54, (presented at the Institution in November 1983).

Commonwealth of Massachusetts. 1984. *State Building Code.* Fourth Edition.

DSI. 1994. Reconstruction and Conversion to "Grand Hotel Taschenberg Palais Kempinski." Promotional literature. Munich, Germany.

DSI. 1994a. Dywidag GEWI Piles. "Pencil Piles Optimize Stability Against Seismic Forces." Promotional literature. Munich, Germany.

Groneck, P.B., D.A. Bruce, J.H. Greenman, and G. Bingham. 1993. "Pin Piles Save Silos." *Civil Engineering.* 63 (9). September. pp 66-68.

Leinenkugel, H.J. 1976. "Deformations und Festigkeitsver Halten Biniger Erdstoff." *Experimentelle Erges-Bnesse Under Ihre Physikalische Deutung Veroffentil.* Insst. f. Bodenmech. u. Felsmech. 66. Karlsruhe.

Littlejohn, G.S. and D.A. Bruce, 1977. "Rock Anchors - State of the Art." Foundation Publications Ltd. Brentwood, Essex, England. 50 pp.

Mason, J.A. 1993. "Caltrans Full-Scale Lateral Load Test of a Driven Pile Foundation in Soft Bay Mud. Preliminary Results." California Department of Transportation (Caltrans) Division of Structures. Sacramento, California. 31 pp. plus figures.

Munfakh, G.A. and N.N. Soliman. 1987. "Back on Track at Coney." *Civil Engineering.* ASCE, Vol. 59. No. 12. pp. 58-60.

New York State Department of Transportation. 1977. "Static Pile Load Test Manual." Official Issuance No. 741-3. NYSDOT, Albany. 41 pp.

Palmerton, J.B. 1984. *Stabilization of Moving Land Masses by Cast-in-Place Piles.* Geotech Lab, USACOE, WES. Vicksburg, Mississippi. Final Report GL-84-4. 134 pp.

Pearlman, S.L., B.D. Campbell, and J.L. Withiam. 1992. "Slope Stabilization Using In Situ Earth Reinforcements." ASCE Specialty Conference on Stability and Performance of Slopes and Embankments II. Berkeley, California. June 29 -July 1. 16 pp.

Pearlman, S.L., J.R. Wolosick, and P.B. Groneck. 1993. "Pinpiles for Seismic Rehabilitation of Bridges." *Proceedings 10th International Bridge Conference.* Pittsburgh, Pennsylvania. June 14-16. 12 pp.

Reese, L., A. Awoshika, P. Lam, and S. Wang. 1987. *Documentation of Computer Program, GROUP1, Analysis of a Group of Piles Subjected to Axial and Lateral Loadings.* Ensoft, Inc. Austin, Texas.

Rodio and Co. 1993. *Brindisi ENEL Power Plant: High-Capacity Micropiles.* Internal Technical Report. Milan, Italy.

Terrasol. 1984. KNU-Turbine Hall: Weak-Zone Reinforcement by Micropiling. Reports DE.83.71/03 and DE.83.71/04.

Turner, M.J. and D.J. Wilson. 1990. "Micropiles at Albert Docks, Liverpool." *Ground Engineering*, Vol. 23, No. 5, September. pp. 40-44.

Winter, H., W. Schwartz, and G. Gudehus. 1983. "Stabilization of Clay Slopes by Piles." *Proceedings 8th European Conference on Soil Mechanics and Foundation Engineers.* Vol. 2. Helsinki. pp. 545-550.

U.S. DEPARTMENT OF COMMERCE
Technology Administration
National Technical Information Service
Springfield, VA 22161 (703) 487-4650