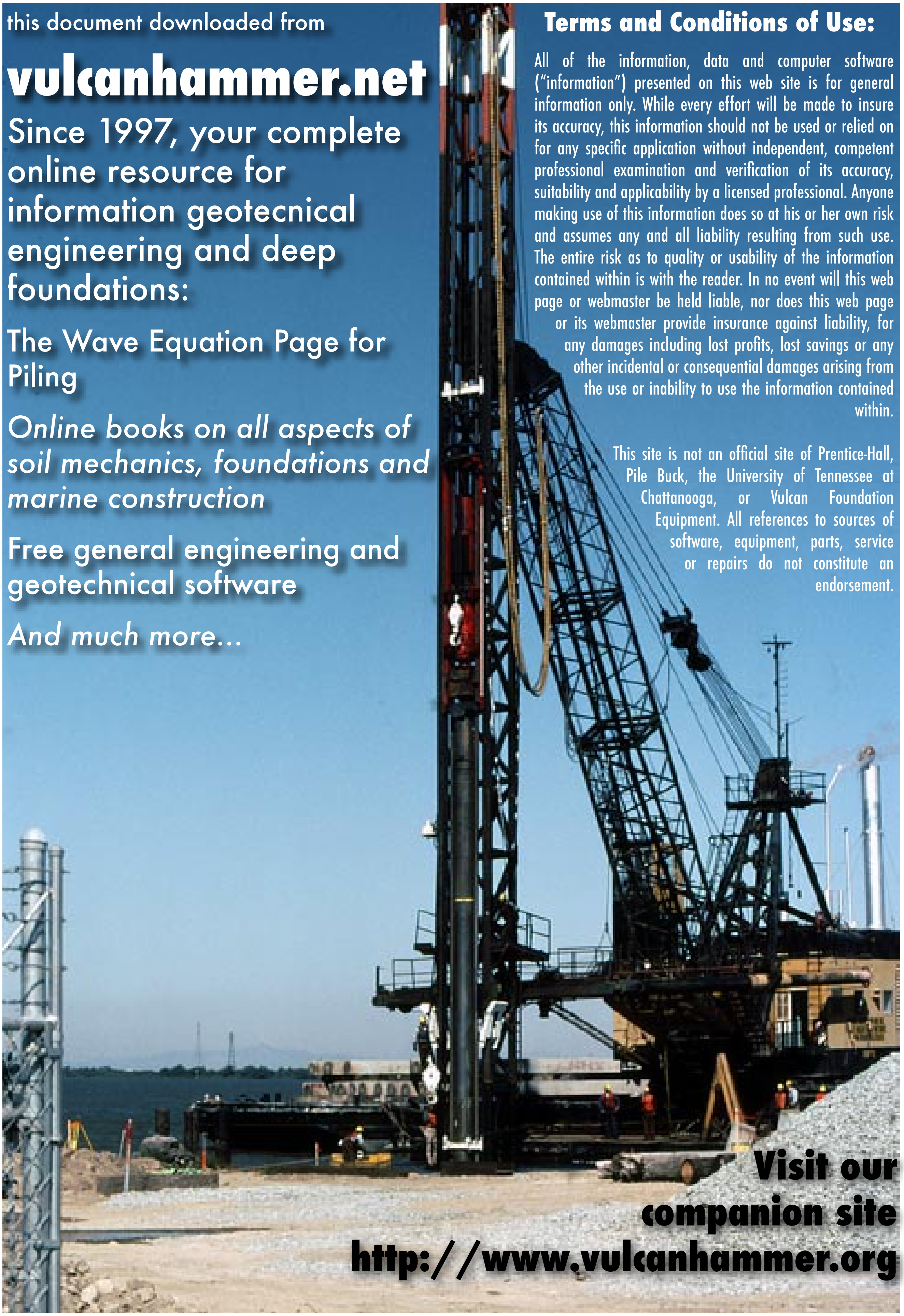

US Department
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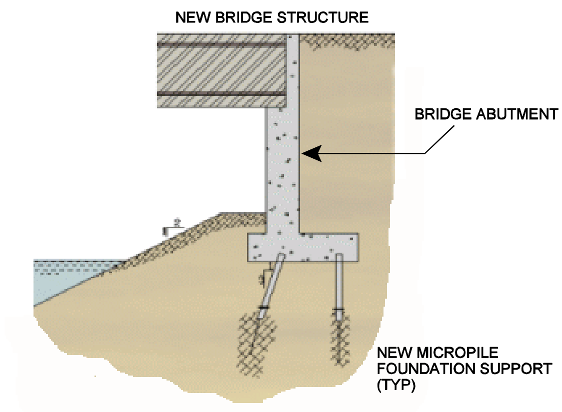
Federal Highway
Administration

Priority Technologies
Program

MICROPILE DESIGN AND CONSTRUCTION GUIDELINES



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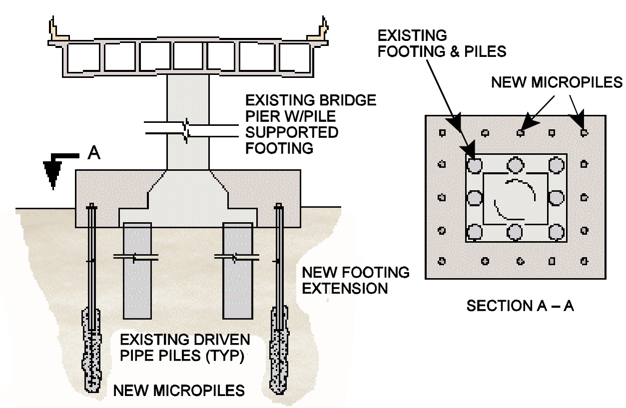

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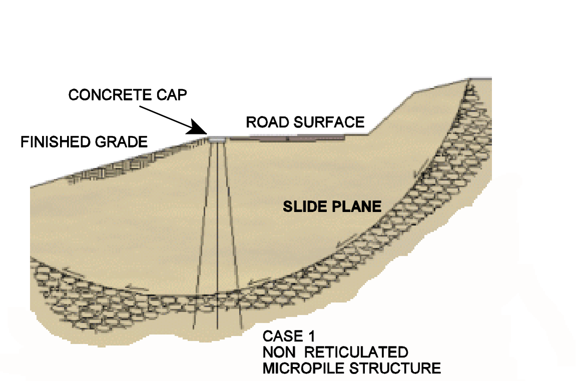

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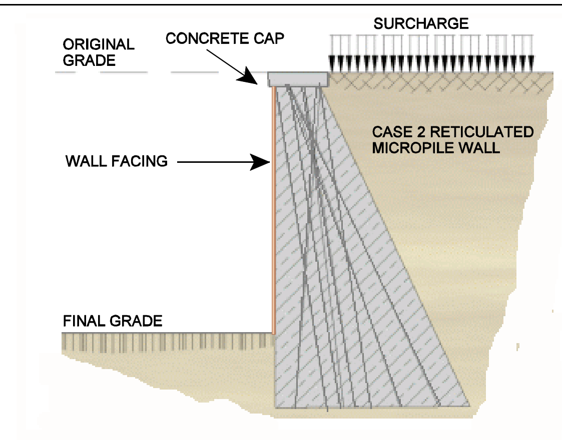

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16. Abstract

The use of micropiles has grown significantly since their conception in the 1950s, and in particular since the mid-1980s. Micropiles have been used mainly as elements for foundation support to resist static and seismic loading conditions and less frequently as in-situ reinforcements for slope and excavation stability. Many of these applications are suitable for transportation structures.

Implementation of micropile technology on U.S. transportation projects has been hindered by lack of practical design and construction guidelines. In response to this need, the FHWA sponsored the development of this Micropile Design and Construction Guidelines Implementation Manual. Funding and development of the manual has been a cooperative effort between FHWA, several U.S. micropile specialty contractors, and several State DOT's. This manual is intended as a "practitioner-oriented" document containing sufficient information on micropile design, construction specifications, inspection and testing procedures, cost data, and contracting methods to facilitate and speed the implementation and cost effective use of micropiles on United States transportation projects.

Chapter 1 provides a general definition and historic framework of micropiles. Chapter 2 describes the newly developed classifications of micropile type and application. Chapter 3 illustrates the use of micropiles for transportation applications. Chapter 4 discusses construction techniques and materials. Chapter 5 presents design methodologies for structural foundation support for both Service Load Design (SLD) and Load Factor Design (LFD). Chapter 6, which was supposed to present a design methodology for slope stabilization, is not included in this version. Chapter 7 describes micropile load testing. Chapter 8 reviews construction inspection and quality control procedures. Chapter 9 discusses contracting methods for micropile applications. Chapter 10 presents feasibility and cost data. Appendix A presents sample plans and specifications for Owner Controlled Design with Contractor Design Build of the micropiles, and/or micropiles and footings.

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PREFACE

The long-term performance of micropiles has been proven after 25+ years of use in Europe and North America. The purpose of this "practitioner-oriented" manual is to facilitate the implementation of micropile technology into American transportation design and construction practice and to provide guidance for selecting, designing and specifying micropiles for those applications to which it is technically suited and economically attractive. A comprehensive review of current design and construction methods has been made and results compiled into a guideline procedure. The intent of presenting the guideline procedure is to help ensure that agencies adopting use of micropile technology follow a safe, rational procedure from site investigation through construction.

Chapter 1 provides a general definition and historic framework of micropiles. Chapter 2 describes the newly developed classifications of micropile type and application. Chapter 3 illustrates the use of micropiles for transportation applications. Chapter 4 discusses construction techniques and materials. Chapter 5 details the design methodologies for structural foundation support and includes worked design examples. Chapter 6 was intended to cover slope stabilization details, but due to a lack of consensus on design methods, this chapter is not included and is still under preparation. When finished, Chapter 6 will be made available as a supplement to this manual. Chapter 7 describes pile load testing. Chapter 8 reviews construction inspection and quality control procedures. Chapter 9 discusses contracting methods for micropile applications. Chapter 10 presents feasibility and cost data. Appendix A presents guideline plans and specifications for Owner Controlled Design with Contractor Design Build of the micropiles, and/or micropiles and footings.

MANUAL LIMITATIONS

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Schnabel Foundation Company

Special thanks are due particularly to Mr. James Keeley (FHWA, Central Federal Lands Division), for co-authoring Chapter 5 on "Structural Foundations; Mr. Al Kilian (FHWA, Western Federal Lands Division) and Mr. Ron Chassie (FHWA, Region 10), for providing the constant assistance, encouragement and funding support that made this manual possible and to Dr. Sunil Sharma (University of Idaho) for technical assistance and for assembling and publishing the final version. An interactive, CD-version of this manual is currently being developed by Dr. Sharma for release in Fall 2000.

Special acknowledgment is made to the following *volunteer* Technical Working Group members (listed in alphabetical order) for significant contributions to the manual and for providing the QA/QC to help assure that the design and construction guidelines and guide specifications presented in the manual are sound, rational, and practitioner oriented:

Tom Armour	DBM Contractors, Inc.	(Lead Coordinator)
Don Chadbourne	Washington State Department of Transportation	

Ron Chassie	FHWA-Region 10
Kerry Cook	FHWA-WFLHD
Paul Groneck	DBM Contractors, Inc.
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Special recognition is given to Dr. Fernando Lizzi of Italy, the "father" of micropiles and to Dr. Donald Bruce, formerly Technical Director with Nicholson Construction Company and currently a Principal with ECO Geosystems, for preparing the FHWA State-of-Practice Review Reports from which some of the information in this manual has been excerpted, and for his tireless efforts as a "champion" of *Micropile Technology* in North America.

Thanks are also due to the members of the Deep Foundation Institute (DFI) Micropile Technical Committee for their thorough technical review of the final draft manual.

The many figures in this manual were originally prepared by DBM Contractors, Inc., and were subsequently enhanced and colorized by Dr. Sharma and Rob Barnes of the University of Idaho. Lastly, and most importantly, heartfelt thanks are given to Ms. Mary Ellen Bruce, who developed Chapters 1-3, Mr. Don Olson (FHWA, retired), who provided technical assistance with Chapter 5 and Ms. Patty Romano of DBM Contractors, Inc., who typed the original version of the manual.

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ENGLISH TO METRIC (SI) CONVERSION FACTORS

The primary metric (SI) units used in civil and structural engineering are:

- meter (m)
- kilogram (kg)
- second (s)
- Newton (N)
- Pascal (Pa = N/m^2)

The following are the conversion factors for units presented in this manual:

Quantity	From English Units	To Metric (SI) Units	Multiply by	For Aid to Quick Mental Calculations
Mass	lb	kg	0.453592	1 lb(mass) = 0.5kg
Force	lb kip	N kN	4.44822 4.44822	1 lb(force) = 4.5N 1 kip(force) = 4.5kN
Force/unit length	plf klf	N/m kN/m	14.5939 14.5939	1 plf = 14.5N/m 1 klf = 14.5kN/m
Pressure, stress, modulus of elasticity	psf ksf psi ksi	Pa kPa kPa MPa	47.8803 47.8803 6.89476 6.89476	1 psf = 48 Pa 1 ksf = 48 kPa 1 psi = 6.9 kPa 1 ksi = 6.9 MPa
Length	inch foot foot	mm m mm	25.4 0.3048 304.8	1 in = 25 mm 1 ft = 0.3 m 1 ft = 300 mm
Area	square inch square foot square yard	mm^2 m^2 m^2	645.16 0.09290304 0.83612736	1 sq in = 650 mm^2 1 sq ft = 0.09 m^2 1 sq yd = 0.84 m^2
Volume	cubic inch cubic foot cubic yard	mm^3 m^3 m^3	16386.064 0.0283168 0.764555	1 cu in = 16,400 mm^3 1 cu ft = 0.03 m^3 1 cu yd = 0.76 m^3

A few points to remember:

1. In a **"soft"** conversion, an English measurement is mathematically converted to its **exact** metric equivalent.
2. In a **"hard"** conversion, a new **rounded**, metric number is created that is convenient to work with and remember.
3. Use only the meter and millimeter for length (avoid centimeter).
4. The Pascal (Pa) is the unit for pressure and stress (Pa and N/m^2).
5. Structural calculations should be shown in MPa or kPa.
6. A few basic comparisons worth remembering to help visualize metric dimensions are:
    - One mm is about 1/25 inch or slightly less than the thickness of a dime.
    - One m is the length of a yardstick plus about 3 inches.
    - One inch is just a fraction (1/64 inch) longer than 25 mm (1 inch = 25.4 mm).
    - Four inches are about 1/16 inch longer than 100 mm (4 inches = 101.6 mm).
    - One foot is about 3/16 inch longer than 300 mm (12 inches = 304.8 mm).

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CHAPTER 1

INTRODUCTION

1.A PURPOSE AND SCOPE OF MANUAL

The use of micropiles has grown significantly since their conception in the 1950s, and in particular since the mid-1980s. Micropiles have been used mainly as elements for foundation support to resist static and seismic loading conditions, and as in-situ reinforcements for slope and excavation stability. Many of these applications are for transportation structures.

In 1993, the Federal Highway Administration (FHWA) sponsored a desk study of the state-of-the-practice of micropiles. The research group for this study consisted of contractors, consultants, academics, and clients. The document produced from this study, entitled *Drilled and Grouted Micropiles – State-of-the-Practice Review* (Federal Highway Administration, 1997) provides a comprehensive international review and detailed analysis of available research and development results, laboratory and field testing data, design methods, construction methodologies, site observations, and monitored case studies. As part of this study, the limitations and uncertainties in the current state-of-the-practice were evaluated, and further research needs were assessed. One of the highlighted needs, voiced mainly by representatives of State Departments of Transportation, was a manual of design and construction guidelines intended for use by practicing, highway agency, geotechnical and structural engineers.

In response to this need, the FHWA sponsored the development of this *Micropile Design and Construction Guidelines, Implementation Manual.* Funding and development of the manual was a cooperative effort between FHWA, several U.S. micropile specialty contractors, and several state DOT's (See Acknowledgement Section). This manual is intended to be a "practitioner-oriented" document containing sufficient information on micropile design, construction specifications, inspection and testing procedures, cost data, and contracting methods to facilitate and speed the implementation and cost-effective use of micropiles on United States transportation projects.

Chapter 1 provides a general definition and historic framework of micropiles. Chapter 2 describes the newly developed classifications of micropile type and application. Chapter 3 illustrates the use of micropiles for transportation applications. Chapter 4 discusses construction techniques and materials. Chapter 5 details the design methodologies for structural foundation support and includes worked design examples. **Chapter 6 was intended to cover slope stabilization details, but due to a lack of consensus on design procedures, this chapter is not included and is still under preparation.** Chapter 7 describes pile load testing. Chapter 8 reviews construction inspection and quality control procedures. Chapter 9 discusses contracting methods for micropile applications. Chapter 10 presents feasibility and cost data. Appendix A presents sample plans and specifications for owner-controlled design with contractor design-build of the micropiles.

A basic introduction to micropiles can also be found in the December 1995 issue of *Civil Engineering* magazine, published by the American Society of Civil Engineers (Bruce et al., 1995), in an article entitled "A Primer on Micropiles." This article, authored by representatives of the FHWA state-of-the-practice research group, includes basic characteristics and definitions of micropiles, classifications of applications, and a discussion of the micropiling market.

1.B MICROPILE DEFINITION AND DESCRIPTION

Piles are divided into two general types: displacement piles and replacement piles (Fleming et al, 1985). Displacement piles are members that are driven or vibrated into the ground, thereby displacing the surrounding soil laterally during installation. Replacement piles are placed or constructed within a previously drilled borehole, thus replacing the excavated ground.

A micropile is a small-diameter (typically less than 300 mm), drilled and grouted replacement pile that is typically reinforced. A micropile is constructed by drilling a borehole, placing reinforcement, and grouting the hole as illustrated in Figure 1-1. Micropiles can withstand axial and/or lateral loads, and may be considered a substitute for conventional piles or as one component in a composite soil/pile mass, depending upon the design concept employed. Micropiles are installed by methods that cause minimal disturbance to adjacent structures, soil, and the environment. They can be installed in access- restrictive environments and in all soil types and ground conditions. Micropiles can be installed at any angle below the horizontal using the same type of equipment used for ground anchor and grouting projects.

Since the installation procedure causes minimal vibration and noise and can be used in conditions of low headroom, micropiles are often used to underpin existing structures. Specialized drilling equipment is often required to install the micropiles from within existing basement facilities.

Most of the applied load on conventional cast-in-place replacement piles is structurally resisted by the reinforced concrete; increased structural capacity is achieved by increased cross-sectional and surface areas. Micropile structural capacities, by comparison, rely on high-capacity steel elements to resist most or all of the applied load. These steel elements have been reported to occupy as much as one-half of the hole volume. The special drilling and grouting methods used in micropile installation allow for high grout/ground bond values along the grout/ground interface. The grout transfers the load through friction from the reinforcement to the ground in the micropile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter, any end-bearing contribution in micropiles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e., pressure grouting or gravity feed. The role of the drilling method is also influential, although less well quantified.

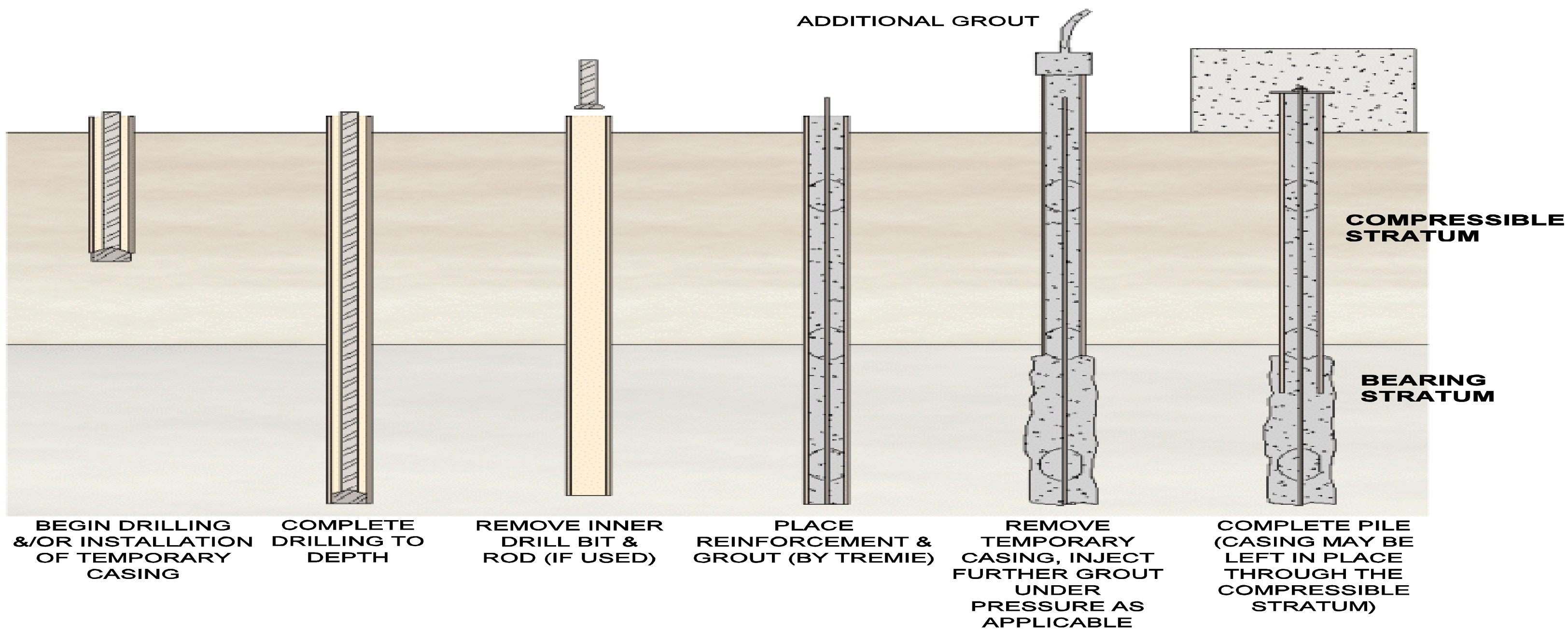

Figure 1 - 1. Micropile Construction Sequence using Casing

1.C HISTORICAL BACKGROUND

Micropiles were conceived in Italy in the early 1950s, in response to the demand for innovative techniques for underpinning historic buildings and monuments that had sustained damage with time, and especially during World War II. A reliable underpinning system was required to support structural loads with minimal movement and for installation in access-restrictive environments with minimal disturbance to the existing structure. An Italian specialty contractor called Fondedile, for whom Dr. Fernando Lizzi was the technical director, developed the *palo radice*, or root pile, for underpinning applications. The *palo radice* is a small-diameter, drilled, cast-in-place, lightly reinforced, grouted pile. The classic arrangement of *pali radice* for underpinning is shown in Figure 1-2.

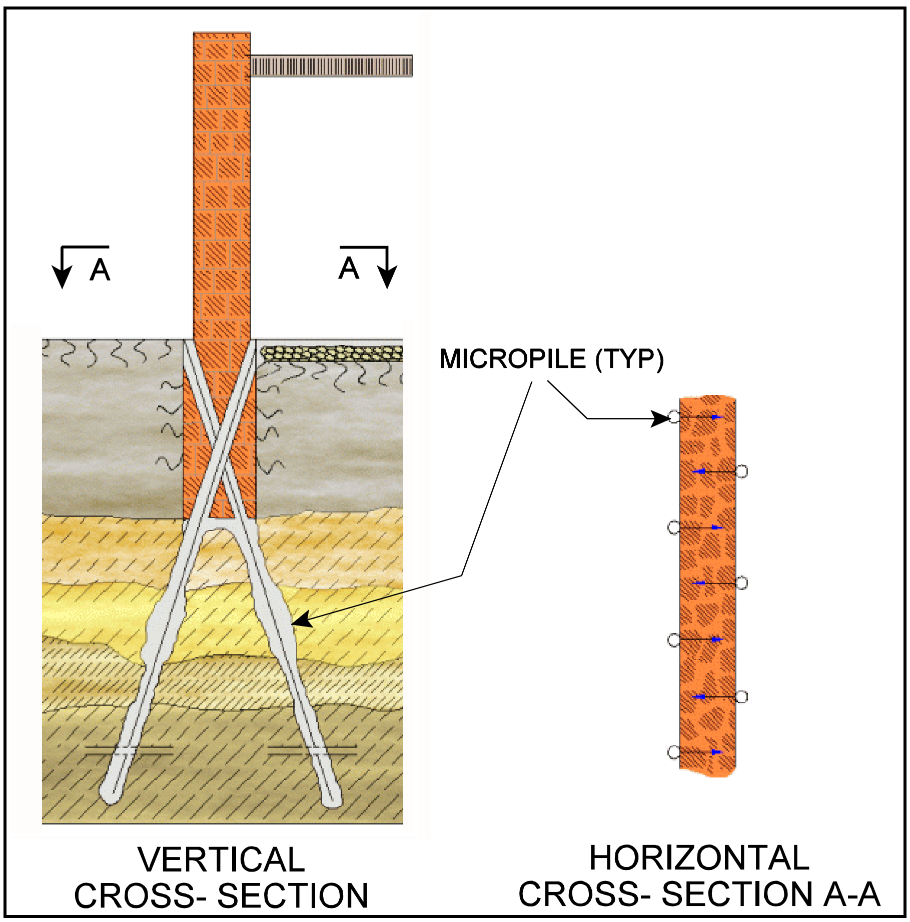

Figure 1 - 2. Classical Arrangement of Root Piles for Underpinning

Although steel was in short supply in postwar Europe, labor was inexpensive, abundant, and often of high mechanical ability. Such conditions encouraged the development of these lightly reinforced, cast-in-place root pile elements, largely designed and installed by specialty contractors on a design-build basis. Load testing on these new root piles measured capacities in excess of 400 kN, although the nominal design capacity-based on contemporary conventional bored pile design methodologies- suggested loads of less than 100 kN. Direct full-scale load tests were performed at relatively little cost, fostering the acquisition and publication of a wealth of testing information. No grout/ground bond failures were recorded during these early tests.

The use of root piles grew in Italy throughout the 1950s. Fondedile introduced the technology in the United Kingdom in 1962 for the underpinning of several historic structures, and by 1965, it was being used in Germany on underground urban transportation schemes. For proprietary reasons, the term “micropile” replaced “root pile” at that time.

Initially, the majority of micropile applications were structural underpinning in urban environments. Starting in 1957, additional engineering demands resulted in the introduction of systems of *reticoli di pali radice* (reticulated root piles). Such systems comprise multiple vertical and inclined micropiles interlocked in a three-dimensional network, creating a laterally confined soil/pile composite structure (Figure 1-3). Reticulated micropile networks were applied for slope stabilization, reinforcement of quay walls, protection of buried structures, and other soil and structure support and ground reinforcement applications.

Other proprietary micropiles were developed in Switzerland and Germany, and the technologies were quickly exported overseas by branches or licensees of the originating contractors. The Far East soon became a major market.

Fondedile introduced the use of micropiles in North America in 1973 through a number of underpinning applications in the New York and Boston areas. The micropile technology did not grow rapidly in the United States, however, until the mid-1980s, after which time an abundance of successful published case histories, consistent influence by specialty contractors,

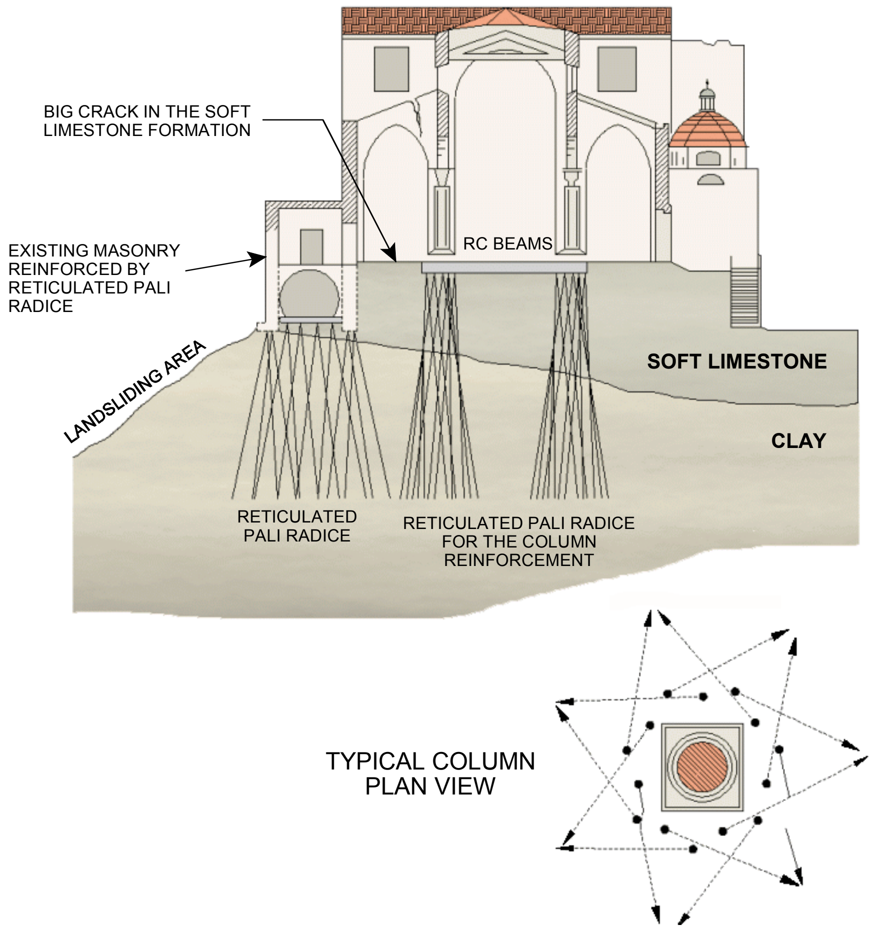

Figure 1 - 3. Typical Network of Reticulated Micropiles

and the growing needs of consultants and owners working in old urban environments overcame the skepticism and concerns of the traditional East Coast piling market (Bruce, 1988.). The abundance of relatively cheap labor, the shortage of steel, and the need for reconstruction programs in urban environments had all promoted the growth and use of micropiles in Europe. Conversely, the slower and later growth of micropile usage in North America is reflective of the abundance of cheap steel, relatively high labor costs, and the need for capital works projects typically outside of the cities. These circumstances fostered the growth of the comparatively low-technology, driven-pile techniques governed by prescriptive specifications. Today, construction costs and technical demands are similar throughout the world and so continue to foster the growth of micropile demand, largely through geotechnical contractors with design-build capabilities.

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CHAPTER 2

MICROPILE CLASSIFICATION SYSTEM

2.A MICROPILE TYPES IN CURRENT USE

One of the most fundamental achievements of the research team that developed the Micropile State-of-the-Practice Report (Federal Highway Administration, 1997) was the new classification criteria for micropiles. This is important because it resolved disagreement within the industry about fundamental differences in design and behavior of elements that are visually similar and constructed with common equipment, materials and techniques.

The State-of-the-Practice Report includes a micropile classification system based on two criteria: 1) philosophy of behavior (design) and 2) method of grouting (construction). The philosophy of behavior dictates the method employed in designing the micropile. The method of grouting defines the grout/ground bond capacity, which is generally the major constructional control over pile capacity. The classification system consists of a two-part designation: *a number*, which denotes the micropile behavior (design), and *a letter*, which designates the method of grouting (construction).

2.A.1 Design Application Classification

The design of an individual or group of micropiles differs greatly from that of a network of closely spaced reticulated micropiles. This led to the definition of CASE 1 micropile elements, which are loaded directly and where the pile reinforcement resists the majority of the applied load (Figure 2-1). CASE 2 micropile elements circumscribes and internally reinforces the soil to make a reinforced soil composite that resists the applied load (Figure 2-2). This is referred to as a reticulated pile network.

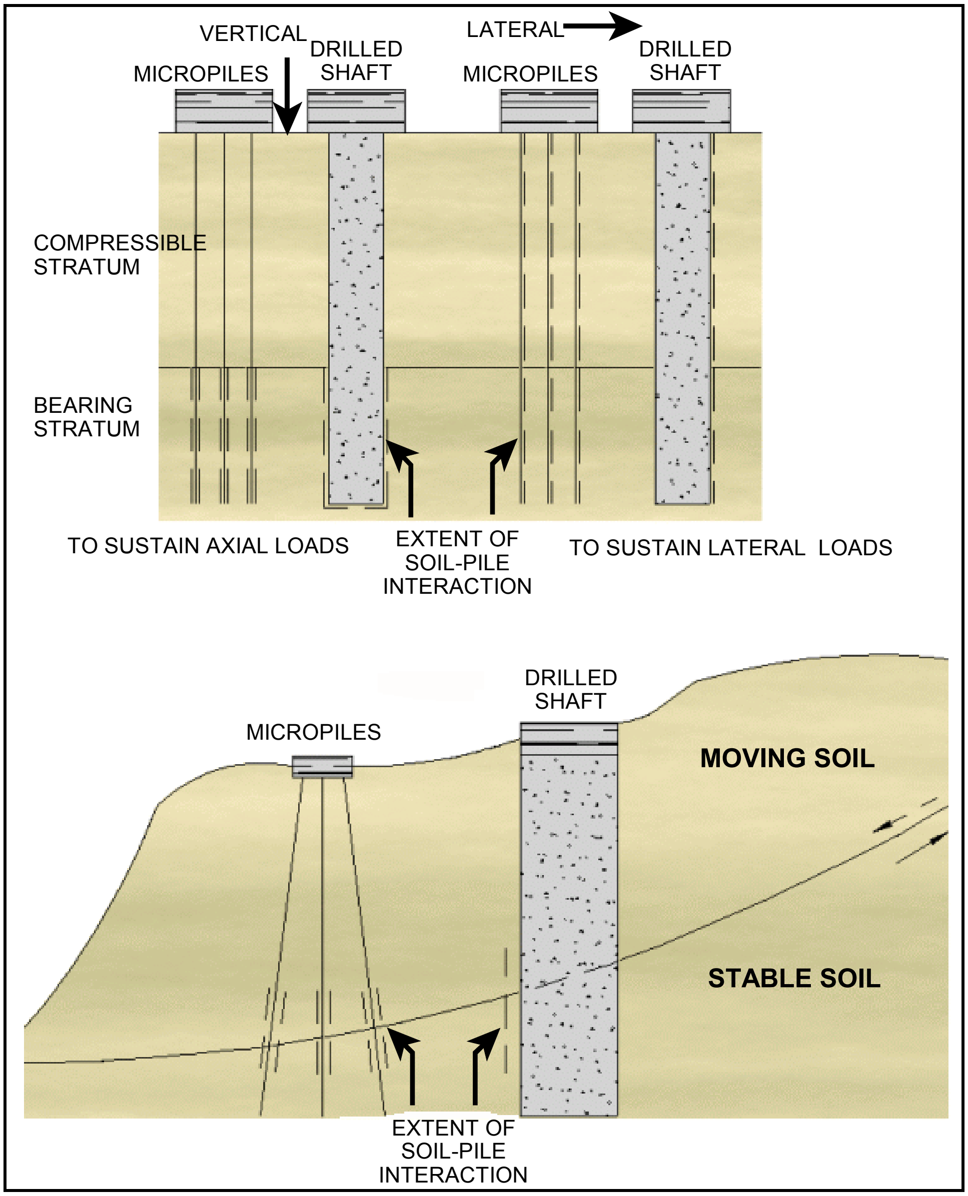

Figure 2 – 1. CASE 1 Micropiles (Directly Loaded)

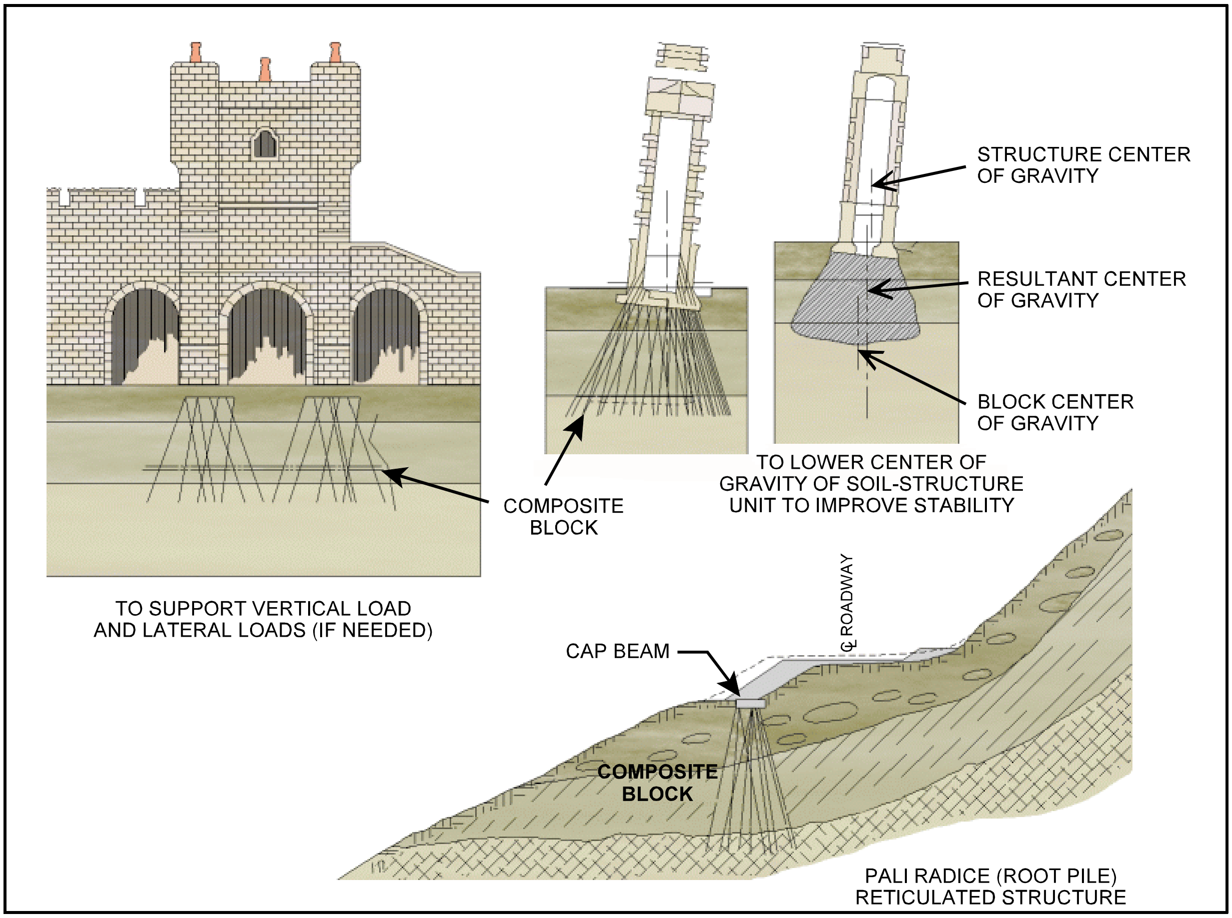

Figure 2 – 2. CASE 2 Micropiles – Reticulated Pile Network with Reinforced Soil Mass Loaded or Engaged

CASE 1 micropiles can be used as substitutes for more conventional types of piles to transfer structural loads to a deeper, more competent or stable stratum. Such directly loaded piles, *whether for axial or lateral loading conditions*, are referred to as CASE 1 elements. The load is primarily resisted structurally by the steel reinforcement and geotechnically by the grout/ground bond zone of the individual piles. At least 90 percent of all international applications to date, and virtually all of the projects in North America, have involved CASE 1 micropiles. Such piles are designed to act individually, although, they may be installed in groups. Typical arrangements of CASE 1 micropiles are illustrated in Figure 2-3.

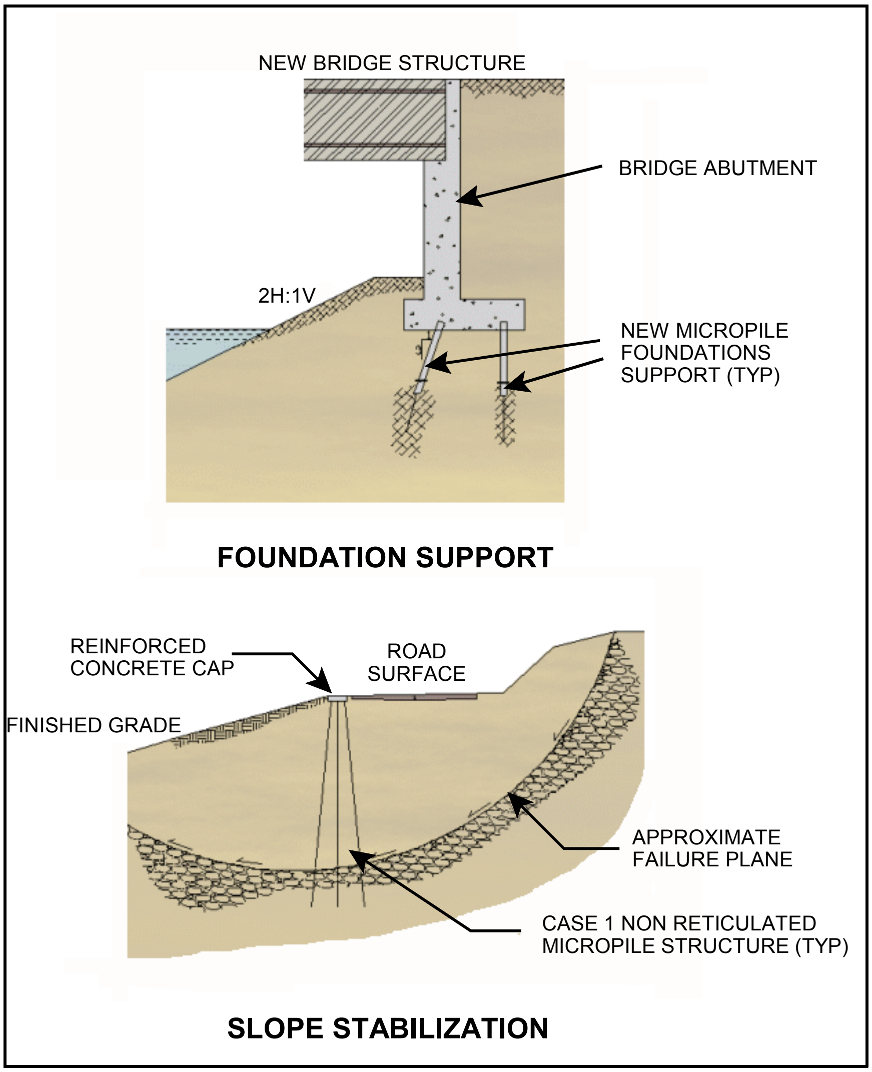

Figure 2 – 3. CASE 1 Micropile Arrangements

The remaining applications involve networks of reticulated micropiles as components of a reinforced soil mass, which is used for stabilization and support. These micropiles are referred to as CASE 2 elements. The structural loads are applied to the entire reinforced soil mass, as opposed to individual piles. CASE 2 micropiles are lightly reinforced because they are not individually loaded as CASE 1 elements. They serve to circumscribe and then to internally strengthen the reinforced soil composite. A typical network of reticulated micropiles is illustrated in Figure 2-4.

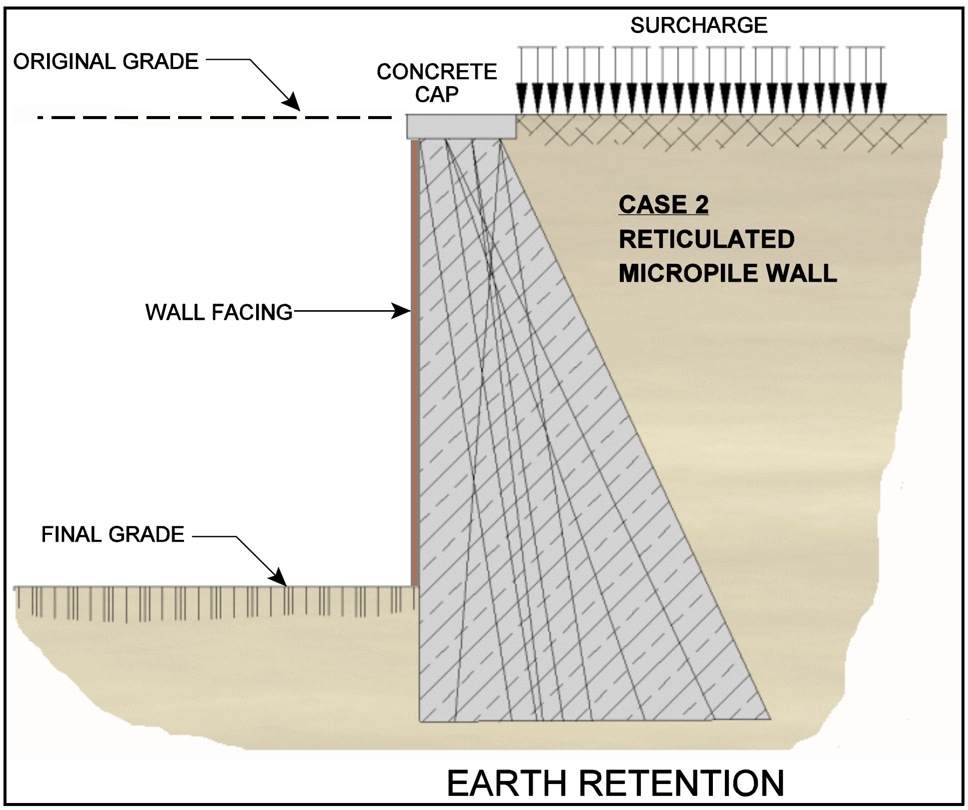

Figure 2 – 4. CASE 2 Micropile Arrangements

There are combination design philosophies between CASE 1 and CASE 2 micropiles. An example is the case of a row of micropiles installed throughout a failure plane to achieve slope stabilization. Recent research (Pearlman, et al, 1992) suggests that pile/ground interaction only occurs near the slide plane. In this situation, the pile acts as a CASE 1 element because it directly resists the load. Above the failure plane, the pile group does add a certain degree of continuity to the reinforced soil composite structure. This behavior is of CASE 2 type. Therefore, this example is between CASE 1 and CASE 2.

This philosophy of behavior (design) of an individual CASE 1 micropile is the same as that of a group of CASE 1 micropiles. A group of CASE 1 elements is defined as a closely spaced (typically parallel) arrangement of micropiles, each of which will be loaded directly. The behavior and design approach of a group of CASE 1 elements should not be confused with those of a reticulated network, although their geometries may appear to be similar. Group effects on micropile capacities are discussed in Chapter 5. Design methodologies for individual CASE 1 elements and groups of CASE 1 elements are discussed in Chapters 5 and 6. **Design methodologies for networks of CASE 2 elements are beyond the scope of this manual.**

2.A.2 Construction Type Classification

The method of grouting is generally the most sensitive construction control over grout/ground bond capacity. Grout/ground bond capacity varies directly with the grouting method. The second part of the micropile classification consists of a letter designation (A through D) based primarily on the method of placement and pressure under which grouting is used during construction. The use of drill casing and reinforcement define sub-classifications. The classification is shown schematically in Figure 2-5.

Type A: The type A classification indicates that grout is placed under gravity head only. Sand-cement mortars, as well as neat cement grouts, can be used because the grout column is not pressurized. The pile hole may be underreamed to increase tensile capacity, although this technique is not common or used with any other pile type.

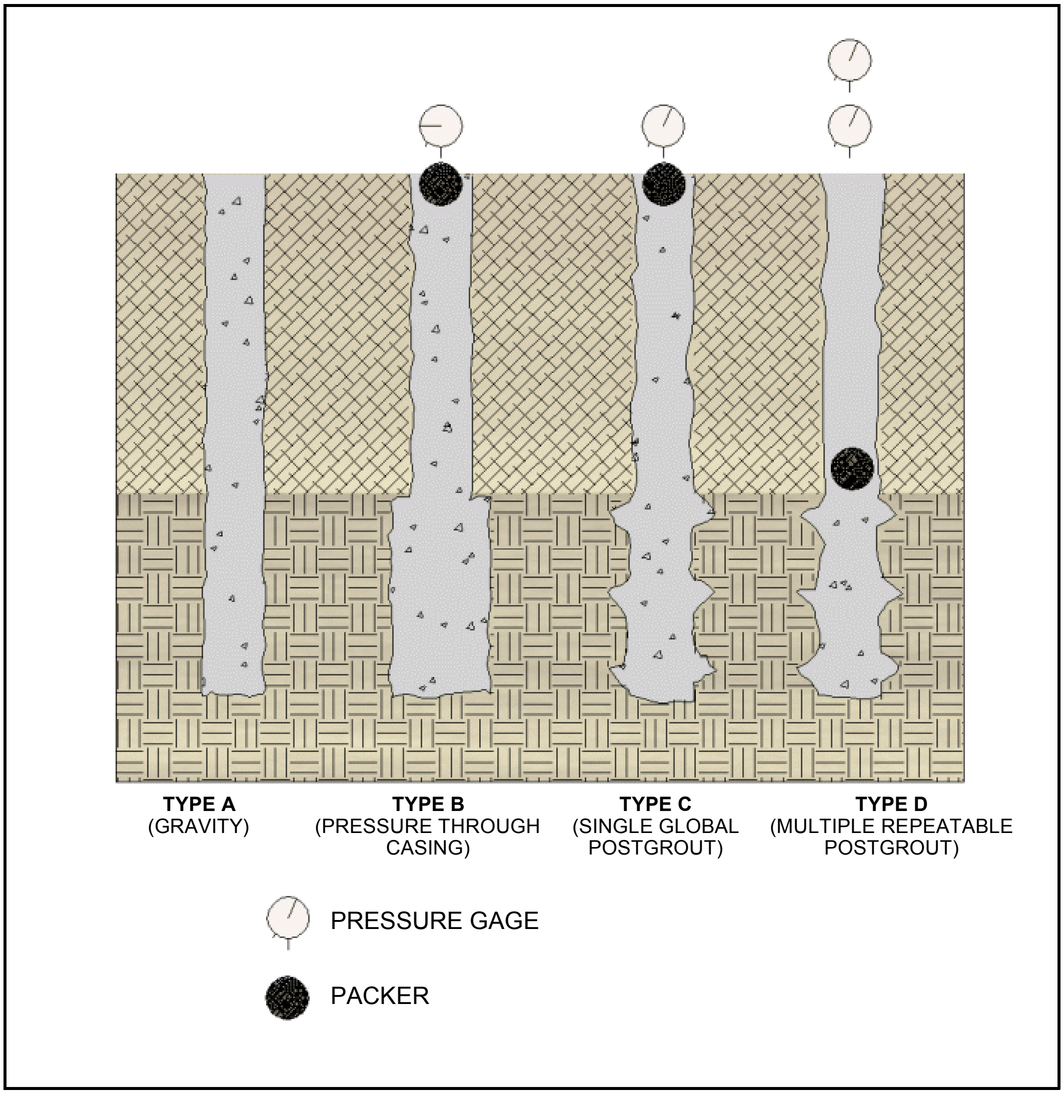

Figure 2 – 5. Micropile Classification Based on Type of Grouting.(Refer to Table 2-1 for details)

Type B: Type B indicates that neat cement grout is placed into the hole under pressure as the temporary steel drill casing is withdrawn. Injection pressures typically range from 0.5 to 1 MPa, and are limited to avoid hydrofracturing the surrounding ground or causing excessive grout takes, and to maintain a seal around the casing during its withdrawal, where possible.

Type C: Type C indicates a two-step process of grouting: 1) neat cement grout is placed under gravity head as with Type A, and 2) prior to hardening of the primary grout (after approximately 15 to 25 minutes), similar grout is injected one time via a sleeved grout pipe without the use of a packer (at the bond zone interface) at a pressure of at least 1 MPa. This pile type appears to be used only in France, and is referred to as IGU (Injection Globale et Unitaire).

Type D: Type D indicates a two-step process of grouting similar to Type C with modifications to Step 2. Neat cement grout is placed under gravity head as with Types A and C and may be pressurized as in Type B. After hardening of the initially placed grout, additional grout is injected via a sleeved grout pipe at a pressure of 2 to 8 MPa. A packer may be used inside the sleeved pipe so that specific horizons can be treated several times, if required. This pile type is used commonly worldwide, and is referred to in France as the IRS (Injection Répétitive et Sélective).

Table 2-1 describes in more detail the micropile classification based on method of grouting (construction). Subclassifications (Numbers 1, 2, and 3) are included in the table to describe the use of drill casing and reinforcement for each method of grouting. These sub-classifications also represent the type of reinforcement required by design (e.g. rebar, casing, none). It is emphasized that Table 2-1 is intended to present a classification system based on the type of micropile construction. It is not intended to be used in contract specifications.

Table 2-1. Details of Micropile Classification Based on Type of Grouting

Micropile Type and Grouting Method	Sub-type	Drill Casing	Reinforcement	Grout
Type A Gravity grout only	A1	Temporary or unlined (open hole or auger)	None, monobar, cage, tube or structural section	Sand/cement mortar or neat cement grout, tremied to base of hole (or casing), no excess pressure applied
	A2	Permanent, full length	Drill casing itself	
	A3	Permanent, upper shaft only	Drill casing in upper shaft, bar(s) or tube in lower shaft (may extend full length)	
Type B Pressure - grouted through the casing or auger during withdrawal	B1	Temporary or unlined (open hole or auger)	Monobar(s) or tube (cages rare due to lower structural capacity)	Neat cement grout is first tremied into drill casing/auger. Excess pressure (up to 1 MPa typically) is applied to additional grout injected during withdrawal of casing/auger
	B2	Permanent, partial length	Drill casing itself	
	B3	Permanent, upper shaft	Drill casing in upper shaft, bar(s) or tube in lower shaft (may extend full length)	
Type C Primary grout placed under gravity head, then one phase of secondary "global" pressure grouting	C1	Temporary or unlined (open hole or auger)	Monobar(s) or tube (cages rare due to lower structural capacity)	Neat cement grout is first tremied into hole (or casing/auger). Between 15 to 25 minutes later, similar grout injected through tube (or reinforcing pipe) from head, once pressure is greater than 1 MPa
	C2	Not conducted	–	
	C3	Not conducted	–	
Type D Primary grout placed under gravity head (Type A) or under pressure (Type B). Then one or more phases of secondary "global" pressure grouting	D1	Temporary or unlined (open hole or auger)	Monobar(s) or tube (cages rare due to lower structural capacity)	Neat cement grout is first tremied (Type A) and/or pressurized (Type B) into hole or casing/auger. Some hours later, similar grout injected through sleeved pipe (or sleeved reinforcement) via packers, as many times as necessary to achieve bond
	D2	Possible only if regrout tube placed full-length outside casing	Drill casing itself	
	D3	Permanent, upper shaft only	Drill casing in upper shaft, bar(s) or tube in lower shaft (may extend full length)	

Source: after Pearlman and Wolosick (1992)

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CHAPTER 3

MICROPILE APPLICATIONS IN TRANSPORTATION PROJECTS

3.A INTRODUCTION

Micropiles are currently used in two general applications: for structural support and less frequently as in-situ reinforcement (Figure 3-1). Structural support includes new foundations, underpinning of existing foundations, seismic retrofitting applications and earth retention. In-situ reinforcement is used for slope stabilization, earth retention, and ground strengthening and protection; settlement reduction; and structural stability. Table 3-1 summarizes the typical design behavior and micropile construction type for each application.

For structural support, micropiles can be used as small-diameter substitutes for conventional pile types. Micropiles used for structural support are usually loaded directly and, therefore, employ a CASE 1 design philosophy. Piles typically used for these applications include Type A (gravity grouted and bonded in soil or rock), Type B (pressure grouted), and Type D (postgrouted). These pile types can provide the high individual capacities typically required by structural support applications in transportation projects.

It is important to note that the in-situ reinforcement applications of slope stabilization and earth retention can employ either CASE 1 or CASE 2 design philosophies. Micropiles used for these applications are typically Type A piles (gravity grouted and fully bonded in soil or rock), because high individual pile capacities are not required due to the reinforced composite material concept of the CASE 2 approach. Recent research (Pearlman et al., 1992) suggests, however, that in certain conditions and for certain pile arrangements, the piles are principally, directly, and locally subjected to bending and shearing forces, specifically near the slide plane. This direct loading, by definition, is CASE 1 design behavior. Micropiles under these conditions are typically heavily reinforced and of Type A or B construction.

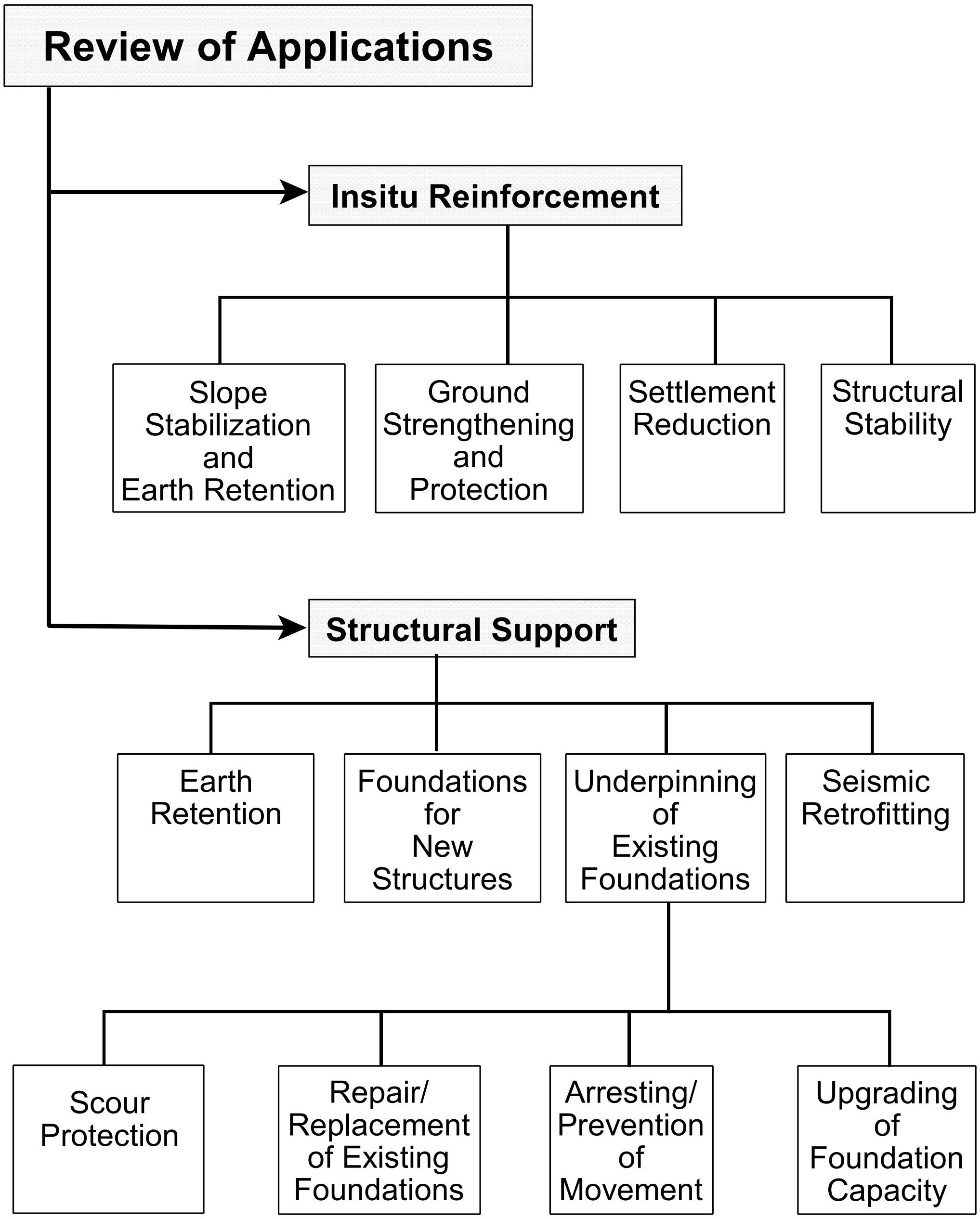


Figure 3 – 1. Classification of Micropile Applications

Table 3-1. Relationship Between Micropile Application, Design Behavior and Construction Type

	STRUCTURAL SUPPORT	IN-SITU EARTH REINFORCEMENT			
Application	Underpinning of Existing Foundations New Foundations Seismic Retrofitting	Slope Stabilization and Earth Retention	Ground Strengthening	Settlement Reduction	Structural Stability
Design Behavior	CASE 1	CASE 1 and CASE 2 with transitions	CASE 2 with minor CASE 1	CASE 2	CASE 2
Construction Type	Type A (bond zones in rock or stiff clays) Type B, C, and D in soil	Type A and Type B in soil	Types A and B in soil	Type A in soil	Type A in soil
Estimate of Relative Application	Probably 95 percent of total world applications	0 to 5 percent			

Source: Federal Highway Administration (1997)

Other insitu reinforcement applications generally employ CASE 2 concepts. Little commercial work has been performed for other CASE 2 major applications beyond the stabilization of high towers in historical monuments. An example is the restoration scheme used to improve the stability of a tall, slender tower in Mosul, Iraq (Lizzi, 1982), as shown in Figure 3-2. This CASE 2 network of reinforced soil is attached to the structure, effectively lowering the center of gravity of the combined structure/soil system and improving stability. The potential for other in-situ reinforcement applications is being studied and pursued in other countries, especially France, Italy, Germany, Austria, and Japan.

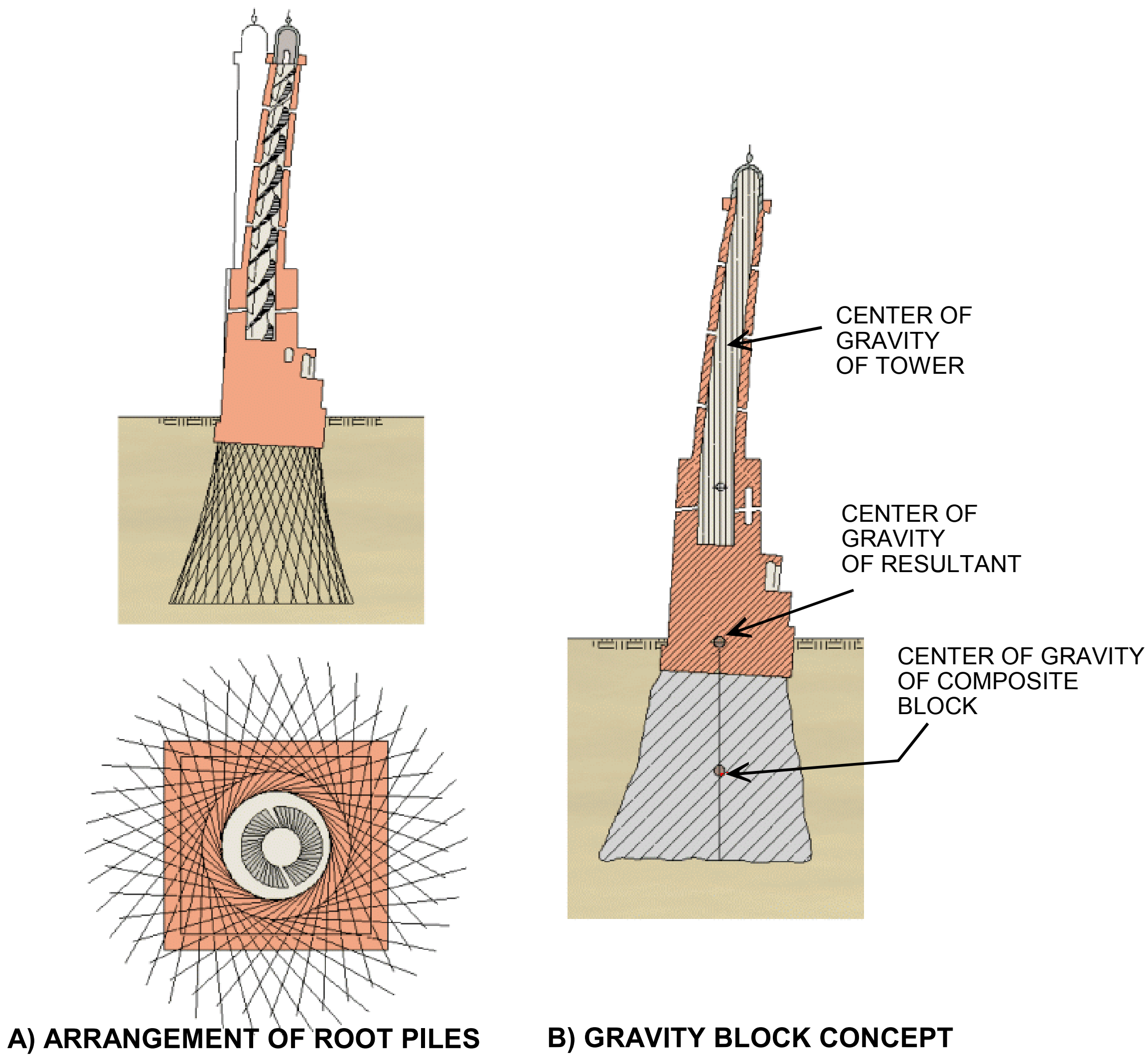


Figure 3 – 2. Restoration Scheme for the Leaning Al Hadba Minaret (Lizzi, 1982)

3.B STRUCTURAL SUPPORT

Micropile applications for structural support include foundations for new structures, underpinning of existing structures, scour protection, and seismic retrofitting of existing structures. Many of these applications have been used for transportation projects.

3.B.1 New Foundations

Micropiles are applicable in new bridge construction in areas that require deep foundation alternatives or in difficult ground (cobbles/boulders obstructions) where installation of conventional piles or drilled shafts is very difficult/expensive.

The new I-78 dual highway, designed to cross the Delaware River between Pennsylvania and New Jersey (Bruce, 1988), is an example. All of the bridge piers were founded either on driven piles or spread footings on rock, with the exception of Pier E-6. At this location, bedrock was encountered below the anticipated depth and was found to be extremely variable. Micropiles and drilled shafts were proposed as alternative foundations to solve this geological problem, and micropiles were selected based on cost, installation time, and test pile performance.

The replacement of a two-span bridge over the Mahoning Creek in Armstrong County, PA (see Photograph 3-1) provides another example. The original stone abutment foundations were constructed in cofferdams and founded on erodible soils overlying competent sandstone. Micropiles were used to support the new abutments as they could be conveniently drilled through the existing stone footings and founded in the underlying sandstone (Pearlman et al., 1992).

New bridges may be constructed in areas of existing overhead restrictions, and with traffic flow that must be maintained. A major improvement project was undertaken to replace the deck of the Brooklyn-Queens Expressway in the Borough of Brooklyn, New York (Bruce and Gemme, 1992.). A new center lane and several new entry/exit ramps were also added. Small-diameter piles were specified and used successfully for the new viaduct and the ramps. Major factors in the selection of micropiles were the relative lack of vibration during installation by comparison to pile-driving methods that could have affected adjacent old and sensitive structures; the variable fluvioglacial deposits; the restricted access; and the need to maintain traffic flow in the area.

Photograph 3 - 1. 175mm Micropiles used under new abutments for bridge over Mahoning Creek, Armstrong County, PA (Pearlman and Wolosick, 1992).

Other micropile applications used for structural support include buildings, earth-retaining structures, and soundwalls. Figure 3-3 shows typical arrangements of micropiles for support of common transportation-related structures.

3.B.2 Underpinning of Existing Foundations

Micropiles were originally developed for underpinning existing structures (Section 1.C). The underpinning of existing structures may be performed for many purposes:

- To arrest and prevent structural movement.
- To upgrade load-bearing capacity of existing structures.
- To repair/replace deteriorating or inadequate foundations.
- To add scour protection for erosion-sensitive foundations.
- To raise settled foundations to their original elevation.
- To transfer loads to a deeper strata.

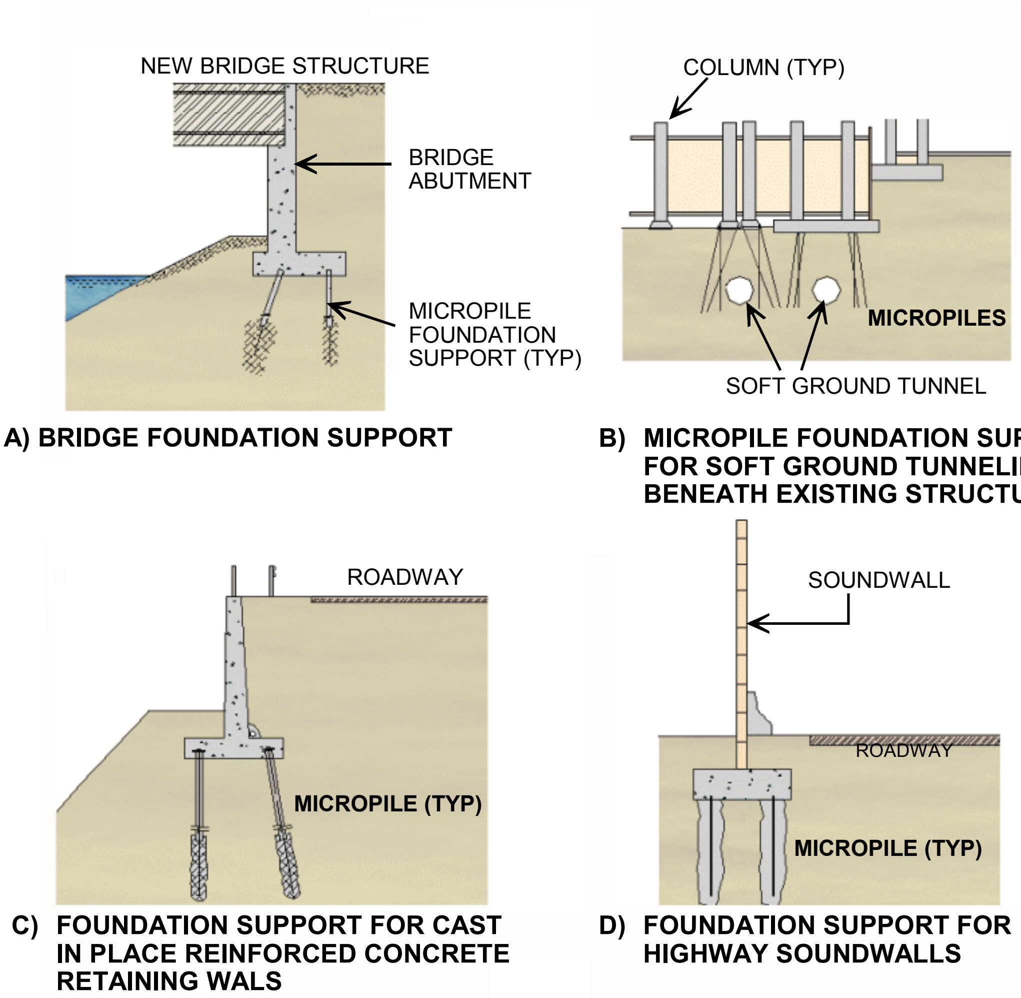


Figure 3 – 3. Micropiles for Foundation Support of Transportation Applications

Micropiles can be installed through, and bonded within, existing structures, providing direct connection with a competent underlying strata without the need for new pile caps, while at the same time reinforcing the structure internally.

Construction can be executed without reducing the existing foundation capacity. Photograph 3-2 is of the West Emerson Street Viaduct in Seattle, Washington, where micropiles were added to five existing bents to provide additional foundation support.

Photograph 3 - 2. Underpinning of West Emerson Street Viaduct, Seattle, Washington

Structural movements can be caused by a variety of factors, including compressible ground beneath the existing foundation, dewatering activities, groundwater elevation fluctuations, deterioration of existing foundations, and adjacent deep excavations and tunneling activities. Micropiles can mitigate this structural movement by being installed to deeper, more competent bearing strata, thus providing improved structural support.

Increased load-bearing capacity of an existing foundation may be required for several reasons. Additional vertical, lateral, or vibratory loads may be applied to the foundation due to expansion of the existing structure, increased magnitude of applied loads, or the addition of vibrating machinery.

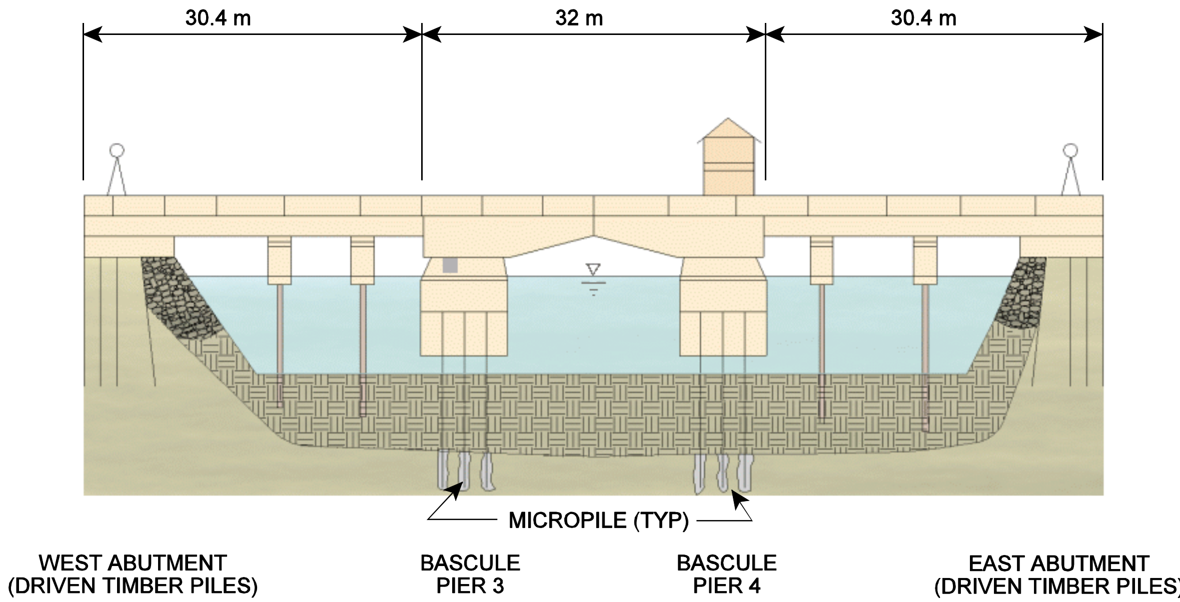


Figure 3 – 4. Underpinning Arrangement for Pocomoke River Bridge, Maryland (Bruce et al., 1990)

The 75-year-old Pocomoke River Bridge in Maryland was rehabilitated when the capacity of the original wooden piles of the pier foundations was compromised by exposure to river scour (Bruce et al., 1990). The underpinning arrangement for this bridge is shown in Figure 3-4. These micropiles were installed throughout the existing foundation and were preloaded to provide support without allowing additional settlement of this sensitive structure.

3.B.3 Seismic Retrofit

Micropiles are being used increasingly for seismic retrofitting of existing highway structures, especially in California. Micropiles may be economically feasible for bridge foundation retrofits having one or more of the following constraints:

- Restrictions on footing enlargements.
- Vibration and noise restrictions.
- Low headroom clearances.

- Difficult access.
- High axial load demands in both tension and compression.
- Difficult drilling/driving conditions.
- Hazardous soil sites.

Micropiles exhibit near equal tension and compression capacities, therefore optimizing the additional foundation support elements used (Bruce and Chu, 1995).

Micropiles were used for the California Department of Transportation's earthquake retrofit of the North Connector over-crossing at I-110 in Los Angeles, (Photograph 3-3) where the use of the previously specified drilled shafts proved unsuccessful (Pearlman et al., 1993). Difficult drilling conditions, including buried concrete obstructions and water-bearing, flowing sand layers, low overhead conditions, and limited right-of-way access prohibited the use of the originally prescribed drilled shaft system.

Micropiles have been used for earthquake retrofit of major bridges in the San Francisco Bay area, New York City and Southern Illinois:

- Benicia-Martinez Bridge, CA.
- 24/580/980 Interchanges, Oakland, CA.
- Williamsburg Bridge, NewYork City, NY.
- Route 57, Illinois.

3.C IN-SITU REINFORCEMENT (SLOPE STABILIZATION & EARTH RETENTION)

The concept of reticulated networks of micropiles (CASE 2) involves the use of an appropriately spaced, three-dimensional arrangement of vertical and inclined piles that encompass and reinforce the ground, and at the same time are supported by the ground. For slope stabilization, Lizzi (1982) suggested that the reticulated network of micropiles creates a stable, reinforced-soil, "gravity-retaining wall", in which the reinforced soil gravity mass supplies the essential resisting force, and the piles, encompassed by the soil, supply additional

Photograph 3 - 3. Seismic Retrofit of I-110, North Connector, Los Angeles, California

resistance to the tensile and shear forces acting on the “wall”. For such applications, the individual piles are engaged as friction piles securing the reinforced soil composite mass in the upper soil mass, and as structural elements subject to shear and bending in the lower competent material. The function of this structure is to provide a stable block of reinforced soil to act as a coherent retaining structure, stabilizing the upper soil mass, while providing resistance to shear across the failure plane. Such an application is, therefore, transitional between CASE 1 and CASE 2 behavior.

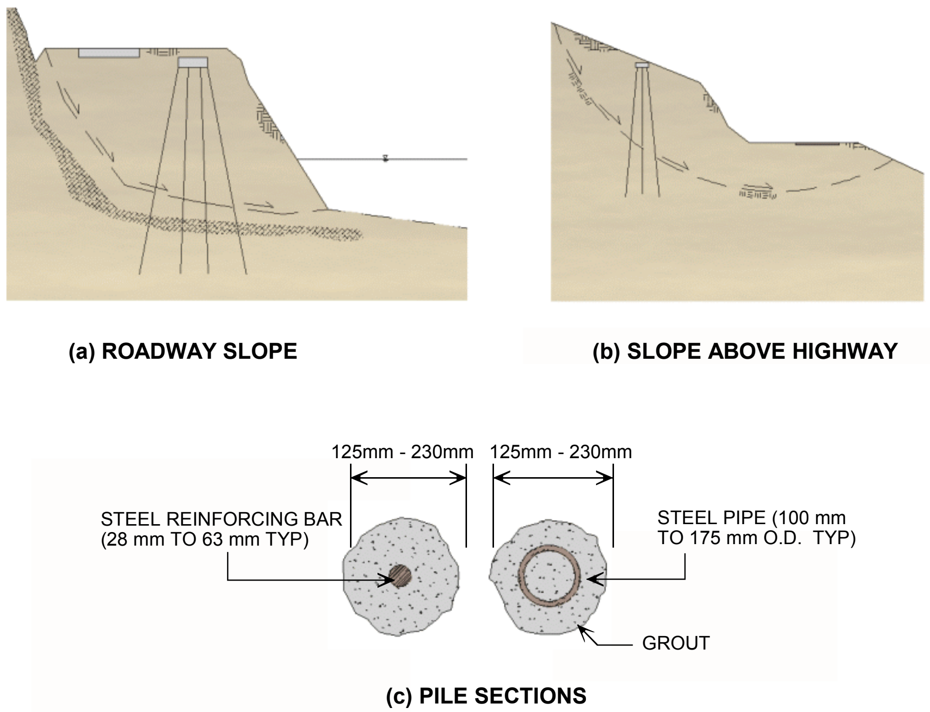


Figure 3 – 5. Typical Configurations for Inclined Micropile Walls

Conversely, the research by Pearlman et al. (1992) and Palmerton (1984) suggests that groups of inclined micropiles serve to connect the moving zone (above the failure surface) to the stable zone (below the failure surface). These piles provide reinforcement to resist the shearing forces that develop along the failure surface and exhibit purely CASE 1 behavior. Typical configurations of inclined nonreticulated micropile walls for slope stabilization and earth retention are shown in Figure 3-5.

For rocky, stiff, or dense materials, the shear resistance of the piles across the failure surface, i.e., individual capacity, is critical (CASE 1). For loose materials, the piles and soil are mutually reinforcing and create a gravity wall, so the individual pile capacities are not as significant (CASE 2).

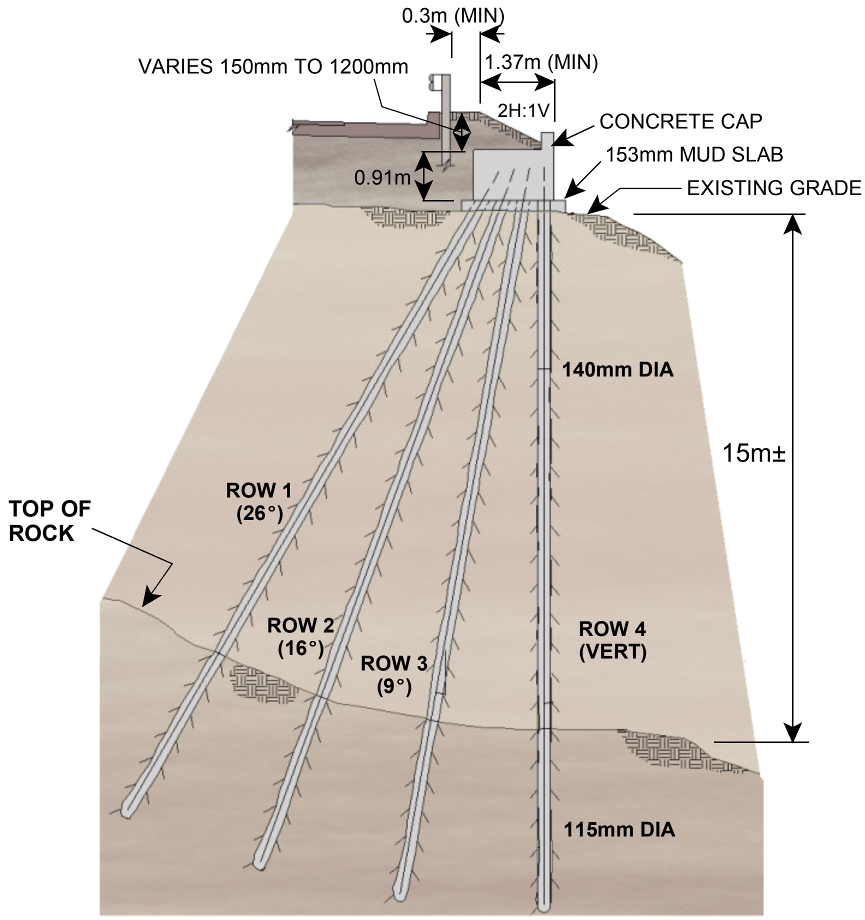


Figure 3 – 6. State Road 4023 Micropile Slope Stabilization, Armstrong County, Pennsylvania

For example, micropiles were used to stabilize a portion of State Road 4023 (S.R. 4023) in Armstrong County, Pennsylvania, as shown in Figure 3-6 (Bruce, 1988a). A 75-m-long section of this road and railroad tracks located up-slope were experiencing damage from slope movements towards an adjacent river. Rock anchors and tangent drilled shafts extending into rock were the proposed remedial techniques. An alternative bid specification allowed the use of inclined CASE 1 nonreticulated micropiles was proposed and accepted, with resultant savings of approximately $1 million compared to the lowest bid for the anchored caisson wall design. The wall included four rows of Type 1A micropiles extending across the failure plane and into competent rock.

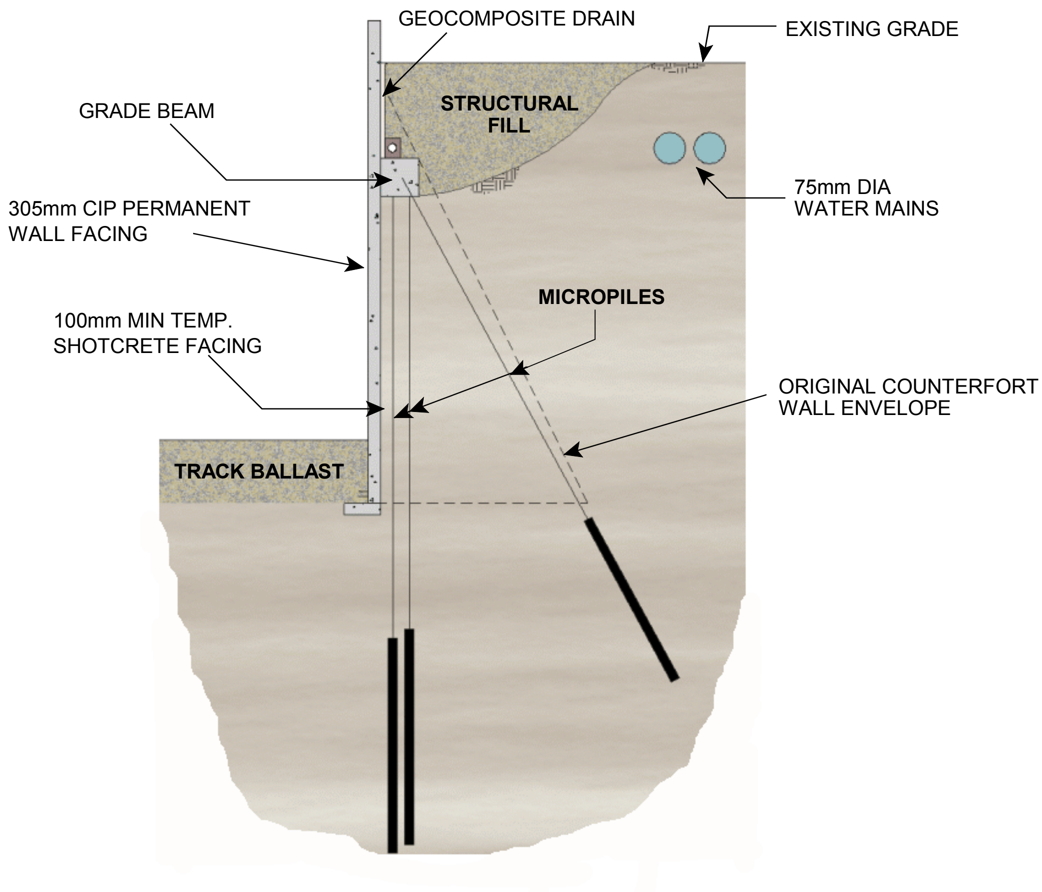


Figure 3 – 7. Wall 600 Permanent Earth Retention, Portland, Oregon

Micropiles were recently used to provide permanent earth retention along a new section of the Portland Westside Lightrail Project in Portland, Oregon, as shown in Figure 3-7 (Ueblacker, 1996). Wall 600 extends from the east portal of the Westside Lightrail cut and cover, tunnel approximately 183 meters to just beneath the Vista Avenue Bridge. Cut heights along the retaining wall range from 4 to 9.5 meters. The wall includes CASE 1 nonreticulated micropiles installed at varying vertical and subvertical angles that were temporally exposed by excavation of the new rail alignment. An architecturally treated cast-in-place reinforced concrete facing is structurally attached to the vertical micropiles, forming the permanent wall face. The micropile wall was accepted as a contractor-proposed value engineering alternate. The original design included a counterfort concrete retaining wall supported on a driven-pile foundation.

Construction of the owner-designed wall required temporary excavation support to maintain service to Jefferson Street and existing utilities. A micropile retaining wall system was the only acceptable option that could be kept within the envelope of the original counterfort structure.

An earlier example of slope stabilization using micropiles as a demonstration project involved networks of reticulated micropiles (CASE 2) used to stabilize Forest Highway 7 in Mendocino National Forest, California, as shown in Figure 3-8 and Photograph 3-4 (Palmerton, 1984). This two-lane road was constructed across a landslide where slide movement was experienced due to excessive rainfall. A 94-m-long section of the road was stabilized using CASE 2 micropiles to reinforce the soil mass and provide additional shear capacity. It is significant that the density of micropiles per lineal meter of wall was significantly higher than that of the CASE 1 design approach used at S.R. 4023. S.R.4023 used *2.9 to 4.1* piles per lineal meter and FH-7 used *7.4* piles per lineal meter. Both structures have performed acceptably (Pearlman, et al., 1992).

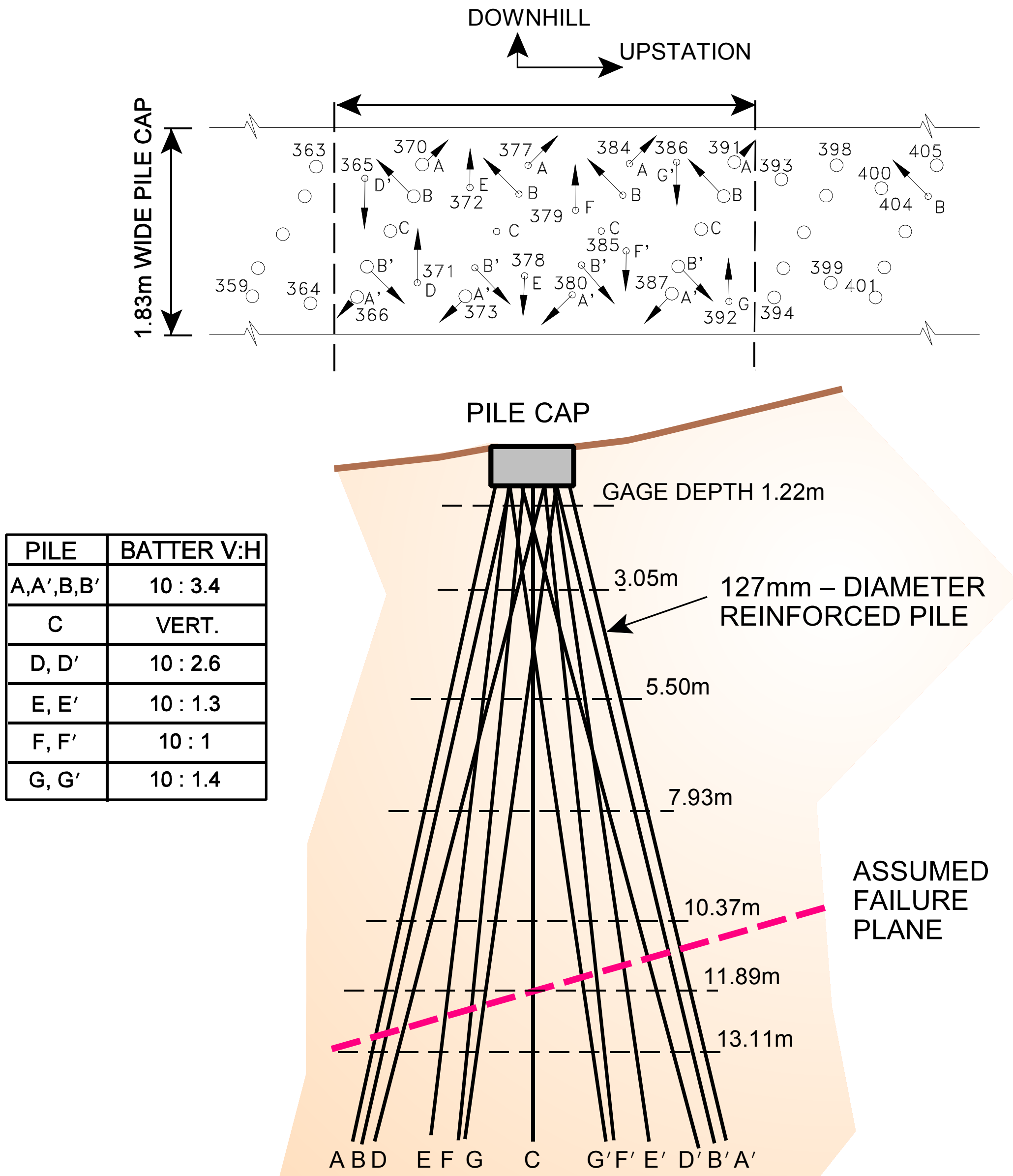
3.D FACTORS INFLUENCING THE CHOICE OF MICROPILES

Several factors influence the choice of micropiles for structure foundations and slope stabilization including:

- Physical Considerations.
  - Restricted access, remote areas.
  - Close pile proximity to existing structures.
- Subsurface Conditions.
  - Difficult geologic conditions.
  - Susceptibility of ground to liquefaction during pile driving.
- Environmental Conditions.
  - Vibration/noise sensitive areas.
  - Hazardous or contaminated soils.
- Existing Structure Adaptation.

- Micropile Limitations.
- Economics.

Further discussion of these factors and how they influence the designer's choice is addressed below.



PILE	BATTER V:H
A,A′,B,B′	10 : 3.4
C	VERT.
D, D′	10 : 2.6
E, E′	10 : 1.3
F, F′	10 : 1
G, G′	10 : 1.4

Figure 3 – 8. CASE 2 Slope Stabilization, FH-7, Mendocino National Forest, California

Photograph 3 - 4. FH-7, Mendocino National Forest, California (Slope Stabilization)

3.D.1 Physical Considerations

The drilling and grouting equipment used for micropile installation is relatively small and can be mobilized in restrictive areas that would prohibit the entry of conventional pile-installation equipment. Photograph 3-5 shows micropiles being installed in low headroom conditions, illustrating the maneuverability of the equipment.

Micropiles can be installed within a few millimeters of existing walls or foundations, provided that there is space above for the drill-head and safe work zone or the piles are battered to provide this space. Their installation is not as affected by overhead power lines or other obstructions as are conventional pile-installation systems. The equipment can be mobilized upsteep slopes and in remote locations. Also, drilling and grouting procedures associated with micropile installations do not cause damage to adjacent existing structures or affect adjacent ground conditions when proper drilling and grouting procedures are utilized.

Photograph 3 - 5.   Low Headroom Micropile Installation

3.D.2 Subsurface Conditions

Micropiles can be installed in areas of particularly difficult, variable, or unpredictable geologic conditions such as ground with cobbles and boulders, fills with buried utilities and miscellaneous debris, and irregular lenses of competent and weak materials. Soft clays, running sands, and high ground water not conducive to conventional drilled shaft systems cause minimal impacts to micropile installations. Micropiles have been applied throughout the world in karstic limestone formations.

The construction of the new I-78 bridge over the Delaware River described in Section 3.B.1 is a good example of the use of micropiles for conditions where difficult geology prevents the use of more conventional systems.

3.D.3 Environmental Conditions

Micropiles can be installed in hazardous and contaminated soils. Their small diameter results in less spoil than caused by conventional replacement piles, and the flush effluent can be controlled easily at the ground surface through containerization or the use of lined surface pits. These factors greatly reduce the potential for surface contamination and handling costs.

Grout mixes can be designed to withstand chemically aggressive ground water and soils. Special admixtures can be included in the grout mix design to reduce and avoid deterioration from acidic and corrosive environments. For example, a micropile “screen” was constructed from the installation of overlapping (secant) piles adjacent to an existing concrete diaphragm wall of an underground parking garage in Barcelona, Spain (Bachy, 1992). The existing wall was physically deteriorating due to extremely aggressive ground water (chlorides, sulfates, and pH values as low as 1.7) originating from an adjacent metallurgical plant (Figure 3-9). No trace of acid was detected in samples of the diaphragm wall collected after construction of the micropile screen.

Micropiles can be installed in environmentally sensitive areas, including areas with fragile natural settings. The installation equipment is not as large or as heavy as conventional pile driving or shaft drilling equipment and can be used in swampy areas or other areas of wet or soft surface soils with minimal impacts to the environment. Portable drilling equipment is frequently used in areas of restricted access.

Micropile installations cause less noise (no more than typical ambient noise levels) and vibration than conventional piling techniques, especially driven piles. The vibration from pile driving is imparted to the soil and can be transferred through the soil to adjacent structures. The use of micropiles in old urban environments and industrial/manufacturing areas, can prevent this potential damage to adjacent sensitive structures and equipment.

Micropiles can be installed in areas where there is a contaminated aquifer overlying a bearing strata. Unlike driven piles that may produce a vertical conduit for contaminates transfer, micropiles can be installed in a manner preventing contamination of the lower aquifers.

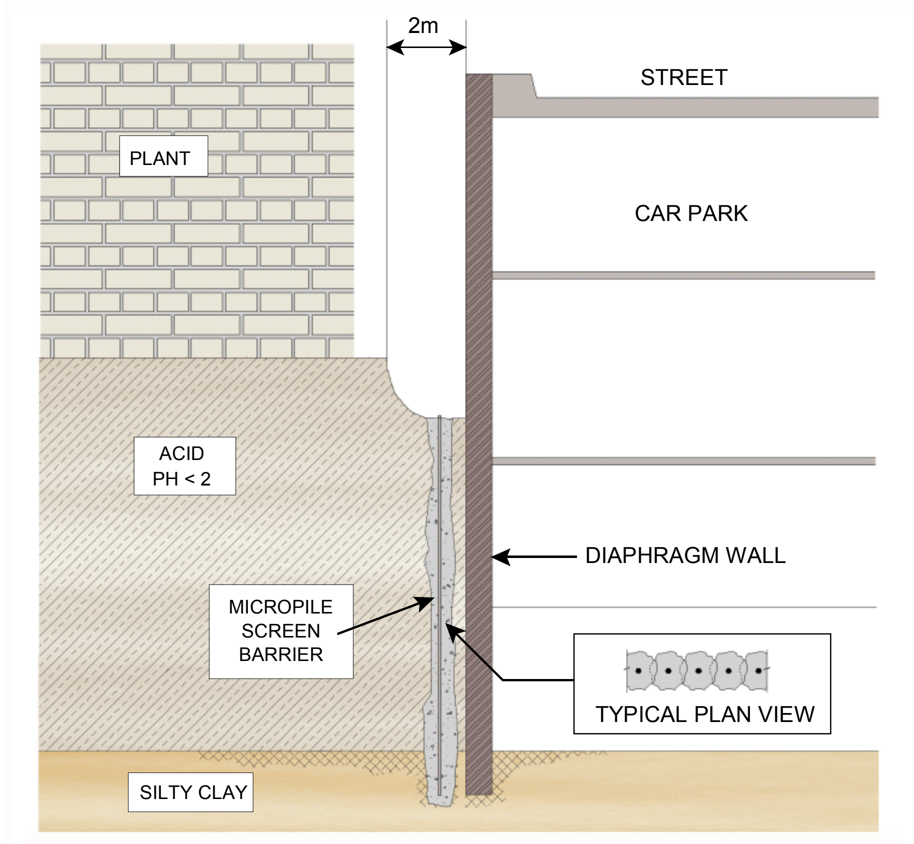


Figure 3 – 9. Protection of an Existing Diaphragm Wall with a Secant Micropile Screen using Anti-acid mortar (Bachy, 1992.)

3.D.4 Existing Structure Adaptation

Micropiles can be added into an existing pile cap on existing piling, thereby eliminating the need for increased footing dimension. This provides the additional compression, tension and moment resistance associated with increased structural loads. Often times adjacent utilities and/or structures restrict the possibility of enlarging the existing pile caps, thus eliminating more traditional piling systems.

3.D.5 Micropile Limitations

Under certain circumstances, vertical micropiles may be limited in lateral capacity and cost effectiveness. Traditionally, axial capacity was also deemed a limitation due to the micropile's relatively small diameter. However, micropiles have now been tested to well beyond 4,500 kN axial capacity in dense sand, as at Vandenberg Air Force Base in California (Federal Highway Administration, 1997), and so one may hope to expect advances in lateral capacity with additional research and pile testing. The ability of micropiles to be installed on an incline provides designers an option for achieving the required lateral capacity in such applications. Because of their high slenderness ratio (length/diameter), micropiles may not be acceptable for conventional seismic retrofitting applications in areas where liquefaction may occur, given the current standards and assumptions on support required for long slender elements. However, the ground improvement which can be induced by micropiles may ultimately yield an improved earthquake mitigation foundation system.

The lineal cost of micropiles usually exceeds that of conventional piling systems, especially driven piles. However, under certain combinations of circumstances, micropiles will be the cost- effective option, and occasionally will be the only feasible constructible option.

Use of micropiles for slope stabilization has been applied to limited heights and is based on very limited experience to date. Due to the limited number of project applications, it is suggested that stabilization applications be instrumented and monitored for performance. On Federal Highway projects, it is suggested that initial projects be designated "Experimental Features". This designation allows construction funds to be used to pay for performance monitoring and evaluation.

3.D.6 Economics of Micropiles

Costs associated with the use of micropiles are discussed in Chapter 10. Cost effectiveness of micropiles is dependent upon many factors. It is important to assess the cost of using micropiles in light of the physical, environmental, and subsurface factors described above. For example, for an open site with soft, clean, uniform soils and unrestricted access, micropiles may not be a competitive solution. However, for the delicate underpinning of an existing bridge pier in a heavily trafficked old industrial or residential area, micropiles can provide the most cost-effective solution.

Care should be taken to clearly define the true final cost of a solution based on micropiles. Cost analysis should be based on all related costs for the entire project and not just the item cost of the piling system. These costs may include:

- Right-of-way acquisition.
- Right-of-way agreements.
- Utility realignment.
- Excavation, shoring and backfill requirements.
- Footing construction.
- Hazardous material handling.
- Dewatering.
- Erosion control.
- Access restrictions.
- Ground improvement.
- Owner and Neighbor disruption.

Refer to Chapter 10 for further clarification on the cost of micropile systems.

CHAPTER 4

CONSTRUCTION TECHNIQUES AND MATERIALS

4.A INTRODUCTION

The intent of this chapter is to familiarize designers and construction personnel with the different techniques and materials utilized in the construction of micropiles.

The construction of a micropile involves a succession of processes, the most significant of which are drilling, placing the reinforcement, and grouting. There are a large number of drilling systems available for both overburden and rock, and many are used for micropile construction. This narrow range of drilling systems tends to be utilized worldwide as a result of the comprehensive international marketing and sales efforts of the drill rig equipment manufacturers and the ongoing exchange of data and experiences in trade and professional organizations and their related journals.

The placement of reinforcement is a fairly standard process, although different countries use different grades, sizes, and configurations. In the United States is it common practice to leave the drill casing in place from the surface down to the top of the bond length of the pile.

It is in the process of grouting that the most extreme range of practices and preferences is evident. This consideration fosters the use of the grouting method as the basis of the micopile type classification, presented in Chapter 2.

Included in this chapter is an examination of the following:

1. The types of drill rigs used for micropile drilling;
2. The various techniques used for overburden and open hole drilling;
3. Types of grout with methods of mixing and placing; and
4. Types of pile reinforcement.

The typical construction sequence for simple Type A and B micropiles (Figure 4-1) includes drilling the pile shaft to the required tip elevation, placing the steel reinforcement, placing the initial grout by tremie, and placing additional grout under pressure as applicable. In general, the drilling and grouting equipment and techniques used for the micropile construction are similar to those used for the installation of soil nails, ground anchors, and grout holes.

4.B DRILLING

Most of the drilling methods selected by the specialty contractor are likely to be acceptable on a particular project, provided they can form a stable hole of the required dimensions and within the stated tolerances, and without detriment to their surroundings. It is important not to exclude a particular drilling method because it does not suit a predetermined concept of how the project should be executed. It is equally important that the drilling contractor be knowledgeable of the project ground conditions, and the effects of the drilling method chosen. Drilling within a congested urban site in close proximity of older buildings or deteriorating foundations has very different constraints than drilling for new foundations on an open field site.

The act of drilling and forming the pile hole may disturb the surrounding ground for a certain time and over a certain distance. The drilling method selected by the contractor should avoid causing an unacceptable level of disturbance to the site and its facilities, while providing for installation of a pile that supports the required capacities in the most cost-effective manner. Vigorous water flushing can increase drilling rates and increase the removal of the fine components of mixed soils, enlarging the effective diameter in the bond zone and aiding in grout penetration and pile capacity. Conversely, the use of higher flush flow rates and pressures should be approached with caution, with consideration to the risks of creating voids and surface settlement, and the risks of hydrofracturing the ground, leading to heaving.

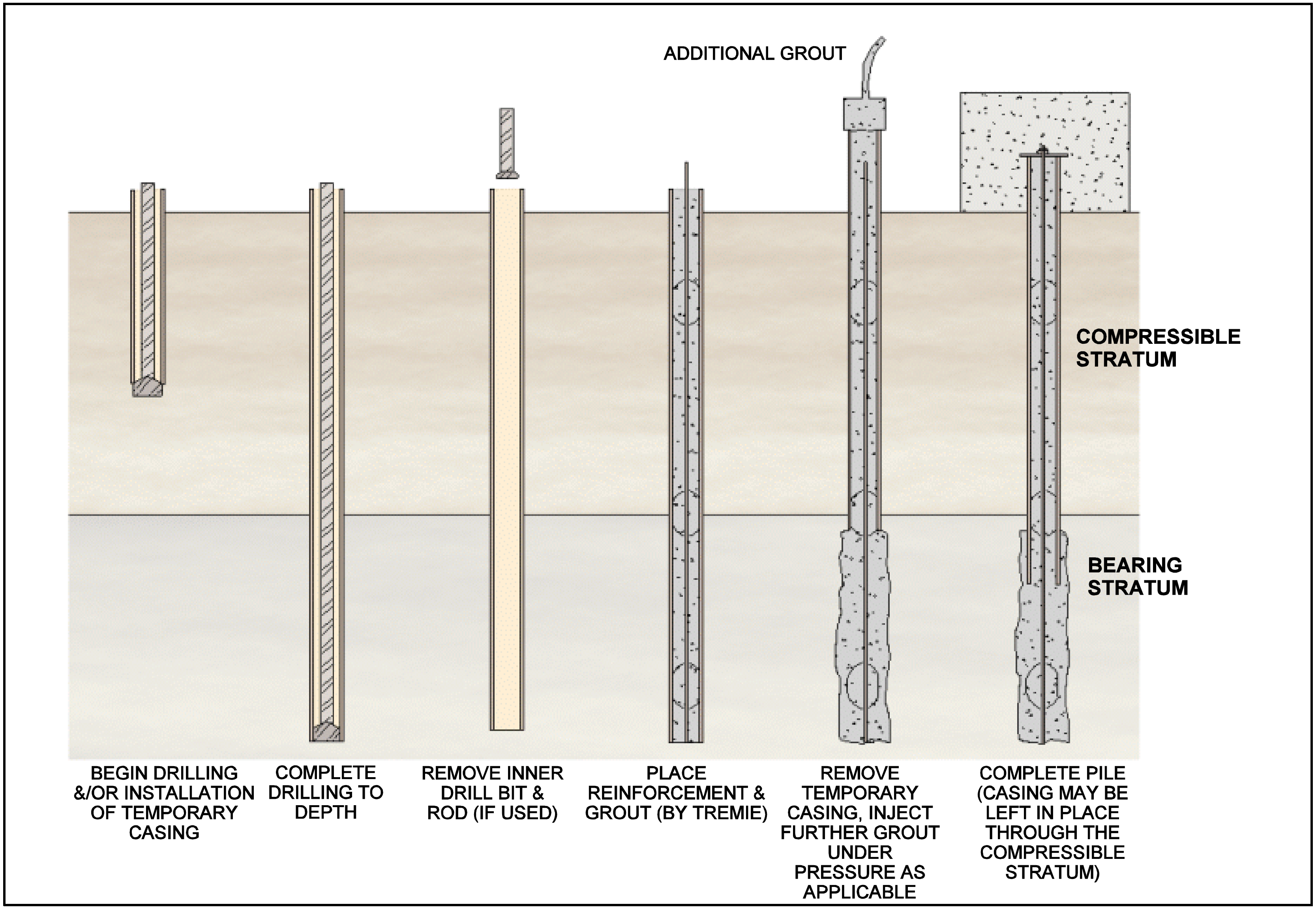


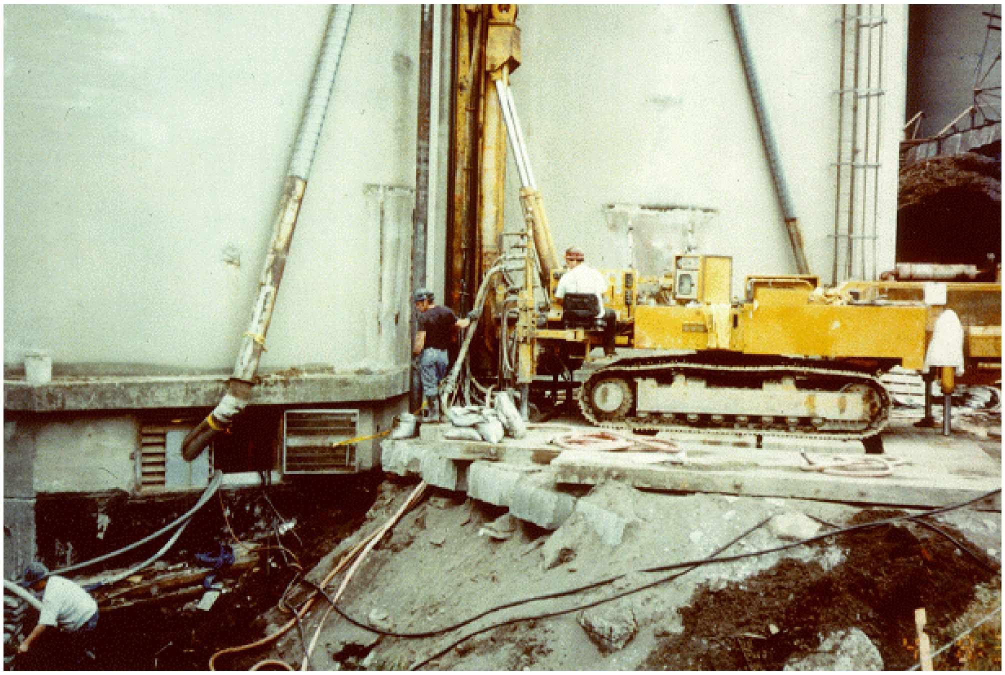
Figure 4 - 1. Typical Micropile Construction Sequence Using Casing

It is typically in the best interest of pile quality that the drilling, installation of the reinforcement, and grouting of a particular pile be completed in a series of continuous processes and executed as expeditiously as possible. Longer duration between completion of drilling and placing of the reinforcement and grout can be detrimental to the integrity of the surrounding soil. Some materials, such as overconsolidated clays and clay shales, can deteriorate, relax, or soften on exposure, resulting in a loss of interfacial bond capacity. In these cases, installation of a pile bond zone should be completed within one day to avoid a pile hole remaining open overnight.

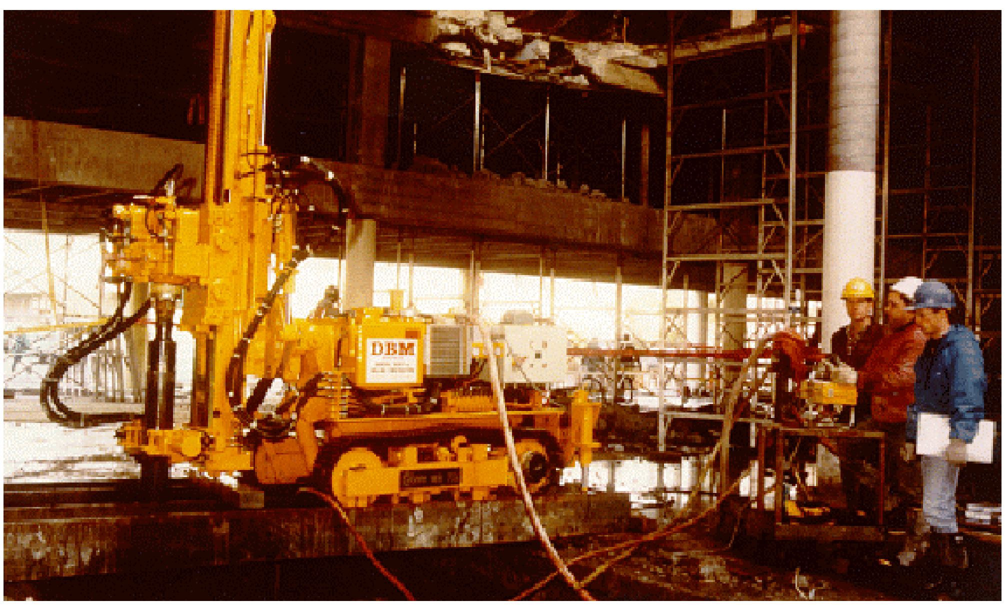
Other site-specific conditions may affect selection of the drilling method and flush type. The use of a water flush can require the supply, handling, and disposal of large quantities of water. In areas where the water supply may be scarce, the setup of a series of ponds or tanks for settlement and recirculation of the water may be necessary. Requirements for cleanliness or lack of space for water handling and disposal may dictate the use of air flush or augers for hole drilling. The presence of hazardous materials in the ground and the need for careful control and disposal of the soil cuttings may also necessitate the use of augers for pile installation.

4.B.1 Drill Rigs

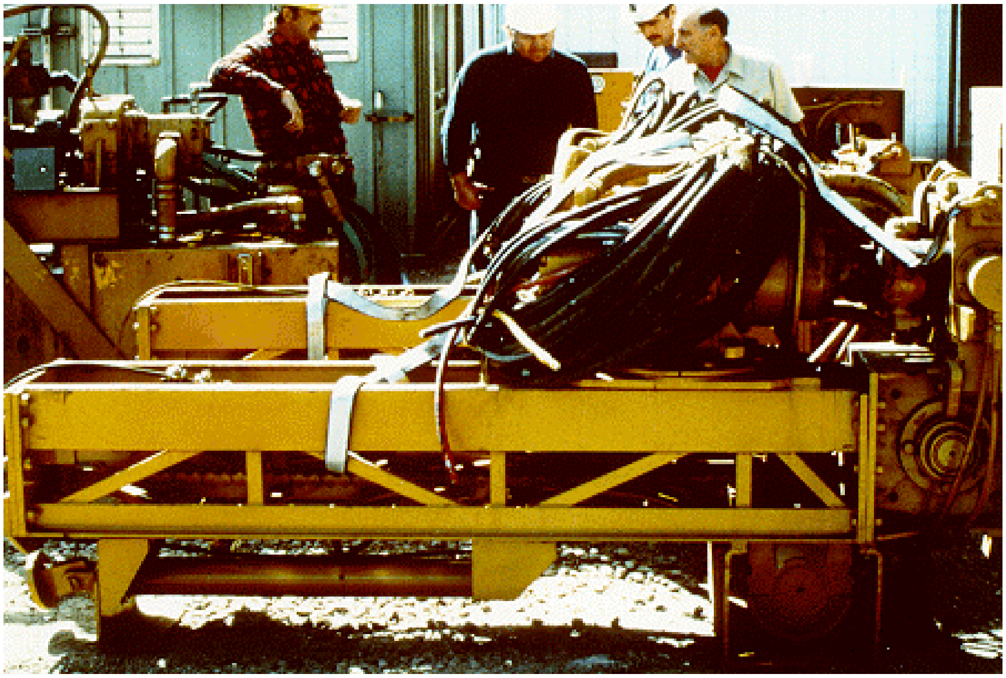
The drill rigs typically used are hydraulic rotary (electric or diesel) power units. They can be track mounted; this allows maneuverability on difficult and sloped terrain. The size of the track-mounted drills can vary greatly, as seen in Photographs 4-1 and 4-2, with the larger drill allowing use of long sections of drill rods and casing in high, overhead conditions, and the smaller drill allowing work in lower overhead and harder-to-reach locations. The drill mast can be mounted on a frame, allowing work in very limited-access and low-overhead areas, such as building basements. A frame-mounted drill, such as the one shown in Photograph 4-3, can be connected with long hoses to a separate hydraulic rotary power unit. This allows placement of the power unit outside the area of work, reducing space requirements, noise in the work area, and problems with exhaust removal. The drill frame can be moved and supported with a fork lift, or moved by hand with winches and supported by bolting to a concrete floor, and/or bridge footing or bracing from a ceiling or bridge soffit.



Photograph 4 - 1.   Large Track-mounted Rotary Hydraulic Drill Rig



Photograph 4 - 2.   Small Track-mounted Rotary Hydraulic Drill Rig



Photograph 4 - 3. Small Frame-mounted Rotary Hydraulic Drill Rig

The rotary head that turns the drill string (casing, augers, or rods) can be extremely powerful on even the smallest of rigs, allowing successful installation in the most difficult ground conditions. Shortening of the drill mast and the use of short jointed sections of drill string and pile reinforcement allows pile installation with less than 3 m of overhead. The pile centerline can be located within 0.3 m of the face of an adjacent wall.

4.B.2 Drilling Techniques

The drilling method is chosen with the objective of causing minimal disturbance or upheaval to the ground and structure, while being the most efficient, economic, and reliable means of penetration. Micropiles must often be drilled through an overlying weak material to reach a more competent bearing stratum. Therefore, it typically requires the use of overburden drilling techniques to penetrate and support weak and unconsolidated soils and fills. In addition, unless the bearing stratum is a self-supporting material, such as rock or a cohesive soil, the drill hole may need temporary support for its full length, e.g. through the use of temporary casing or suitable drilling fluid. If self-supporting material is present for the full depth of the pile, the drillhole can possibly be formed by open hole techniques, i.e., without the need for temporary hole support by drill casing or hollow stem auger.

A different drilling method may be used to first penetrate through an existing structure. Concrete coring techniques may be used to provide an oversized hole in existing slabs and footings, to allow the subsequent drill casing to pass through. In some cases, conventional rock drilling methods involving rotary percussive techniques can be used to penetrate existing footings or structures with only light reinforcement. Rotary percussive or rotary duplex techniques may be used to first penetrate an initial obstruction layer, such as concrete rubble, with more conventional single-tube advancement drilling used for completion of the pile shaft in the soil layers below.

Water is the most common medium for cleansing and flushing the hole during drilling, followed by air, drill slurries, and foam. Caution should be exercised while using air flush to avoid injection of the air into the surrounding ground, causing fracturing and heaving. The use of bentonite slurries to stabilize and flush holes is generally believed to impair grout /ground bond capacity by creating a skin of clay at the interface; however, this is not an uncommon choice in Italian and French practice with Type D piles. Polymer drilling muds have been used successfully in micropile construction in all types of ground. This slurry type reduces concern for impairment of the bond capacity, and allows for easier cleanup and disposal versus bentonite slurry.

4.B.3 Overburden Drilling Techniques

There is a large number of proprietary overburden drilling systems sold by drilling equipment suppliers worldwide. In addition, specialty contractors often develop their own variations in response to local conditions and demands. The result is a potentially bewildering array of systems and methods, which does, however, contain many that are of limited application, and many that are either obsolete or virtually experimental. Closer examination of this array further confirms that there are essentially six generic methods in use internationally in the field of specialty geotechnical construction (i.e., diameters less than 300 mm, depths less than 60 m). The following is a brief discussion of these six methods. These six methods are also summarized in Table 4-1, and simply represented in Figure 4-2.

Single-tube advancement - external flush (wash boring): By this method, the toe of the drill casing is fitted with an open crown or bit, and the casing is advanced into the ground by rotation of the drill head. Water flush is pumped continuously through the casing, which washes debris out and away from the crown. The water-borne debris typically escapes to the surface around the outside of the casing, but may be lost into especially loose and permeable upper horizons. Care must be exercised below sensitive structures in order that uncontrolled washing does not damage the structure by causing cavitation.

Air flush is not normally used with this system due to the danger of accidentally overpressurizing the ground in an uncontrolled manner, which can cause ground disturbance. Conversely, experience has shown that polymer drill flush additives can be very advantageous in certain ground conditions, in place of water alone (Bruce, 1992.). These do not appear to detrimentally affect grout-to-soil bond development as may be the case with bentonite slurries.

Table 4-1. Overburden Drilling Methods

	Drilling Method	Principle	Common Diameters and Depths	Notes
1	Single-tube advancement: a) Drive drilling b) External flush	Casing with "lost point" percussed without flush. Casing, with shoe, rotated with strong water flush.	50-100 mm to 30 m 100-250 mm to 60 m	Obstructions or very dense soil problematical. Very common for anchor installation. Needs high torque head and powerful flush pump.
2	Rotary duplex	Simultaneous rotation and advancement of casing plus internal rod, carrying flush.	100-220 mm to 70 m	Used only in very sensitive soil/site conditions. Needs positive flush return. Needs high torques. (Internal flushing only)
3	Rotary percussive concentric duplex	As 2, above, except casing and rods percussed as well as rotated.	89-175 mm to 40 m	Useful in obstructed/rocky conditions. Needs powerful top rotary percussive hammer.
4	Rotary percussive eccentric duplex	As 2, except eccentric bit on rod cuts oversized hole to ease casing advance.	89-200 mm to 60 m	Expensive and difficult system for difficult overburden.
5	"Double head" duplex	As 2 or 3, except casing and rods may rotate in opposite directions.	100-150 mm to 60 m	Powerful, new system for fast, straight drilling in very difficult ground. Needs significant hydraulic power.
6	Hollow-stem auger	Auger rotated to depth to permit subsequent introduction of grout and/or reinforcement through stem.	100-400 mm to 30 m	Obstructions problematical; care must be exercised in cohesionless soils. Prevents application of higher grout pressures.

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

Source: after Bruce et al., 1989.

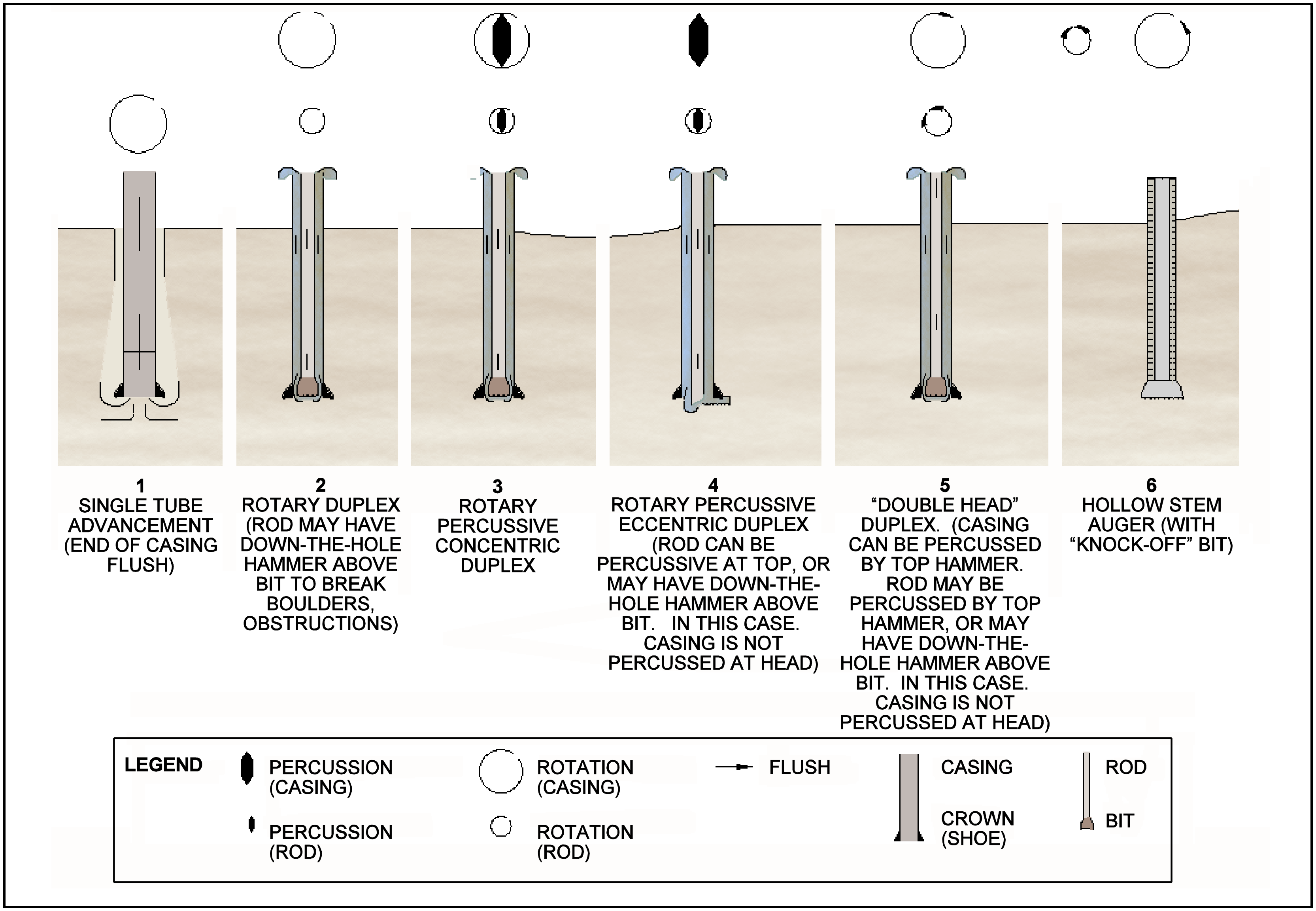


Figure 4 - 2. Overburden Drilling Methods (Bruce, 1988)

Rotary Duplex: With the rotary duplex technique, drill rod with a suitable drill bit is placed inside the drill casing. It is attached to the same rotary head as the casing, allowing simultaneous rotation and advancement of the combined drill and casing string. The flushing fluid, usually water or polymer flush, is pumped through the head down through the central drill rod to exit from the flushing ports of the drill bit. The flush-borne debris from the drilling then rises to the surface along the annulus between the drill rod and the casing. At the surface, the flush exits through ports in the drill head. Although any danger with duplex drilling is less than when using the single-tube-method, air flush must be used with caution because blockages within the annulus can allow high air pressures and volumes to develop at the drill bit and cause ground disturbance.

Rotary Percussive Duplex (Concentric): Rotary percussive duplex systems are a development of rotary duplex methods, whereby the drill rods and casings are simultaneously percussed, rotated, and advanced. The percussion is provided by a top-drive rotary percussive drill head. This method requires a drill head of substantial rotary and percussive energy.

Rotary Percussive Duplex (Eccentric or Lost Crown): Originally sold as the Overburden Drilling Eccentric (ODEX) System, this method involves the use of rotary percussive drilling combined with an eccentric underreaming bit. The eccentric bit undercuts the drill casing, which then can be pushed into the oversized drill hole with much less rotational energy or thrust than is required with the concentric method just described. In addition, the drill casing does not require an expensive cutting shoe and suffers less wear and abrasion.

The larger diameter options, of more than 127 mm in diameter, often involve the use of a down-the-hole hammer acting on a drive shoe at the toe of the casing, so that the casing is effectively pulled into the borehole as opposed to being pushed by a top hammer.

Most recently, systems similar to ODEX, which is now sold as TUBEX, have appeared from European and Japanese sources. Some are merely mechanically simpler versions of TUBEX. Each variant, however, is a percussive duplex method in which a fully retractable bit creates an oversized hole to ease subsequent casing advancement.

Double Head Duplex: With the double head duplex method, a development of conventional rotary duplex techniques, the rods and casings are rotated by separate drill heads mounted one above the other on the same carriage. These heads provide high torque (and so enhanced soil-and obstruction-cutting potential), but at the penalty of low rotational speed. However, the heads are geared such that the lower one (rotating the outer casing), and the upper one (rotating the inner drill string) turn in opposite directions. The resulting aggressive cutting and shearing action at the bit permits high penetration rates, while the counter-rotation also discourages blockage of the casing/rod annulus by debris carried in the exiting drill flush. In addition, the inner rods may operate by either purely rotary techniques or rotary percussion using top-drive or down-the-hole hammers. The counter-rotation feature promotes exceptional hole straightness, and encourages penetrability, even in the most difficult ground conditions.

Hollow-Stem Auger: Hollow-stem augers are continuous flight auger systems with a central hollow core, similar to those commonly used in auger-cast piling or for ground investigation. These are installed by purely rotary heads. When drilling down, the hollow core is closed off by a cap on the drill bit. When the hole has been drilled to depth, the cap is knocked off or blown off by grout pressure, permitting the pile to be formed as the auger is withdrawn. Such augers are used mainly for drilling cohesive materials or very soft rocks.

Various forms of cutting shoes or drill bits can be attached to the lead auger, but heavy obstructions, such as old foundations and cobble and boulder soil conditions, are difficult to penetrate economically with this system. In addition, great care must be exercised when using augers: uncontrolled penetration rates or excessive “hole cleaning” may lead to excessive spoil removal, thereby risking soil loosening or cavitation in certain circumstances.

4.B.4 Open-Hole Drilling Techniques

When the micropile can be formed in stable and free-standing conditions, the advancement of casing may be suspended and the hole continued to final depth by open-hole drilling techniques. There is a balance in cost between the time lost in changing to a less-expensive open-hole system and continuing with a more expensive overburden drilling system for the full hole depth. *Contractors need to be cautious with open-hole drilling operations. The micropile installation contractor is ultimately responsible for selection and proper performance of the drilling and installation method(s).* Open-hole drilling techniques may be classified as follows:

Rotary Percussive Drilling: Particularly for rocks of high compressive strength, rotary percussive techniques using either top-drive or down-the-hole hammers are utilized. For the small hole diameters used for micropiles down-the-hole techniques are the most economical and common. Air, air/water mist, or foam is used as the flush.

Top-drive systems can also use air, water, or other flushing systems, but have limited diameter and depth capacities, are relatively noisy, and may cause damage to the structure or foundation through excessive vibration.

Solid Core Continuous Flight Auger: In stiff to hard clays without boulders and in some weak rocks, drilling may be conducted with a continuous flight auger. Such drilling techniques are rapid, quiet, and do not require the introduction of a flushing medium to remove the spoil. There may be the risk of lateral decompression or wall remolding/interface smear, either of which may adversely affect grout/soil bond. Such augers may be used in conditions where the careful collection and disposal of drill spoils are particularly important environmentally.

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

4.C GROUTING

As described in Chapter 2, the grouting operations have a major impact on micropile capacity and form the most fundamental construction basis for micropile classification. Details of each type of grouting operation vary somewhat throughout the world, depending on the origins of the practice and the quality of the local resources. However, as general observations, it may be noted that:

- Grouts are designed to provide high strength and stability, but must also be pumpable. As shown in Figure 4-3, this implies typical water/cement (w/c) ratios in the range of 0.40 to 0.50 by weight for micropile grout.

- Grouts are produced with potable water, to reduce the danger of reinforcement corrosion.

- Type I/II cement conforming to ASTM C150/AASHTO M85 is most commonly used, supplied either in bagged or bulk form depending on site condition, job size, local availability, and cost.

- Neat cement-water grout mixes are most commonly used, although sand is a common additive in certain countries (e.g., Italy, Britain). Bentonite (which reduces grout strength) is used in primary mixes only with extreme caution, while additives are restricted only to those that improve pumpability over long distances and/or in hot conditions (e.g., high-range water reducers).
- Design compressive strengths of 28 to 35 MPa can reasonably be attained with properly produced neat cement grouts.

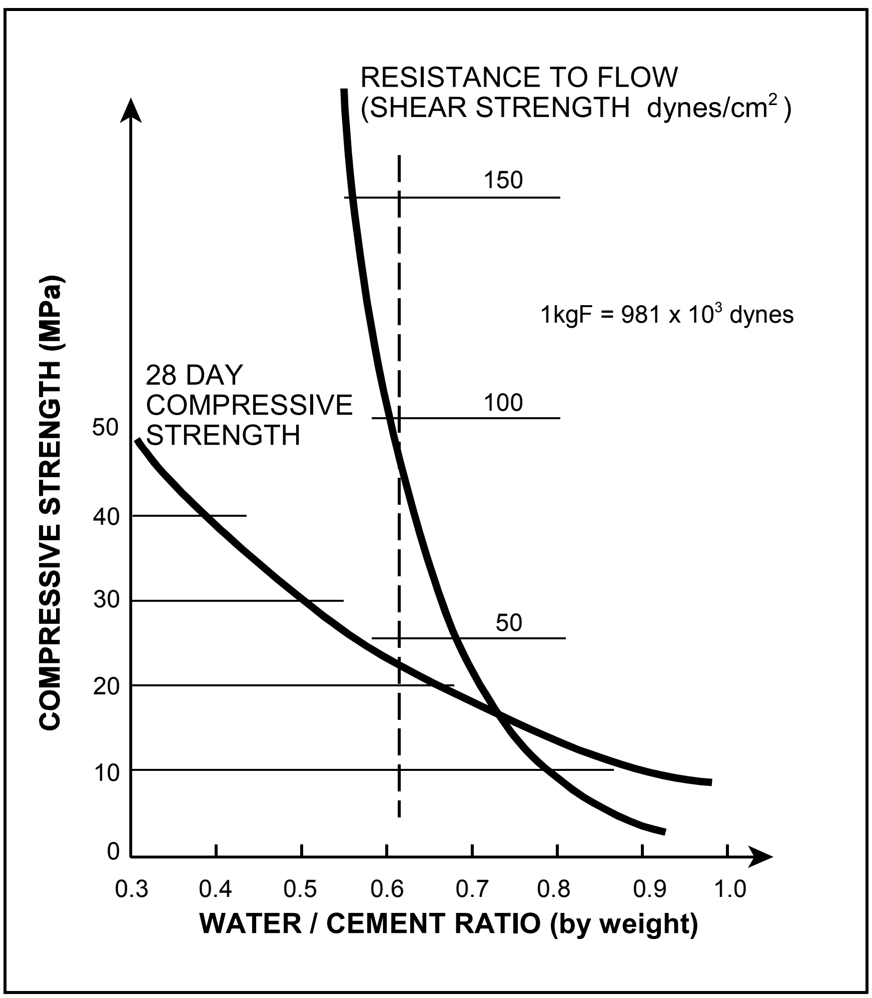


Figure 4 - 3. Effect of Water Content on Grout Compressive Strength and Flow Properties (Barley and Woodward, 1992)

The critical importance of the grouting operation is underlined by the fact that the placed grout is required to serve a number of purposes:

- It transfers the imposed loads between the reinforcement and the surrounding ground.
- It may form part of the load-bearing cross section of the pile.
- It serves to protect the steel reinforcement from corrosion.
- Its effects may extend beyond the confines of the drill hole by permeation, densification, and/or fissuring.

The grout, therefore, needs to have adequate properties of fluidity, strength, stability, and durability. The need for grout fluidity can mistakingly lead to the increase in water content; this has a negative impact on the other three properties. Of all the factors that influence grout fluidity and set properties, the water/cement ratio is the dominant. Again, Figure 4-3 illustrates why this ratio is limited to a range of 0.45 to 0.50, although even then, additives may be necessary to ensure adequate pumpability for ratios less than 0.40.

It is essential to the integrity of the pile that on completion of the grouting operation there is no significant loss of grout from any part of the pile that will be relied upon for load bearing or corrosion protection. This can be achieved by grouting to refusal during pile formation, i.e. continue grouting until no more grout take occurs. Problems with grout loss may necessitate the use of a filler such as sand for plugging the permeable layer, or may require grouting the hole and redrilling and regrouting after set of the initial grout.

For a Type B pile, it may not always be possible to attain the desired pressures during grouting; the soil seal around the casing may not always be adequate to contain the pressurized grout. This may occur after partial pressure grouting of the bond length. If this occurs, the grout should be pumped until the level reaches the top of pile, at which time grouting is discontinued. Maintaining grout pressures at a reasonable level (0.70 MPa or less) will help prevent this from occurring. If the bond lengths of the test piles (that verified the geotechnical capacity) are grouted full length with the desired pressure, questions may be raised as to the adequacy of piles that are grouted under partial pressure. One benefit of conducting pile tests to a high load

(150 to 200 percent design load) or to bond failure is that it helps to determine if the piles have excess geotechnical capacity, relaxing this concern. Production proof tests may be conducted on the suspect piling.

Because the grout is such a vital component of the micropile, close attention must be paid to the control and quality of the product. A grout quality control plan, which at the minimum should include cube compression testing and grout density (water /cement ratio) testing, is discussed in Chapter 8.

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

4.C.1 Grout Equipment

As a general statement, any plant suitable for the mixing and pumping of fluid cementitious grouts may be used for the grouting of micropiles. The best quality grouts, in terms of both fluid and set properties, are produced by high-speed, high-shear colloidal mixers (Figure 4-4) as opposed to low-speed, low-energy mixers, such as those that depend on paddles (Figure 4-5). Mixing equipment can be driven by air, diesel, or electricity, and is available in a wide range of capacities and sizes from many manufacturers.

For grout placement, lower pressure injection (say, to 1 MPa) is usually completed using constant pressure, rotary-screw type pumps (Moyno pumps), while higher pressure grouting, such as for Type C or D micropiles, usually requires a fluctuating pressure piston or ram pump.

4.C.2 Grout Mixing

The measured volume of water is usually added to the mixer first, followed by cement and then aggregate or filler if applicable. It is generally recommended that grout be mixed for a minimum of two minutes and that thereafter the grout be kept in continuous slow agitation in a holding tank prior to being pumped to the pile. Only in extreme cases- for example, where exceptionally large takes are anticipated- should ready- mix grout supply be required. The grout should be injected within a certain maximum time after mixing. This "safe workability" time should be determined on the basis of on- site tests, as it is the product of many factors, but is typically not in excess of one hour.

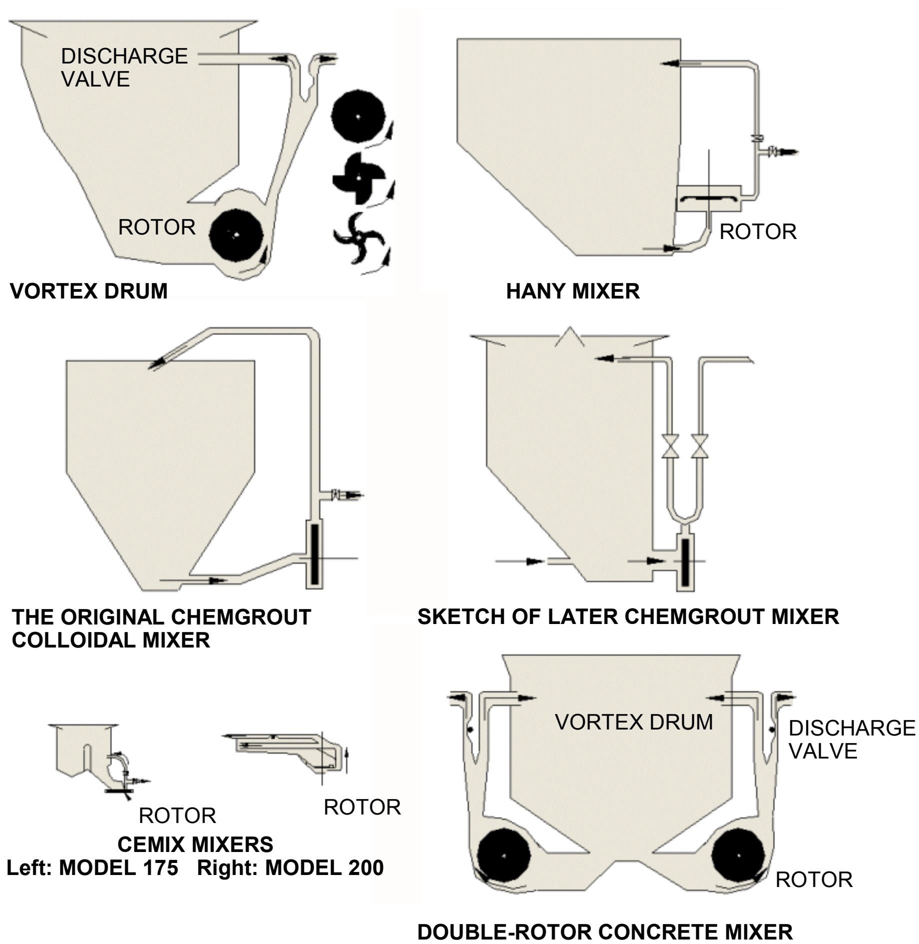


Figure 4 - 4. Various Types of Colloidal Mixers

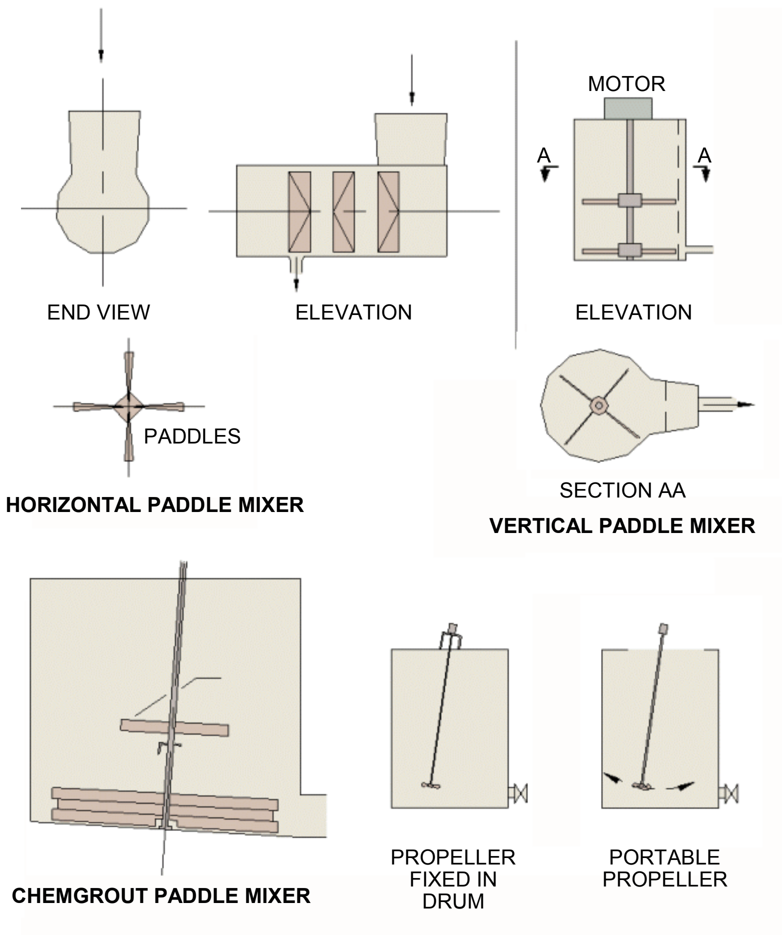


Figure 4 - 5. Various Types of Paddle Mixers

The water is typically batched into the mixer by means of a calibrated tank or flow meter. Cement is typically batched by weight, either in bags or by bulk from a silo. Sand or fillers are also batched by weight from premeasured bags or more commonly, by using a gagebox that has previously been checked and weighed. For bulk material, some method must be provided for controlling the quantities of components (volume or weight measurement) added to the mix. Admixtures are usually provided ready-proportioned to a single bag of cement, or the dosage can be adjusted by the mixer operator.

4.C.3 Grout Placement Techniques

Four classifications of grouting methodology are established in Chapter 2 of this manual. These methods are described in further detail as follows.

4.C.3.1 Gravity Fill Techniques (Type A Micropiles)

Once the hole has been drilled to depth, it is filled with grout and the reinforcement is placed. Grout should always be introduced into the drill hole through a tremie pipe exiting at the bottom of the hole. Grout is pumped into the bottom of the hole until grout of similar quality to that being injected is freely flowing from the mouth of the borehole. No excess pressure is applied. Steps are taken to ensure that the quality of grout is maintained for the full length of the borehole. This type and phase of grouting is referred to as the *primary treatment*.

The grout usually comprises a neat cement mix with w/c ratio between 0.45 and 0.50 by weight. Additionally, sanded mixes of up to 1:1 or 2:1 sand:cement ratio have been used in European practice, but they are becoming less common due to a growing trend towards the use of higher grouting pressures involving neat cement grouts. Gravity fill techniques tend now to be used only when the pile is founded in rock, or when low-capacity piles are being installed in stiff or hard cohesive soils, and pressure grouting is unnecessary (Bruce and Gemme, 1992). For sanded mixes, the w/c ratio is often extended to 0.60, assuming the resultant mix remains stable (Barley and Woodward, 1992).

4.C.3.2 Pressure Grouting Through the Casing (Type B Micropiles)

Additional grout is injected under pressure after the primary grout has been tremied, and as the temporary casing is being withdrawn. The aim is to enhance the grout/soil bond characteristics. This operation can be limited to the load transfer length within the design-bearing stratum, or may be extended to the full length of the pile where appropriate.

Pressure grouting is usually conducted by attaching a pressure cap to the top of the drill casing (this is often the drilling head itself) and injecting additional grout into the casing under controlled pressure. In the early days, pressurization of the grout was achieved by applying compressed air through the grout line, since contemporary drill head details and grout pump technology could not accommodate the relatively viscous, sand-cement mortars. This method has now been rendered obsolete by the developments in pump capabilities, combined with the trend to use stable, neat cement grouts without sand.

Grout pressures are measured as close to the point of injection as possible, to account for line losses between pump and hole. Commonly, a pressure gauge is mounted on the drill rig and monitored by the driller as a guide to rate of casing withdrawal during the pressurization phase. Alternatively, if a grouting cap is used and the casing is being extracted by means other than the drill rig (e.g., by hydraulic jacks), it is common to find a pressure gauge mounted on the cap itself. Practitioners acknowledge that there will be line losses in the system, but typically record the pressure indicated on the pressure gauge without the correction, reasoning that such losses are compensated by the extra pressure exerted by the grout column due to its weight in the borehole.

American practice is to inject additional grout at a typical average pressure between 0.5 to 1MPa, with the aim of reinstating insitu lateral soil pressures that may have been reduced by the drilling process and achieving permeation into coarser grained granular soils or fractured rocks. The effective injection pressures (typically 20 kPa per meter of depth in loose soils and 40 kPa per meter of depth in dense soils) are dictated by the following factors:

- The need to avoid ground heave or uncontrolled loss of grout.
- The nature of the drilling system (permissible pressures are lower for augers due to leakage at joints and around the flights).
- The ability of the ground to form a “seal” around the casing during its extraction and pressure grouting.
- The need to avoid “seizing” the casing by flash setting of the grout due to excessive pressure, preventing proper completion of the pile.
- The “groutability” of the ground.

- The required grout/ground bond capacity.
- Total pile depth.

The injection of grout under pressure is aimed at improving grout/ground skin friction, thus enhancing the load-carrying capacity of the micropile. Extensive experience with ground anchors has confirmed the effect of pressure grouting on ultimate load-holding capacity. This is discussed in detail in Chapter 5.

When pressure grouting in granular soils, a certain amount of permeation and replacement of loosened soils takes place. Additionally, a phenomenon known as pressure filtration occurs, wherein the applied grout pressure forces some of the integral mixing water out of the cement suspension and into the surrounding soil. This process leaves behind a grout of lower water content than was injected and is thus quicker setting and of higher strength. It also causes the formation of cake-like cement paste along the grout/soil interface that improves bond. In cohesive soils, some lateral displacement, compaction, or localized improvement of the soil can occur around the bond zone, although the improvement is generally less well marked than for cohesionless soils.

Pressure grouting also appears to cause a recompaction or redensification of the soil around the borehole and increases the effective diameter of the pile in the bond zone. These mechanisms effectively enhance grout/soil contact, leading to higher skin friction values and improved load/displacement performance. Such pressure grouting may also mechanically improve the soil between piles. This is an interesting concept within the CASE 2 pile application but is, as yet, untested.

4.C.3.3 Postgrouting (Type C and D Micropiles)

It may not be possible to exert sufficiently high grout pressures during the casing removal stage. For example, there may be ground hydrafracture or leakage around the casing. Alternatively, some micropile construction methods may not use or need a temporary drill casing, and so pressure grouting of the Type B method is not feasible. These circumstances have led to the development of post-grouting techniques, whereby additional grout can be injected via special grout tubes some time after the placing of the primary grout. Such grouts are always neat cement-water mixes (for the ease of pumpability) and may therefore have higher water contents than the primary grout, being in the range of 0.50 to 0.75 by weight. It is reasoned that excess water from these mixes is expelled by pressure filtration during passage into the soil, and so the actual placed grout has a lower water content (and therefore higher strength).

As described in the following paragraphs, high postgrouting pressures are typically applied, locally, for quite restricted periods; it may only take a few minutes to inject a sleeve. As mentioned by the Federal Highway Administration (1997) report, Herbst noted that the required aim of providing higher grout/ground bond capacity may, in fact, be more efficiently achieved in Type B micropiles, where grouting pressures are lower but are exerted over a larger area and a much longer period. This has yet to be evaluated.

The construction-based classification of Section 2.A.2 identified two types of postgrouted piles, namely Type C and Type D.

Type C: Neat cement grout is placed in the hole as done for Type A. Between 15 and 25 minutes later, and before hardening of this primary grout, similar grout is injected once from the head of the hole without a packer, via a 38- to 50-mm diameter preplaced sleeved grout pipe (or the reinforcement) at a pressure of at least 1 MPa.

Type D: Neat cement grout is placed in the hole as done for Type A. When this primary grout has hardened, similar grout is injected via a preplaced sleeved grout pipe. Several phases of such injection are possible at selected horizons and it is typical to record pressures of 2 to 8 MPa, especially at the beginning of each sleeve treatment when the surrounding primary grout must be ruptured for the first time. There is usually an interval of at least 24 hours before successive phases. Three or four phases of injection are not uncommon, contributing additional grout volumes of as much as 250 percent of the primary volume.

Variations on the technique exist. The postgrout tube can be a separate 25- or 38-mm diameter sleeved plastic pipe (tube à manchette) placed together with the steel reinforcement (Figure 4-6), or it can be the reinforcement tube itself, suitably sleeved (Figure 4-7). In each of these cases, a double packer may be used to grout through the tubes from the bottom sleeve upwards.

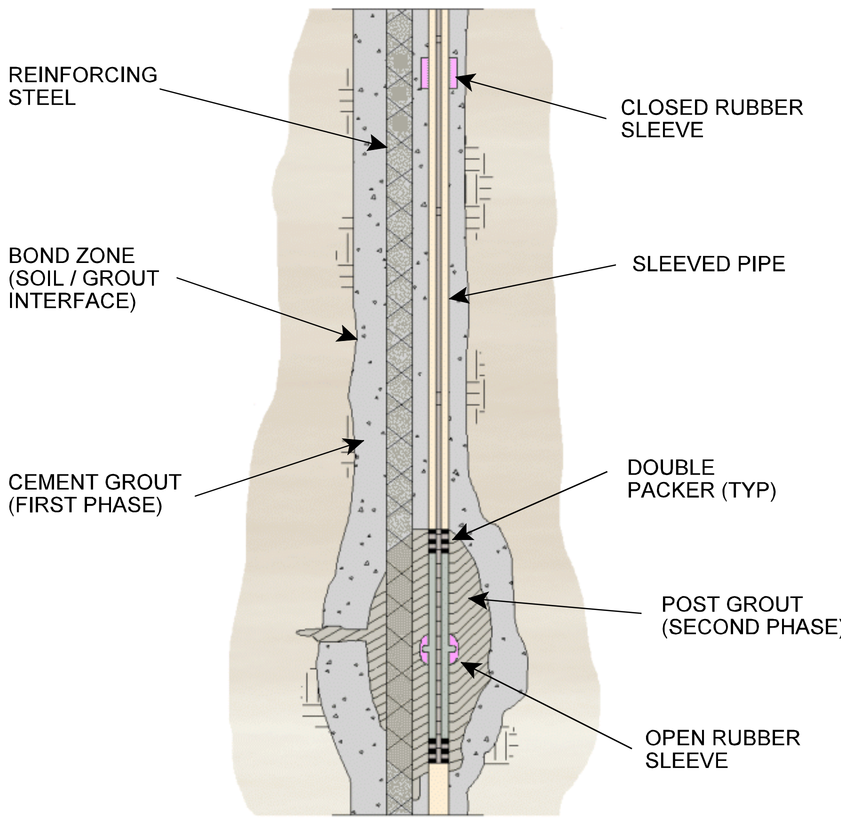


Figure 4 - 6. Principle of the Tube à Manchette Method of Postgrouting Injection

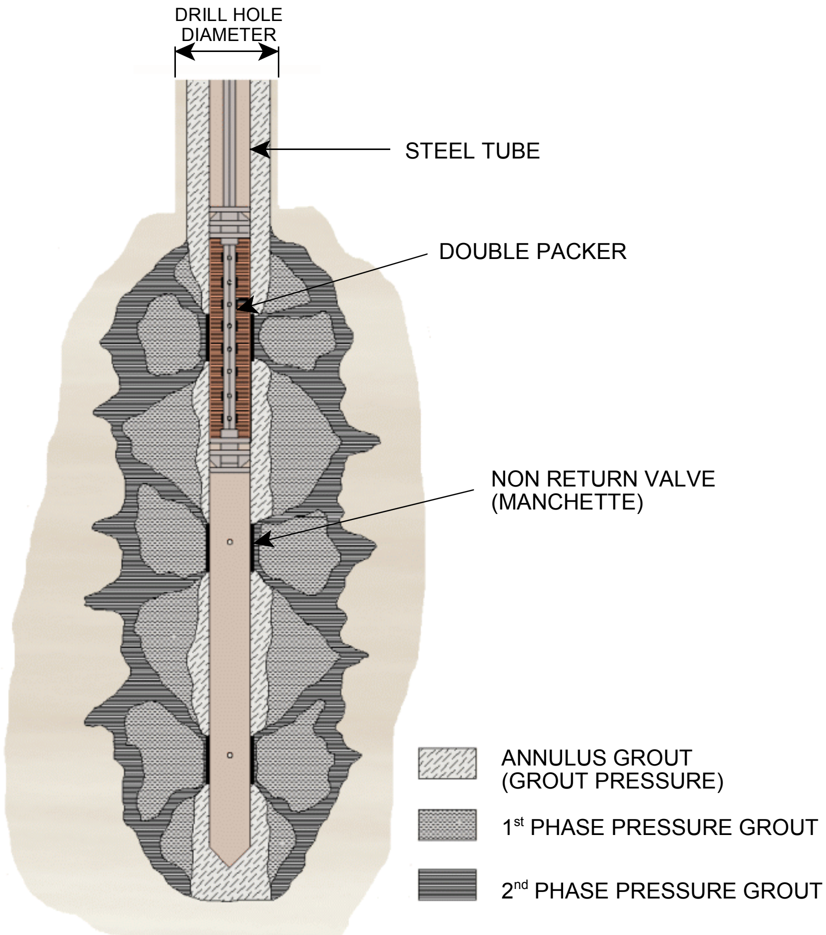


Figure 4 - 7. Use of Reinforcement Tube as a Tube á Manchette Postgrouting System

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

A slight cautionary note against the use of postgrouting was raised by Dr. Lizzi, who was concerned about leaving "foreign materials" (e.g., the postgrouting tubes) in the pile, and about contractors being tempted to use low-strength bentonite primary grouts to reduce "break out" pressures. Consideration should be given to these issues if the pile structural design places dependency on the compressive strength of the grout.

4.C.4 Top-Off (Secondary) Grouting

Due to slow grout seepage, bleed, or shrinkage, it is common to find that the grout level drops a little prior to stiffening and hardening. In ground anchorage practice, this is simply rectified by topping off the hole with the lowest water-content grout practical, at some later phase. However, in micropile practice where a permanent casing for reinforcement of the upper pile length is not used, such a cold joint is best avoided, since the grout column should be continuous for load holding and corrosion protection reasons. Topping off is therefore best conducted during the stiffening phase to ensure integrity. Where particularly high interfacial bond stresses must be resisted between the pile and an existing structure, the use of a high-strength non-shrink grout may be considered.

4.D REINFORCING STEEL

The amount of steel reinforcement placed in a micropile is determined by the loading it supports and the stiffness required to limit the elastic displacement. Reinforcement may consist of a single reinforcing bar, a group of reinforcing bars, a steel pipe casing or rolled structural steel.

4.D.1 Placement of Reinforcement

Reinforcement may be placed either prior to grouting, or placed into the grout-filled borehole before the temporary support (if used) is withdrawn. It must be clean of deleterious substances such as surface soil and mud that may contaminate the grout or coat the reinforcement, impairing bond development. Suitable centralizers should be firmly fixed to maintain the specified grout cover. Pile cages and reinforcement groups, if used, must be sufficiently robust to withstand the installation and grouting process and the rotation and withdrawal of the temporary casing.

4.D.2 Reinforcement Types

Further description of the various types of reinforcement follows.

Concrete Reinforcing Steel Bars (rebar): Standard reinforcing steel (Table 4-2), conforming to ASTM A615/AASHTO M31and ASTM A706, with yield strengths of 420 and 520 MPa, is typically used. Bar sizes range in diameter from 25 mm to 63 mm. A single bar is typically used, but a group of bars is possible. For a group, the individual bars can be separated by the use of spacers or tied to the helical reinforcement, to provide area for grout to flow between the bars and ensure adequate bonding between the bars and grout (Figure 4-8). Or bars can be bundled, provided adequate development length for bundle is provided.

For low overhead conditions where placement of full-length bars is not feasible, mechanical couplers can be used. Field adjustment of individual bar lengths can be difficult if the coupler type requires shop fabrication.

Continuous-Thread Steel Bars: Steel reinforcing bars that have a continuous full-length thread, such as the Dywidag Systems International (DSI) Threadbar or the Williams All-Thread Bar.

The DSI Threadbar system, which is also named a GEWI pile (Figure 4-9 and Table 4-3), is a common choice throughout the world for micropile reinforcement. The bar has a coarse pitch, continuous ribbed thread rolled on during production. It is available in diameters ranging from 19 mm to 63 mm in steel conforming to ASTM A615/AASHTO M 31, with yield strengths of 420, 520, and 550 MPa. The size range of 44 mm to 63.5 mm is most commonly used. Higher strength bars of steel conforming to ASTM A722/AASHTO M 275 with an ultimate strength of 1,035 MPa are also available, in diameters of 26, 32, and 36 mm.

Table 4-2. Dimensions, Yield, and Ultimate Strengths for Standard Reinforcing Bars

	Grade 420						Grade 520			
Rebar Size (mm)	25	29	32	36	43	57	36	43	57	63
Area (mm^2)	510	645	819	1006	1452	2581	1006	1452	2581	3168
Yield Strength (kN)	211	267	339	416	600	1068	520	751	1334	1742
Ultimate Strength (kN)	316	400	508	624	901	1601	690	997	1770	2168

Notes:

1. Grade 420 reinforcing steel has yield stress of f_y= 420 MPa and tensile strength of f_u = 620 MPa.
2. Grade 520 reinforcing steel has yield stress of f_y= 520 MPa and tensile strength of f_u = 690 MPa.
3. 63 mm threadbars by DSI have a minimum yield strength of f_y = 550 MPa.

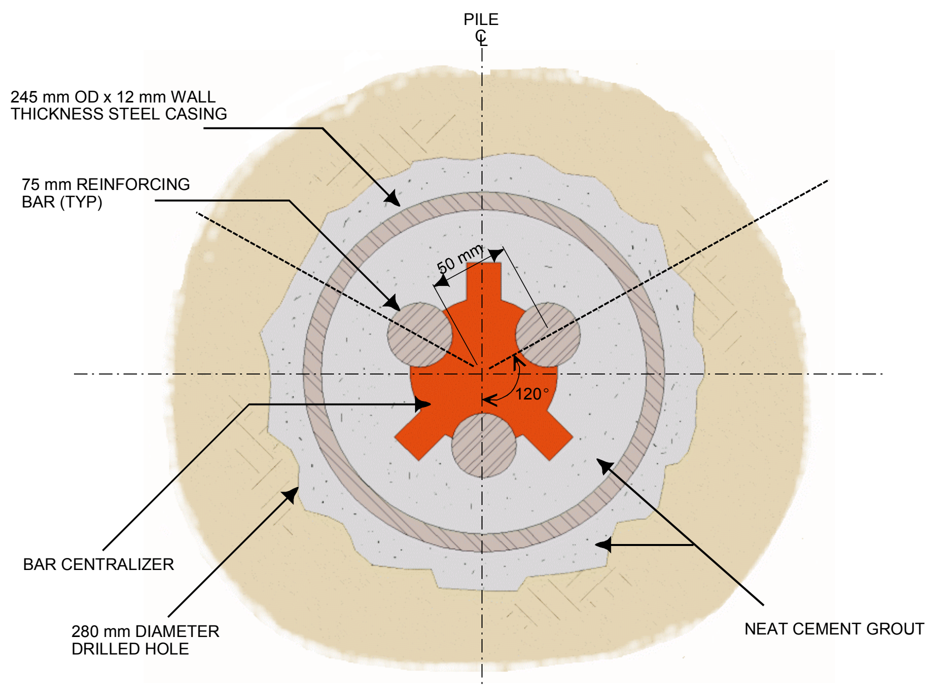


Figure 4 - 8. Multiple Bar Reinforcement with Bar Centralizer/Spacer

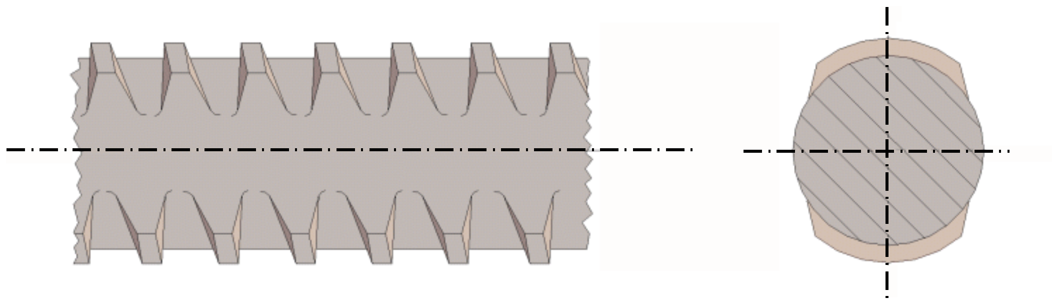


Figure 4 - 9. Details of Continuously Threaded Dywidag Bar (DSI, 1993)

The Williams All-Thread Bar is available in diameters ranging from 20 mm to 53 mm in steel conforming to ASTM A615/AASHTO M 31 and from 26 mm to 45 mm in steel conforming to ASTM A722/AASHTO M 275 , with an ultimate strength of 1,035 MPa. The bar has a finer thread than used on the Dywidag bar.

The thread on the bars not only ensures grout-to-steel bond, but also allows the bar to be cut at any point and joined with a coupler to restore full tension/compression capacity. The continuous thread also simplifies pile-to-structure connections where the bar is connected to a anchor plate. A hex nut is used to connect the plate, with the continuous thread allowing easy adjustment of the plate location.

Continuous-Thread Hollow-Core Steel Bars: Steel reinforcing bars that have a hollow core and a continuous full-length thread include the Dwyidag (Type MAI), Ischebeck Titan and Chance IBO Injection Boring Rods. These bar types offer the advantages of continuous thread, and the hollow core allows the bar to be used to drill the pile hole. A drill bit is mounted on the tip of the bar, and the bar is drilled in with grout flush pumped to the bit through the hollow core of the bar. Alternately, an air or water flush can be used, with the grout placed through the bar after drilling to the final depth.

Table 4-3 Dywidag Threadbars – Technical Data (Courtesy of DSI)

DYWIDAG THREAD BARS TECHNICAL DATA [METRIC UNITS]								
STEEL GRADE f_y/f_u N/mm^2, (MPa)		NOMINAL DIAMETER mm		STEEL AREA A_s mm^2	YIELD LOAD $P_Y=f_Y*A_s$ kN	UTL. LOAD $P_u=f_u*A_s$ kN	UNIT WT. kg/m	MAX BAR φ mm
413/620 MPa CSA G30.18M92 Hot-Rolled Reinforcing Threadbar		19	#6	284	117		2.24	22.00
		22	#7	387	160		3.04	25.30
Tie-rods	Hanging rods	25	#8	510	211		3.97	28.50
Gewi-piles	Anchor bolts	28	#9	645	266		5.06	32.20
Concrete reinforcing	Seismic anchors	32	#10	819	338		6.40	36.40
Rock bolts	Ground anchors	35	#11	1,006	415		7.91	41.00
Sail nailing	Precast connections	44	#14	1,452	600		11.38	47.30
		57	#18	2,581	1,066		20.24	64.00
517/690 MPa CSA G30.18M92 Hot-Rolled Reinforcing Threadbar		19	#6	284	146		2.24	22.00
		22	#7	387	200		3.04	25.30
(SAME AS ABOVE)		25	#8	510	263		3.97	28.50
		57	#18	2,581	1,234		20.24	64.00
		63.5	#20	3,167	1,742		24.86	69.00
500/550 MPa CSA G30.18M92 Quenched & Tempered Weldable Low Alloy Threadbar		25T	25T	491	246		3.85	28.50
		28T	28T	616	308		4.83	32.20
Seismic Restrainers Seismic Anchors Ductile Reinforcement Cold Region Anchors and Anchor Bolts		32T	32T	804	402		6.31	36.40
		40T	40T	1,260	628		9.87	44.40
		50T	50T	1,960	982		15.41	56.00
835/1030 MPa CSA G279 M82 Hot-Rolled Proof Stressed & Stress Relieved		26	1"	551	460	568	4.48	31.00
		32	1¼"	806	673	828	6.53	37.00
P.T. Ground Anchors Post-Tensioning		36	1⅜"	1,018	850	1,049	8.27	41.40
900/1100 MPa Hot-Rolled Threadbar		15	⅝"	177	159	195	1.50	18.00
Form Ties	Temp. Soil Nailing Post-Tensioning	20	¾"	315	284	347	2.60	23.00
800/900 MPa Cold-Rolled Threadbar		15	⅝"	190	150	170	1.40	17.00
Form-Use	Temp. Soil Nailing	20	¾"	335	270	300	2.50	22.00
480/600 MPa DSI-MAI Hollow-Core Self-Drilling Injection Anchors		R25N	1"		150	200	2.60	25.00
		R32N	1¼"		230	280	3.50	32.00
Temporary Shoring	Temp. Soil Nailing	R38N	1½"		430	500	6.00	38.00
Micropiles (Gewi-piles)	Overburden Drilling Without Casing	R51N	2"		630	800	9.63	51.00

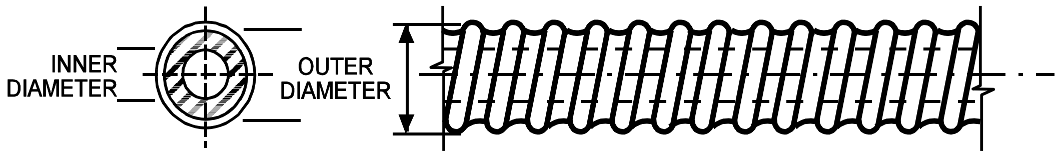
Approximate Modulus of Elasticity, E = 200,000 MPa

The continuous thread allows the bar to be cut to length and coupled, and allows the use of a hex nut for the pile top connection. The main drawback of this type of reinforcement is the higher cost. Table 4-4 presents various bar sizes and strength capacities for the MAI, Titan and IBO bars. *(Caution to designers and specification writers: Currently these types of proprietary bar systems are manufactured outside of the United States and are subject to Buy America or Buy American Provisions on federally funded transportation projects.)*

Steel Pipe Casing: With the trend towards micropiles that can support higher loads at low displacements and for the requirement to sustain lateral loads, steel-pipe reinforcement has become more common. Pipe reinforcement can provide significant steel area for support of high loading and contribution to the pile stiffness, while providing high shear and reasonable bending capacity to resist the lateral loads.

Pipe reinforcement is placed by either using the drill casing as permanent reinforcement, or by placing a smaller diameter permanent pipe inside the drill casing. Use of the drill casing for full-length reinforcement is typical only for micropiles founded in rock, where extraction of the casing for pressure grouting is not necessary. The length of the pipe sections used is dictated by the length of the drill mast and by the available overhead clearance. Casing sections are typically joined by a threaded connection, which is machined into the pipe. The reduced area of the threaded joint should be considered in the structural design of the pile, particularly for the capacity in tension and bending. Methods exist for reinforcement of the threaded joints that can provide a strength equivalent to the full casing section.

Table 4-4. Hollow Injection Bars



Rod Size	Yield (kN)	CAPACITY (kN)				Diameter (mm)		Weight (kg/m)
		Ultimate 100%	Test 80%	Design Load 70%	Design Load 60%	Inner	Outer	
MAI R25N	150	200	160	140	120	12	25	2.6
MAI R32N	230	280	224	196	168	18	32	3.5
MAI R38N	430	500	400	350	300	19	38	6.0
MAI R51N	630	800	640	560	480	34	51	9.6
TITAN 40/16	525	660	528	462	396	16	40	6.9
TITAN 73/53	970	1160	928	812	696	53	73	12.8
TITAN 103/78	1570	1950	1560	1365	1170	78	103	24.7
IBO R2512	160	190	152	133	114	12	25	2.5
IBO 3220	220	250	200	175	150	19	30	3.5
IBO 3817	420	490	392	343	294	17	38	6.6

Pipe in the sizes typically used for micropile construction are available in steel conforming to ASTM A53, A519, A252 and A106 with a typical yield strength of 241 MPa. Availability of the desired pipe size may determine the grade of steel used. The main drawbacks of using these pipe grades is the relatively low yield strength and a very high unit cost per linear meter.

API 5CT or 5L (N-80) casing may be used. The high yield strength of 551 MPa greatly aids the micropile's ability to support high loads, and improves the strength of threaded joints machined into the pipe wall. The pipe is also readily available in the form of mill secondary material at a reasonable unit cost. The overwhelming majority of higher capacity micropiles installed to date in the United States have used the N-80 casing. The use of this pipe source requires verification of steel quality through tensile and chemical testing of sampled steel rather than through mill certification, which is typically not available.

Due to the high strength and typical chemical composition of the API N-80 casing, weldability of the casing requires special welding procedures. Prior to welding the N-80 casing, welding procedures must be submitted to the owner for approval.

Pipe dimensions and yield strengths, for various grades of steel, are presented in Table 4.5.

Table 4-5. Dimensions and Yield Strength of Common Micropile Pipe Types and Sizes

API N-80 Pipe – Common Sizes				
Casing OD Wall (mm)	139.7	177.8	177.8	244.5
Thickness Steel (mm)	9.17	12.65	18.54	11.99
Area (mm^2)	3760	6563	9276	8756
Yield Strength (kN)	2075	3619	5117	4829

ASTM A519, A106 Pipe – Common Sizes				
Casing OD Wall (mm)	141.3	168.3	203.2	273.1
Thickness Steel (mm)	12.7	12.7	12.74	15.9
Area (mm^2)	5128	6204	8230	12,820
Yield Strength (kN)	1919	2319	3075	4794

Notes:

1. Casing outside diameter (OD) and wall thickness (t) are nominal dimensions.
2. Steel area is calculated as $A_s = (\pi/4) \times (OD^2 - ID^2)$
3. Nominal yield stress for API N-80 steel is F_y = 551 MPa
4. Nominal yield stress for ASTM A519 & A106 steel is F_y = 241 MPa
5. Other pipe sizes are manufactured but may not be readily available. Check for availability through suppliers.

Composite Reinforcement: For micropiles with partial length permanent drill casing (Type 1C, 2C and 4C), the use of a steel bar for reinforcement of the bottom portion of the pile is common, resulting in a composite reinforced pile (Figure 4-10). The reinforcing bar may be extended to the top of the micropile for support of tension loading. The use of varying reinforcement adds complexity to the pile structural analysis, with particular attention needed for the location of reinforcement transition.

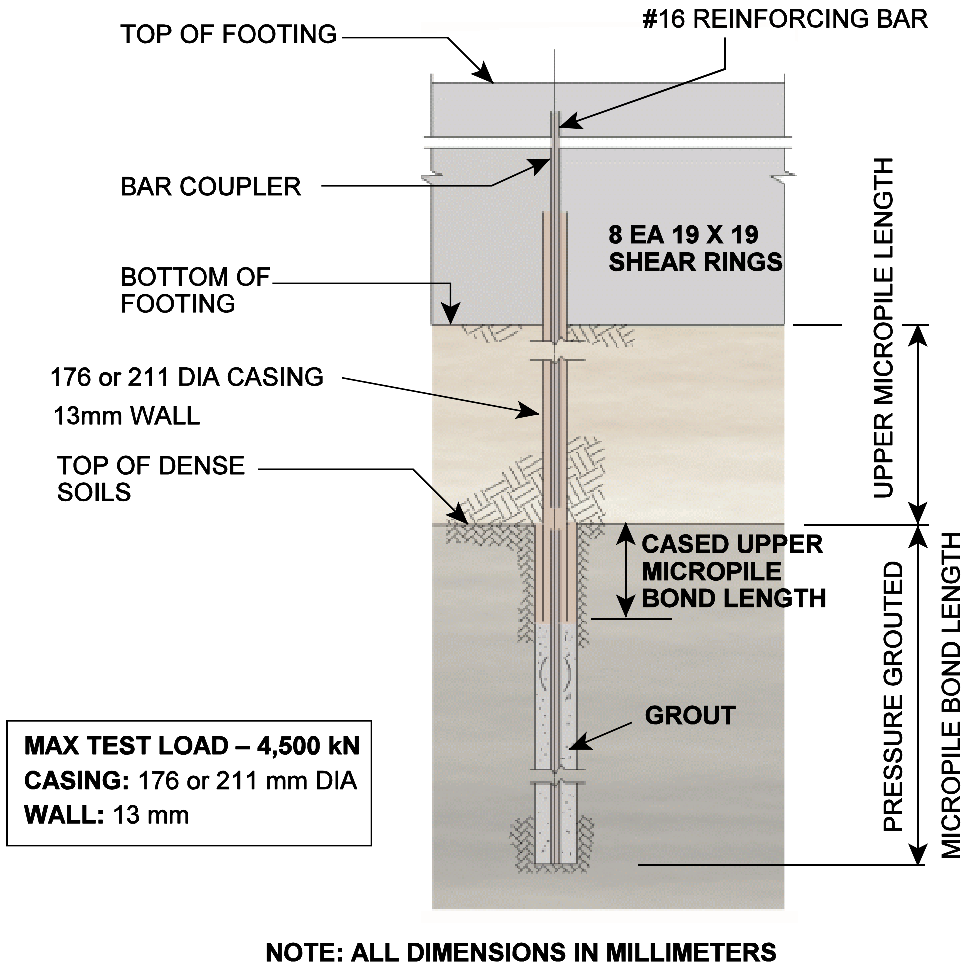


Figure 4 - 10. Details of Composite High-capacity Type 1B Micropiles, Vandenberg Air Force Base, California

4.D.3 Reinforcement Corrosion Protection

Traditionally, with the exception of permanent tension piles in aggressive ground conditions, little corrosion protection other than the surrounding cement grout has been provided to the reinforcement in most countries.

The most common and simplest tests utilized to measure the aggressiveness of the soil environment include electrical resistivity, pH, chloride, and sulfate. Per FHWA-RD-89-198, the ground is considered aggressive if any one of these indicators show critical values as detailed in Table 4-6.

Table 4-6. Corrosion Critical Values

Property	Test Designation*	Critical Values*
Resistivity	AASHTO T–288 ASTM G 57	below 2,000 ohm-cm
pH	AASHTO T–289 ASTM G 51	below 5
Sulfate	AASHTO T–290 ASTM 516M ASTM D4327	above 200 ppm
Chloride	AASHTO T–291 ASTM 512 ASTM D4327	above 100 ppm

*Note – Corrosion protection requirements vary between Transportation Agencies. The designer should check test standards for latest updates and individual transportation agencies may have limits on critical values different than those tabulated above. Standard specifications or test methods for any of the above items which are common to your agency can be referenced in lieu of the above listed AASHTO / ASTM references.

The various levels of corrosion protection that can be applied to reinforcing bars are discussed below.

Grout Protection Only - Reinforcing Bar: Centralizers are applied along the length of the bar (Figure 4-11a) to ensure adequate cover of grout between the bar and the side of the borehole. Centering of the reinforcement in the grout is also structurally desirable for compression piles. Internationally, various codes require minimum grout cover of 20 to 30 mm. Alternative approaches regarding this level of protection include "geometric" corrosion protection, which relates to the concept that a progressive loss of section with time is allowable, and typical rates are widely quoted (Fleming et al., 1985). Corrosive potential of the existing ground, magnitude of the tension loading, and the structural detailing of the pile must all be considered for this level of protection.

Protective Barriers (Epoxy coating or Encapsulation) - Reinforcing Bar: Additional protection may be required in those cases where a continuous grout cover of adequate thickness cannot be guaranteed, where the pile is installed in aggressive ground conditions, or the reinforcement may cause tension cracking of the grout, providing a corrosion path to the steel. Options for protective barriers include providing a coating on the bar, such as an electrostatically applied epoxy coating, or providing a encasing sheath (encapsulation), such as corrugated plastic, with the annulus between the bar and the sheath filled with high-strength, non-shrink grout. The use of a grout-filled corrugated sheath is a common feature of permanent anchor tendons and the DSI threadbar and GEWI Bar (Figure 4-11b), and is often referred to as double corrosion protection.

For micropiles with composite reinforcement (bar and pipe), the permanent grout-filled pipe provides protection in a manner similar to the encapsulation method for the upper portion of the bar encased by the pipe. Protection may still be necessary for the uncased portion of the pile.

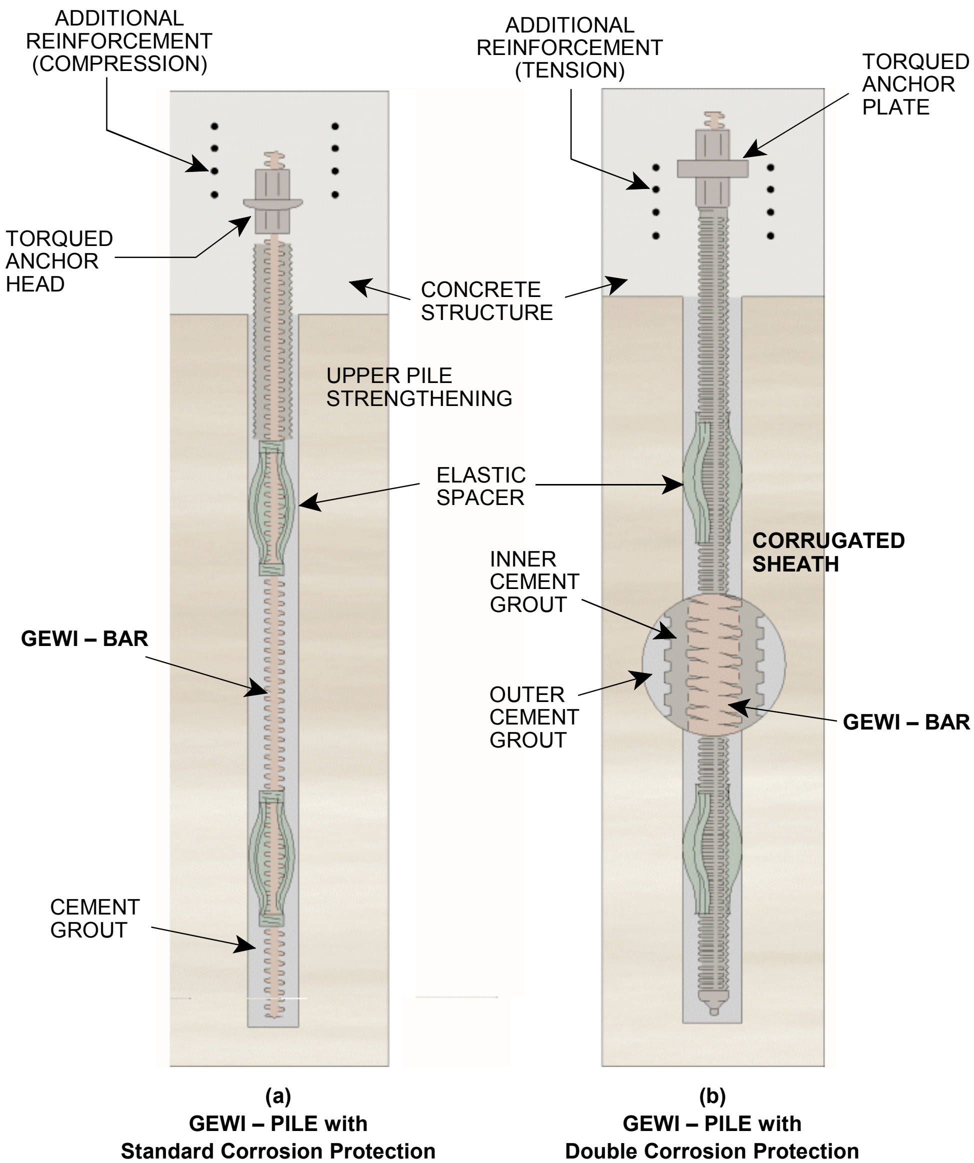


Figure 4 - 11. GEWI Piles with (a) Grout protection only, and (b) Double corrosion protection (Courtesy DSI)

Table 4-7. Minimum Dimensions (mm) of Shell Thickness as Corrosion Protection

Soil Type	Service Life (years)			
	25	50	75	100
Not Aggressive	0.25	0.60	0.70	0.80
Barely Aggressive	1.00	1.60	2.00	2.50
Very Aggressive	2.50	4.00	5.00	6.00

Source: CCTG, 1993.

Corrosion protection of pipe reinforcement can be more difficult, particularly if the drill casing has been used as the reinforcement. The various levels of corrosion protection that can be applied to permanent casings follow.

Sacrificial Steel - Casing: According to AASHTO section 4.5.7.4 (16th Edition), for concrete-filled pipe piles in installations, where corrosion may be expected, 1.6 mm shall be deducted from the shell thickness to allow for reduction in the section due to corrosion.

The French code CCTG (1993) recommends adoption of the minimum dimensions of shell thickness to be sacrificed as corrosion protection in the absence of specific studies as summarized in Table 4-7. The effect on pile strength and stiffness should be included in the consideration of sacrificial steel as corrosion protection.

Grout Cover - Casing: The pile shaft annulus around a drill casing can vary typically from 10 to 75 mm, depending on the soil conditions and methods of drilling and grouting. Grout cover may not be present in this annulus if the soil "seal" contains the grout in the lower bond length during pressure grouting, preventing it from filling in around the upper portion of the pile. This may not be a concern in cohesive soil or rock conditions. The method to ensure a minimum grout cover around pipe reinforcement, particularly in granular soil conditions, may be to place a separate permanent pipe and completely retract the drill casing.

Protective Barriers - Casing: Where the drill casing provides permanent reinforcement, the use of a coating such as epoxy paint is not desirable due to the abrasive action of the soil on the outer casing wall and probable resulting damage to or wearing off of the coating. An encasing sheath in the form of an additional outer casing provides very effective corrosion protection, but at considerable additional cost.

Protective Barriers - Pipe: Protective coating such as epoxy paint can be applied to pipe inserted into a temporarily supported hole so that the coating will not be damaged during drilling.

CHAPTER 5

DESIGN OF MICROPILES FOR STRUCTURE FOUNDATIONS (CASE 1 PILES)

5.A INTRODUCTION

This chapter outlines subsurface investigation requirements, and the geotechnical and structural design considerations for CASE 1 micropiles. The section on pile geotechnical design includes guidance for estimating the grout-to-ground bond capacity of micropiles. A table presenting nominal (ultimate) grout-to-ground bond strength values typically attainable for various soil/rock types is also included. The section on pile structural design includes methods for determination of pile component structural strengths in accordance with the 1996 AASHTO Standard Specifications for Highway Bridges, Sixteenth Edition.

The geotechnical load capacity of a micropile is highly sensitive to the processes used during pile construction, principally the techniques used for drilling the pile shafts, flushing the drill cuttings, and grouting the pile. Therefore, verification of the grout-to-ground nominal bond strength assumed in design via pile load testing during construction is essential to ensure structure safety; the construction load testing should be considered an extension of design.

The basic philosophy of micropile design differs little from that required for any other type of pile. The system must be capable of sustaining the anticipated loading conditions with the pile components operating at safe stress levels, and with resulting displacements falling within acceptable limits. For conventional piling systems, where the large cross sectional area results in high structural capacity and stiffness, the design is normally governed by the geotechnical load carrying capacity. With a micropile's smaller cross sectional area, the pile design is more frequently governed by structural and stiffness considerations. This emphasis on the structural pile design is further increased by the high grout-to-ground bond capacities that can be attained using the pressure grouting techniques described in Chapters 2 and 4. Furthermore, the use of higher strength steel reinforcement in the micropile can result in a design that is governed by stiffness considerations.

With respect to axial, lateral, or combined loading, the design of micropiles consists of two basic aspects:

- The evaluation of the geotechnical load capacity of the micropile, which requires appropriate estimation of the grout/ground interface parameters and the state of stress in the ground after micropile installation (effects of pressure grouting).
- The structural load capacity and stiffness performance of the micropile section which depends mainly on the area of the composite reinforced micropile and the strength of the section materials.

The following design steps are suggested:

1. Review available project information.
   (a) Requirements of the job, pile loading requirements, pile layout constraints.
   (b) Special conditions such as available access and overhead clearance, presence of hazardous materials, environmental constraints.
   (c) Contractual requirements.
2. Review geotechnical data.
   (a) Obtain geotechnical/geological subsurface profile.
   (b) Estimate geotechnical design parameters.
   (c) Obtain soil properties that determine corrosion protection requirements.
   (d) Identify problem areas if any.

3. Complete initial geotechnical pile design.

    (a) Estimate load transfer parameters (grout-to-ground bond) for the different subsurface layers and determine the pile bond length required to support the loading.

    (b) Evaluate pile spacing for impact to geotechnical capacity from group effects.

4. Complete pile structural design for the various components:

    (a) Pile cased length structural capacity (bar and/or pipe reinforcement with grout).

    (b) Pile uncased length structural capacity (bar reinforcement with grout).

    (c) Grout to steel bond capacity.

    (d) Transition between reinforcement types (cased to uncased section).

    (e) Strain compatibility between structural components/ductility.

    (f) Reinforcement splice connections (bar and/or pipe reinforcement).

    (g) Pile to footing connection.

5. Complete combined geotechnical and structural design considerations.

    (a) Anticipated settlement/required stiffness analysis.

    (b) Lateral load capacity/Anticipated lateral displacement and combined stresses (axial + bending) due to lateral loading conditions.

    (c) Buckling of the pile/soil lateral support considerations.

6. Complete additional micropile system considerations.

    (a) Corrosion protection requirements.

    (b) Determine construction load testing and quality control program requirements.

    (c) Examine constructability and cost effectiveness of the design.

5.B COMMENT ON THE USE OF THIS MANUAL FOR DESIGN

This manual emphasizes that the geotechnical load capacity of a micropile can be highly sensitive to the processes used during pile construction, principally the techniques used for drilling the pile shafts, flushing the drill cuttings, and grouting the pile. Problems may occur if the designer lacks expertise in micropile design and construction techniques or lacks the control of construction on site to avoid methods that may be detrimental to the pile's capacity.

Therefore, this chapter is intended to assist a designer in determining the feasibility of installing a micropile system that will meet predetermined performance criteria at a given site. The most optimum pile design and method of installation may be obtained through the use of a design/build type performance specification, allowing the use of experienced micropile specialty contractors' methods and expertise to optimize the system. This chapter also provides the designer with the necessary tools to properly evaluate a contractors proposed micropile system.

The information on specifying a design/build project element is included in Chapter 9 and provides methods for selecting a contractor who is qualified to do the work. Chapters 7 and 8 include information on load testing, inspection, and quality control, which intend to verify that the installed piles meet the specified criteria.

5.C EXPLANATION OF SLD AND LFD DESIGN METHODS

The following explanation of SLD (Service Load Design) and LFD (Load Factor Design) design methods and φ calibration procedure was developed from Appendix A of NCHRP 343 (Procedures for Evaluating Performance Factors–Load Factor Design Criteria for Highway Structure Foundations NCHRP 24-4, 1991).

Introduction

This chapter includes both the SLD–Service Load Design Method (also known as Allowable Stress Design) and the LFD–Load Factor Design Method (also known as the Strength Design Method) as described in the 1996 AASHTO Standard Specifications for Highway Bridges, Sixteenth Edition. Both methods are included in order to address the current state-of-the-practice with structural and geotechnical engineers across the United States. Structural engineers are currently using both SLD and LFD methods, however most are moving away from SLD toward LFD and its newest version, the LRFD–Load and Resistance Factor Design Method (the new AASHTO LRFD Bridge Design Specification). Geotechnical engineers are mostly using the SLD method, however they are in the early stages of moving toward the LFD and LRFD methods as well.

All three of these design methods provide factors-of-safety to allow for uncertainties in the loads and uncertainties in the structural and geotechnical material strengths due to their inherent randomness. The safety factors also account for the natural lack of perfect information and knowledge during design.

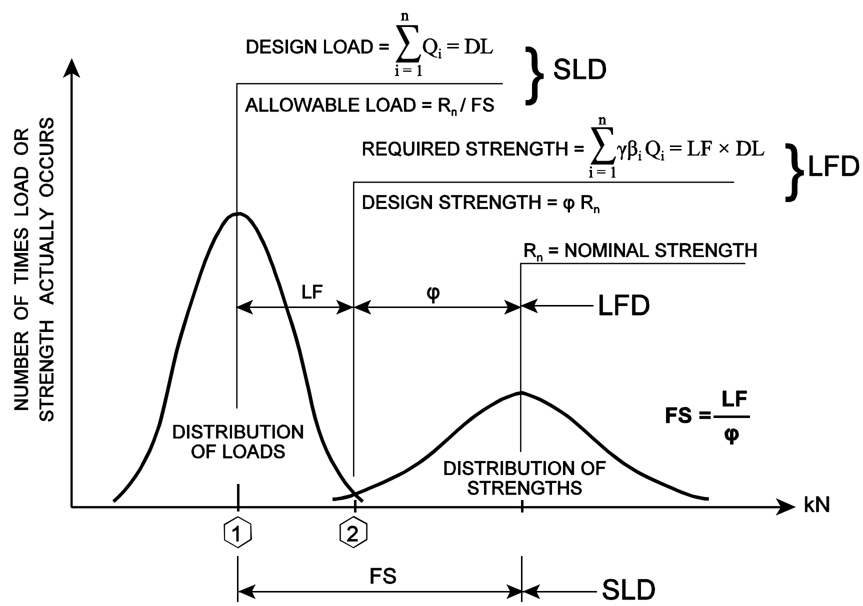


Figure 5 - 1. SLD and LFD Description and Relationships

Q_i = load effect due to load component (dead load, live load, earth pressure, etc.)

φ = strength reduction factor

γ = group load factor per AASHTO Table 3.22.1 A

β_i = individual load component factor per AASHTO Table 3.22.1 A

LF = combination of γ and β_i load factors

FS = factor-of-safety

n = number of load components

See Figure 5-1 for a description of SLD and LFD terms and their relationships. In the SLD method, the factor-of-safety is selected in a rather intuitive manner and is based on experience, tradition, and engineering judgment. SLD treats design loads and material strengths as deterministic constants. It reduces the nominal strength (ultimate strength) of the material to an allowable load which must not be exceeded by the design load. The design load is the controlling group of load components (dead load, live load, earth pressure, etc.) as prescribed in Table 3.22.1 A of the AASHTO specification. Table 3.22.1 A governs the combination of all loads except seismic loads which are covered under Division I-A of the AASHTO specification (see AASHTO section 3.21). SLD corresponds to position 1 in Figure 5-1.

In the LFD and LRFD methods the factor-of-safety is comprised of two components, load factors (LF) which amplify the design load, and a strength reduction (LFD) factor φ, which reduces the nominal strength of the material. In LFD and LRFD all the elements of a structure are assigned the same probability of failure, thus all the elements are working equally without one element being way over or under designed compared with another element of the structure. The uncertainty in load is represented by a load factor (LF) and the uncertainty in material strength is represented by a strength reduction factor φ. This allows us to adjust ("calibrate") the factors as we get more data and further improve on the design method. LFD is a probabilistic approach rather than the SLD deterministic approach toward the randomness of design loads and material strengths. It uses probabilistic theory to separate the distributions of design loads and material strengths sufficiently in order to maintain an acceptable level of risk against failure. LFD requires that the load components be amplified by load factors and grouped as prescribed in Table 3.22.1 A of the AASHTO specification. The controlling group of these factored loads (required strength) must be less than the design strength which is the nominal strength reduced by the strength reduction factor φ. As mentioned previously, Table 3.22.1 A governs the combination of all loads except seismic loads which are covered under Division I-A of the AASHTO specification (see AASHTO section 3.21). LFD corresponds to position 2 shown in Figure 5-1.

Special Design Method for Seismic Load Groups

Division I-A of the AASHTO specification specifies procedures for seismic design which are based on using the full nominal foundation strength to resist the controlling group of unfactored loads considered to coincide with seismic loads (dead load, buoyancy, stream flow, and earth pressure). For the grout-to-ground bond design this manual recommends a LFD method for seismic load groups using load factors and φ factor equal to 1.

Nominal Strength for Grout-to-Ground Bond

The nominal strength value for use in both SLD and LFD methods is obtained by picking a unit strength per surface area value from Table 5-2.

SLD Factor-of-Safety for Grout-to-Ground Bond

The factor-of-safety (FS) recommended in this chapter for the grout-to-ground bond allowable value for typical SLD designs is 2.5. This recommendation is based on experience nationwide and the design and construction procedures outlined in this manual. It is also tied to the field verification and proof test requirements outlined in Chapter 7. Reduction of this value may be justified where specific knowledge of the site ground conditions indicate very consistent and competent conditions, such as fresh unfractured rock.

LFD φ_G Factors for Grout-to-Ground Bond

At this time φ_G factors for grout-to-ground bond strengths for micropiles have not been determined systematically as required by the probablistic theory. There is, however, a wealth of experience with SLD designs for micropiles and ground anchors which are similar in many respects. Therefore this chapter recommends the determination of φ_G factors for the grout-to-ground bond strengths by calibration to the SLD factor-of-safety in order to use that body of successful knowledge and experience until the probabilistic based values are developed and accepted by practitioners for use. This procedure will provide a LFD design that equals a SLD one for the grout-to-ground bond. The following procedure shows how to calibrate LFD φ_G factors to a SLD FS. It shows that each unique group of loads requires a different φ_G factor.

SLD Method

Allowable Load ≥ Design Load

$$\frac{R_n}{FS} \geq \sum_{i=1}^{n} Q_i$$

$$\text{and} \quad R_n \geq FS \times \sum_{i=1}^{n} Q_i$$

LFD Method

Design Strength ≥ Required Strength

$$\varphi_G R_n \geq \sum_{i=1}^{n} \gamma \; B_i Q_i$$

$$\text{and} \quad \varphi_G \geq \frac{\sum_{i=1}^{n} \gamma \; B_i Q_i}{R_n}$$

As R_n is the same for SLD and LFD the φ_G required to calibrate LFD to SLD is:

$$\varphi_G = \frac{\sum_{i=1}^{n} \gamma \; B_i \; Q_i}{FS \times \sum_{i=1}^{n} Q_i}$$

For AASHTO Group I Load Group:

$$\varphi_G = \frac{\gamma \; (B_D Q_D + B_L Q_L + B_{CF} Q_{CF} + B_E Q_E + B_B Q_B + B_{SF} Q_{SF})}{FS \times (Q_D + Q_L + Q_{CF} + Q_E + Q_B + Q_{SF})}$$

$\gamma = 1.3$

$B_D = 1$; $B_L = 1.67$; $B_{CF} = 1$; $B_E = 1.3$ for lateral earth pressure

$B_B = 1$; $B_{SF} = 1$

FS = Factor of Safety

Q_D = Dead Load; Q_L = Live Load; Q_{CF} = Centrifugal Force

Q_E = Earth Pressure; Q_B = Buoyancy; Q_{SF} = Stream Flow Pressure

Table 5-1 shows how φ_G varies with different combinations of loads for AASHTO Group I with dead load, live load, and earth pressure for a FS = 2.5. The φ_G value in this table is calculated using:

$$\varphi_G = \frac{1.3 Q_D + 2.17 Q_L + 1.69 Q_E}{2.5 \; (Q_D + Q_L + Q_E)}$$

It also shows LF = φ_G (FS) which is the combination of γ and β load factors. These data indicate that $\varphi_G = 0.6$ could be used for a conservative value for typical LFD designs. Higher φ_G values may be calibrated for the controlling group of loads with their load factors.

For AASHTO Load Groups IA – X,

$$\varphi_G = \frac{\sum_{i=1}^{n} \gamma \; B_i \; Q_i}{FS \times \dfrac{\sum_{i=1}^{n} B_i \; Q_i}{OS}}$$

where OS = Overstress factor per AASHTO Table 3.22.1A.

Table 5-1. Calibrated LFD φ_G Factor for Various Ratios of Q_D, Q_L and Q_E

Q_D	Q_L	Q_E	φ_G	LF
1.0			0.52	1.3
0.9	0.1	0.0	0.55	1.38
	0.0	0.1	0.54	1.35
0.8	0.1	0.1	0.57	1.43
	0.2	0.0	0.59	1.48
0.7	0.1	0.2	0.59	1.48
	0.2	0.1	0.61	1.53
	0.3	0.0	0.62	1.55
0.6	0.1	0.3	0.60	1.50
	0.2	0.2	0.62	1.55
	0.3	0.1	0.64	1.60
0.5	0.1	0.4	0.62	1.55
	0.2	0.3	0.64	1.60
	0.3	0.2	0.66	1.65
	0.4	0.1	0.67	1.68
	0.5	0.0	0.69	1.73
0.4	0.1	0.5	0.63	1.58
	0.2	0.4	0.65	1.63
	0.3	0.3	0.67	1.68
	0.4	0.2	0.69	1.73
	0.5	0.1	0.71	1.78
	0.6	0.0	0.73	1.83
0.3	0.1	0.6	0.65	1.63
	0.2	0.5	0.67	1.68
	0.3	0.4	0.69	1.73
	0.4	0.3	0.71	1.78
	0.5	0.2	0.73	1.83
	0.6	0.1	0.74	1.85
	0.7	0.0	0.76	1.90
0.2	0.1	0.7	0.66	1.65
	0.2	0.6	.0.68	1.70
	0.3	0.5	.0.70	1.75
	0.4	0.4	0.72	1.80
	0.5	0.3	0.74	1.85
	0.6	0.2	0.76	1.90
	0.7	0.1	0.78	1.95
	0.8	0.0	0.80	2.00
0.1	0.1	0.8	0.68	1.70
	0.8	0.1	0.81	2.03

Note: LF for most designs varies from 1.3 to 1.7

5.D GEOTECHNICAL DESIGN

5.D.1 Geotechnical Investigation Requirements

The subsurface investigation required for the design of micropiles is typically no more extensive than that which is required for any other type of deep foundation element such as drilled shafts or driven piles. The following information is necessary for proper micropile design:

- General geology.
- Site history (mining, previous excavations, any problems with previous construction, construction methods for adjacent utilities or basements or foundations).
- Description of geologic processes or modes of deposition of strata.
- Logs of soil borings completed in close proximity to the structure that includes description and classification of the soil strata encountered, unit weights, moisture contents, standard penetration tests (SPT) or cone penetrometer test (CPT) values, and description of groundwater conditions. Boring depths should extend beyond anticipated tip elevations of the piles with detailed soil description, particularly for the bond zone strata.
- A subsurface soil profile along the alignment of the structure developed from the soil boring information presenting soil type and SPT values as a minimum, must be provided for evaluation of the variability of soil strata and determination of the worst case soil conditions.
- An estimation of soil shear strength parameters. Determination of liquid and plastic limits for cohesive soils, and determination of the grain size distribution for granular soils.
- If rock is encountered, logs with rock classifications, penetration rates, degree of weathering and fracturing, recovery and RQD measurements, unconfined compressive strength, and driller's observations should be provided.
- Determination and discussion of the presence of hazardous, contaminated, and/or corrosive conditions if applicable. This may include resistivity, pH, and the presence of lead, sulfates, and chloride content.

In general, all geotechnical data interpretations should be provided. The basic character and extent of the soil strata determined from the geotechnical investigation can be verified during pile installation by monitoring and logging of the penetration rates, drilling action, flush return, and soil cuttings.

5.D.2 Geotechnical Bond Capacity

For design purposes, micropiles are usually assumed to transfer their load to the ground through grout-to-ground skin friction, without any contribution from end bearing due to the following factors:

- The high grout-to-ground bond capacities that can be attained as a consequence of the micropile installation methods. These capacities can reach ultimate values in excess of 365 kN per meter of bond length in dense granular soils and 750 kN per meter of bond length in competent rock, for typical micropile bond zone diameters (150 – 300 mm).
- The area available for the skin friction is significantly greater than that for end bearing. For a pile that is 200 mm in diameter with a 6 m long bonded length, the area available for skin friction is 120 times greater than that available for end bearing.
- The pile movement needed to mobilize frictional resistance is significantly less than that needed to mobilize end bearing.

The dependence on skin friction results in a pile which is considered geotechnicaly equivalent in tension and compression. This is a common design assumption for determining the bond length for a compression/ tension pile.

The value typically considered when determining the grout-to-ground bond, either empirically or through load testing, is the average value over the entire bond length. Instrumentation of tieback anchor and micropile testing has shown that particularly for dense and stiff soils and competent rock, the rate of load transfer to the ground is higher at the top of the bond length. This is most significant when calculating anticipated pile settlements. A practical consideration is that concentration of the reaction to the applied loading in the upper portion of the bond length effectively shortens the length over which the pile deforms elastically, reducing the magnitude of the settlement, particularly in stiff soils and rock.

While the application of micropiles is growing rapidly, the current state of the practice for the geotechnical design is primarily based upon the experience and research on drilled shafts, soil nails, and tieback anchors. Detailed information on empirical methods used for estimating grout-to-ground bond capacities are available in the following publications:

- "*Post-Tensioning Institute (PTI) Recommendations for Prestressed Rock and Soil Anchors"* (1996) – Includes a section on determining bond capacity for the various types of anchors.
- *"Construction, Carrying Behavior, and Creep Characteristics of Ground Anchors," H. Ostermayer, 1975 Conference on Diaphragm Walls and Anchorages, Institute of Civil Engineers, London* – Includes information on the bond capacity of pressure grouted and post grouted anchors.
- "*Ground Control and Improvement" (1994) by Xanthakos, Abramson, and Bruce* (ISBN 0-471-55231-3, John Wiley & Sons, Inc., New York, NY) – Includes a chapter on micropiles with a brief discussion on pile geotechnical design.
- *FHWA Report No.s RD-96-016/017/018/019; Volumes I - IV: "FHWA Drilled and Grouted Micropiles State-of-the-Practice Review"* – Includes discussion on micropile research developments, construction methods, load test data and case histories
- *FHWA Report No. FHWA/RD-82/047 "Tiebacks" and FHWA-DP-68-1R "Permanent Ground Anchors"* – Includes a section on determining bond capacity for the various types of anchors.
- *FHWA Publication No. FHWA-HI-88-042 "Drilled Shafts: Construction Procedures and Design Methods"* – Includes a section on determining skin friction and end bearing capacities for drilled piers.

5.D.3 Summary of Grout-to-Ground $\alpha_{\text{Bond Nominal Strength}}$ Values

Table 5-2 provides guidance for estimating the unit values for grout-to-ground bond nominal (ultimate) strengths. The table includes ranges for the four methods of pile grouting (Type A, Type B, Type C, and Type D) in a variety of ground conditions.

Values for the grout-to-ground nominal bond strength are commonly based on the experience of the local Contractors or Geotechnical Engineers. Table 5-2 presents ranges of typical values of the nominal bond strength for various installation methods and ground conditions.

These values can be used to estimate the pile geotechnical tension and compression axial design values for SLD and LFD as shown in 5.D.3.1 and 5.D.3.2. As these values vary with actual ground conditions, drilling, grouting and pile installation procedures, the final pile design should be completed by a specialty contractor who is qualified to perform micropile design and construction (see section 5B). The following values are intended to assist a designer with preliminary design and general evaluation of specialty contractor final designs. Higher bond values may be used based on proper documentation and load test data.

Table 5-2. Summary of Typical $\alpha_{\text{bond nominal strength}}$ Values (Grout-to-Ground Bond) for Preliminary Micropile Design that have been used in Practice.

Soil / Rock Description	**Typical Range of Grout-to-Ground Bond Nominal Strengths (kPa)**			
	Type A	**Type B**	**Type C**	**Type D**
Silt & Clay (some sand) (soft, medium plastic)	35-70	35-95	50-120	50-145
Silt & Clay (some sand) (stiff, dense to very dense)	50-120	70-190	95-190	95-190
Sand (some silt) (fine, loose-medium dense)	70-145	70-190	95-190	95- 240
Sand (some silt, gravel) (fine-coarse, med.-very dense)	95-215	120-360	145-360	145-385
Gravel (some sand) (medium-very dense)	95-265	120-360	145-360	145-385
Glacial Till (silt, sand, gravel) (medium-very dense, cemented)	95-190	95-310	120-310	120-335
Soft Shales (fresh-moderate fracturing, little to no weathering)	205-550	N/A	N/A	N/A
Slates and Hard Shales (fresh-moderate fracturing, little to no weathering)	515-1,380	N/A	N/A	N/A
Limestone (fresh-moderate fracturing, little to no weathering)	1,035-2,070	N/A	N/A	N/A
Sandstone (fresh-moderate fracturing, little to no weathering)	520-1,725	N/A	N/A	N/A
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	1,380-4,200	N/A	N/A	N/A

Type A - Gravity grout only

Type B - Pressure grouted through the casing during casing withdrawal

Type C - Primary grout placed under gravity head, then one phase of secondary “global” pressure grouting

Type D - Primary grout placed under gravity head, then one or more phases of secondary “global” pressure grouting

5.D.3.1 Geotechnical Bond Length Tension And Compression Allowable Axial Load – SLD

$$P_{G-\text{allowable}} = \frac{\alpha_{\text{bond nominal strength}}}{FS} \times 3.14 \times DIA_{\text{bond}} \times (\text{bond length})$$

Use FS = 2.5 (soil or rock) for non-seismic load groups.

5.D.3.2 Geotechnical Bond Length Tension And Compression Design Axial Strength – LFD

$$P_{G-\text{design strength}} = \varphi_G \times (\alpha_{\text{bond nominal strength}}) \times 3.14 \times DIA_{\text{bond}} \times (\text{bond length})$$

Use $\varphi_G = 0.60$ for typical designs for non-seismic load groups or calibrated to SLD as shown in section 5.C.

Use $\varphi_G = 1.0$ for seismic load groups.

5.D.4 Geotechnical End Bearing Capacity

Moderately loaded micropiles have been designed for end bearing on rock. The design may be done similar to end bearing drilled shafts or driven piles, or may be based on previous load test experience of similar micropiles.

5.D.5 Group Effect For Axially Loaded Micropiles

The design of foundation support systems incorporating micropiles may require the installation of groups of closely spaced piles. With conventional piles, depending on the pile type, installation method, and soil conditions, the capacity of a pile group can be significantly smaller and the settlement of the group larger than the capacity and settlement of a single pile under the same average load per pile in the group.

This effect is more significant for pile types such as drilled piers, where the opening of the large diameter pier shaft reduces the effective stresses acting against the walls and base of the adjacent installed piers. The group effect for pile types such as driven friction piles or pressure grouted micropiles is much less significant, even beneficial, due to the increase in the effective stress of the soil due to the soil displacement by the driven pile or the compaction of the soil due to the pressure grouting.

Model and full scale load testing have verified what has been referred to as a "knot effect" (Lizzi, 1982; ASCE 1987) where a positive group effect is achieved in the loading of the soil-pile system. This effect is more prominent in granular soils than in cohesive soils.

For driven piles, AASHTO section 4.5.6.4 recommends no individual pile capacity reduction for group considerations with the exception of friction piles in cohesive soils, where an efficiency factor of 0.70 should be applied for piles with a center to center spacing of less than 3.0 times the pile diameter. For pressure grouted micropiles (Type B, C, and D) with a typical grouted diameter of 200 mm and typical minimum center to center spacing in the range of 0.75 to 1 meter, it is unnecessary to consider a group reduction effect under this criteria.

For gravity grouted (Type A) piles, the soil type and installation method should be examined for impact to the effective stress of the soil surrounding the piles and the impact to the capacity of the pile group. A method similar to that used for drilled piers may be applicable where the pile group is treated as one pile, with the perimeter and base area of the group establishing the pile dimension.

5.E MICROPILE STRUCTURAL DESIGN

In the structural design of micropiles, reference must be made to local construction regulations or building codes. The special considerations of micropile design may not always be specifically or adequately addressed in these regulations and codes. In that event, sensible interpretation or extrapolation is essential by all parties, backed up by appropriate field testing. In this section, design of the various sections of a composite reinforced micropile are examined in accordance with the 1996 AASHTO Standard Specifications for Highway Bridges, Sixteenth Edition. Both SLD and LFD methods are presented. See Section 5.C for explanation.

The micropile design examined in this section is represented in Figure 5-2 and consists of an upper length reinforced with a permanent steel casing with a center reinforcing bar and a lower pressure grouted bond length reinforced with a center reinforcing bar.

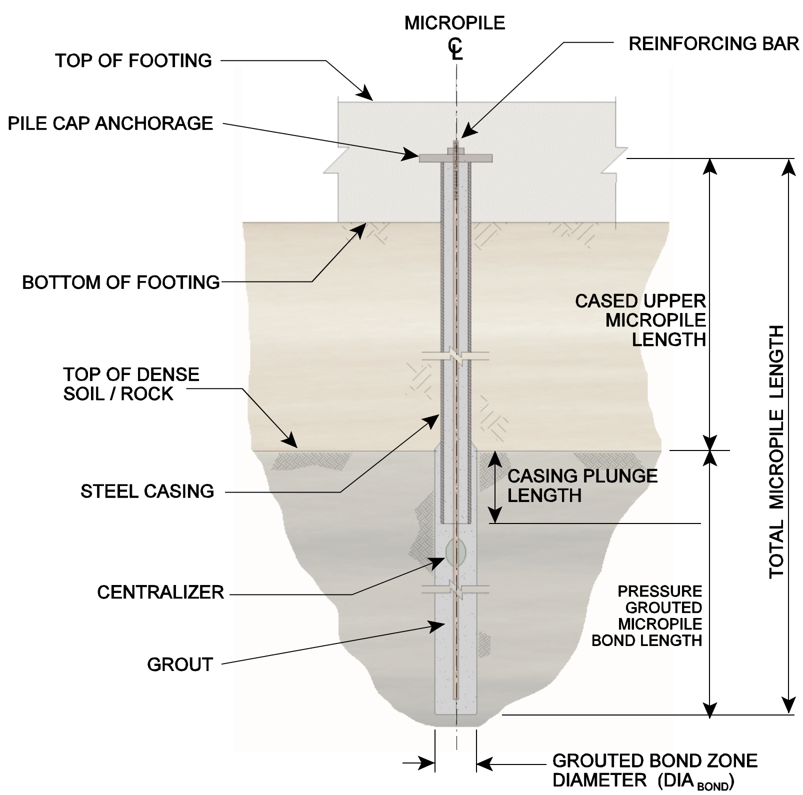


Figure 5 - 2. Detail of a Composite Reinforced Micropile

5.E.1 Notation

C_c = column slenderness ratio (SLD and LFD)

DL = Design Load (SLD)

E_{steel} = modulus of elasticity of steel (SLD and LFD)

FS = factor-of-safety (SLD)

F_a = allowable axial unit stress (SLD)

F_a = nominal axial steel stress (LFD)

$f'_{c\text{-}grout}$ = compression strength of grout (SLD and LFD)

F_y = specified minimum yield point of steel (SLD and LFD)

K = effective length factor (see AASHTO for a table of values) (SLD and LFD)

L = actual unbraced length (SLD and LFD)

$P_{c\text{ - }allowable}$ = allowable structural axial compression load (SLD)

$P_{t\text{ - }allowable}$ = allowable structural axial tension load (SLD)

$P_{c\text{ -}nominal}$ = nominal structural compression strength (LFD)

$P_{t\text{ -}nominal}$ = nominal structural tension strength (LFD)

$P_{c\text{ -}design}$ = design structural compression strength (LFD)

$P_{t\text{ -}design}$ = design structural tension strength (LFD)

r = governing radius of gyration (SLD and LFD)

φ = strength reduction factor (LFD)

$\alpha_{\text{bond nominal strength}}$ = geotechnical unit grout-to-ground bond strength (SLD and LFD)

$P_{G\text{-}allowable}$ = allowable geotechnical bond axial load (SLD)

$P_{\text{G-design strength}}$ = geotechnical bond axial design strength (LFD)

$P_{\text{transfer allowable}}$ = cased bond length (plunge zone) allowable transfer load (SLD)

$P_{\text{transfer design}}$ = cased bond length (plunge zone) design strength (LFD)

5.E.2 Pile Cased Length Structural Capacity

The tension and compression allowable loads (SLD) and design strength (LFD) for the cased upper portion of the pile can be determined with the equations included in the following two subsections. Since it is common for the upper section of a pile to be located in a weak upper soil, consideration of a laterally unsupported length may be included in determination of the compression capacity. See Section 5.F.5 for discussion on lateral stability considerations. See Section 5.E.7 regarding compatibility of strain between the grout and various strengths of steel used for the bar and casing reinforcement.

An alternative method for computing allowable loads (SLD) and design strengths (LFD) utlizing the transformed section of the pile could be used. It would, however, require careful consideration and documentation of the allowable (SLD) and ultimate (LFD) strains for each of the component materials.

Most piles that have soil surrounding the pile have no effective unsupported length and thus no reduction for buckling except for piles extending above ground, piles subject to scour, piles through mines/caves and piles through soil that may liquefy. The allowable loads formulae for piles with unsupported lengths are presented in Section 5.F.5.

5.E.2.1 Pile Cased Length (Service Load Design)

For strain compatibility between casing and bar (see section 5.E.7), use the following for steel yield stress:

$$F_{y-steel} = \text{the minimum of} \quad F_{y-bar} \;\; \& \;\; F_{y-casing}$$

Tension - Allowable load

$$P_{t-allowable} = 0.55\, F_{y-steel} \times (\,Area_{bar} + Area_{casing}\,)$$

(Note: 0.55 is per AASHTO Table 10.32.1A)

Compression - Allowable load

$$F_a = \frac{F_{y-steel}}{FS} \;......\; FS = 2.12$$

Allowable load –

$$P_{c-allowable} = 0.40\,f'_{c-grout}\ Area_{grout} + 0.47\,F_{y-steel}\,[\,Area_{bar} + Area_{casing}\,]$$

(Note: 0.40 is per AASHTO 8.15.2.1.1; 0.47 is per AASHTO, i.e. 1 ÷ 2.12 = 0.47)

5.E.2.2 Pile Cased Length (Load Factor Design)

For strain compatibility considerations between casing and bar (see section 5.E.7), use the following for steel yield stress: $F_{y-steel}$ = the minimum of F_{y-bar} and $F_{y-casing}$

Tension - Design strength

$$P_{t-nominal} = F_{y-steel}\,[\,Area_{bar} + Area_{casing}\,]$$

$$\text{Use } \varphi_t = 0.90 \;...\; \text{and with } P_{t-design} = \varphi_t \times P_{t-nominal}$$

$$P_{t-design} = 0.90 \times F_{y-steel}\,[\,Area_{bar} + Area_{casing}\,]$$

Compression - Design strength

$$P_{c-nominal} = 0.85\,f'_{c-grout}\,Area_{grout} + F_{y-steel}\left[Area_{bar} + Area_{casing}\right]$$

$$\text{Use } \varphi_c = 0.85 \quad \text{and with } P_{c-design} = \varphi_c \times P_{c-nominal}$$

$$P_{c-design} = 0.85 \times 0.85\,f'_{c-grout}\,Area_{grout} + F_{y-steel}\left[Area_{bar} + Area_{casing}\right]$$

5.E.3 Pile Uncased Length

The tension and compression allowable loads (SLD) and design strengths (LFD) for the pile uncased bond length are shown in the following two subsections. Since the lower uncased portion of the bond length is the weakest structurally (typically), an allowance is made for the geotechnical grout-to-ground capacity developed along the upper cased portion of the bond length (plunge length). This capacity ($P_{transfer\ allowable}$ for SLD and $P_{transfer\ design}$ for LFD) adds to the structural capacity of the uncased pile for resisting the design load (SLD) or the required strength (LFD). Another way to view $P_{transfer}$ is that it is the grout-to-ground bond capacity developed along the plunge length which can be used to reduce the pile load for design of the uncased length of the pile. In the design process the $P_{transfer}$ value is typically estimated and later verified. $P_{transfer}$ may be conservatively ignored and does not apply to end bearing piles. It is based on $\alpha_{bond\ nominal\ strength}$ over the plunge zone length (length of casing that is plunged into the bond length). See section 5.E.6 for further discussion about the plunge length and $P_{transfer}$ capacity. The compression φ_c value for LFD ($\varphi_c = 0.75$) was selected to provide a similar factor-of-safety to the SLD compression value. Higher values of φ_c (e.g. 0.85) are justified since the uncased length is composed mostly of a steel reinforcing bar surrounded by grout and fully supported by the ground.

See section 5.E.7 regarding strain compatibility between the pile components. This may cause the use of less than actual material design strength values.

5.E.3.1 Pile Uncased Length (Service Load Design)

Cased bond length (plunge length) allowable load $= P_{transfer\ allowable}$

$$P_{transfer\ allowable} = \left[\frac{\alpha_{bond\ nominal\ strength}}{FS}\right] \times 3.14 \times DIA_{bond} \times (Plunge\ length)$$

The plunge length is typically assumed and later verified. See section 5.E.6 for further discussion.

Tension - Allowable load

$$P_{t\text{-allowable}} = 0.55\ F_{y\text{-bar}}\ \ Area_{bar} + P_{transfer\ allowable}$$

(Note: 0.55 is per AASHTO Table 10.32.1A)

Compression - Allowable load

$$P_{c\text{-allowable}} = 0.40\ f'_{c\text{-grout}}\ \ Area_{grout}\ +\ 0.47\ F_{y\text{-bar}}\ \ Area_{bar} + P_{transfer\ allowable}$$

(Note: 0.40 is per AASHTO 8.15.2.1.1; 0.47 per AASHTO, i.e 1 ÷ 2.12 = 0.47)

5.E.3.2 Pile Uncased Length (Load Factor Design)

Cased bond length (plunge length) design strength = $P_{transfer\ design}$

$$P_{transfer\ design} = \varphi_G\ (\alpha_{bond\ nominal\ strength}) \times 3.14 \times DIA_{bond} \times (\text{plunge length})$$

The plunge length is typically assumed and later verified. See section 5.E.6 for further discussion. In the above equation, φ_G = 0.60 for typical designs for non seismic load groups or calibrated to SLD as shown in 5.C, or φ_G = 1.0 for seismic load groups

Tension - Design strength

$$P_{t\text{-design}} = [0.90 \times F_{y\text{-bar}} \times Area_{bar}] + P_{transfer\ design}$$

(Note: 0.90 is per AISC LRFD, D1-1)

Compression - Design strength

$$P_{c\text{-design}} = (0.75)\ (0.85\ f'_{c\text{-grout}} + F_{y\text{-bar}}\, Area_{bar}) + P_{transfer\ design}$$

Note: The design strength formulas for the pile uncased length are expressed as shown since the structural φ values (0.9 for tension and 0.75 for compression) are different from the φ_G values used for $P_{transfer\ design}$. The φ_G = 1.0 for seismic load groups is only allowed for the geotechnical $\alpha_{bond\ nominal\ strength}$ values.

5.E.4 Recommended Safety Factors and Test Loads for Field Verification and Proof Tests

Geotechnical

The verification test load and the proof test load are the maximum test loads computed from the following criteria considering both non-seismic and seismic load groups.

SLD – Use only for non-seismic load groups

Verification Test Load = 2.5 × DL = Verification Test FS × DL

Proof Test Load = 1.67 × DL = Proof Test FS × DL

DL is the *unfactored* controlling pile load.

LFD – Use for both non-seismic and seismic load groups

Verification Test Load = Required Nominal Strength = (DL × LF) ÷ φ_G = 2.5 × DL
where DL × LF is the *factored* controlling pile load.

Use φ_G = 0.60 for typical designs for non-seismic load groups or calibrated to SLD as shown in Section 5.C.

Use φ_G = 1.0 for seismic load groups

Proof Test Load = 1.67 ÷ 2.5 × [Verification Load or Nominal Strength]
or Proof Test Load = DL × LF

See Figure 5-3 and worked example problem No. 1 for application of the above.

The verification test is primarily intended to confirm that the selected bond length for the pile and the Contractor's installation equipment, methods and procedures are capable of producing the required grout-to-ground bond nominal strength. The verification test pile is usually a sacrificial pile tested prior to the installation of production piles. The proof tests are intended to confirm adequate and consistent Contractor pile installation procedures for the production piles and to confirm that the production piles will carry the required design service loads without excessive, long-term deflection. Proof tests are typically performed on permanent production piles. Both tests are required to assure that all permanent production piles (including the untested ones) provide the required allowable load for SLD and the required design strength for LFD.

The rationale for selection of the recommended micropile geotechnical safety factors is based on comparison to those currently in use for soil nails and permanent ground anchors, both of which have similarities to micropiles. A comparison of Industry Standards (U.S.) for maximum test loads applied to soil nails and ground anchors, with those recommended in this manual for micropiles, is as follows:

	Verification Test	Proof Test
Micropiles	2.5 × DL	1.67 × DL
Soil Nails	2.0 × DL	1.5 × DL
Ground Anchors		1.33 × DL

DL = Design Load = *unfactored* controlling pile design load

A graphical representation of the above comparison is also shown in Figure 5-3 that helps relate the tests to the SLD and LFD design criteria. It shows that the proof tests for micropiles, soil nails, and ground anchors are selected to provide assurance that the DL × LF criterion is met or exceeded. The verification test is used, in combination with the proof tests, to provide this assurance for micropiles and soil nails as only 5 percent of these elements are tested. The combination of verification and proof tests is meant to assure that the distribution of the strengths will not exceed φ_G and that all untested elements will meet or exceed DL × LF.

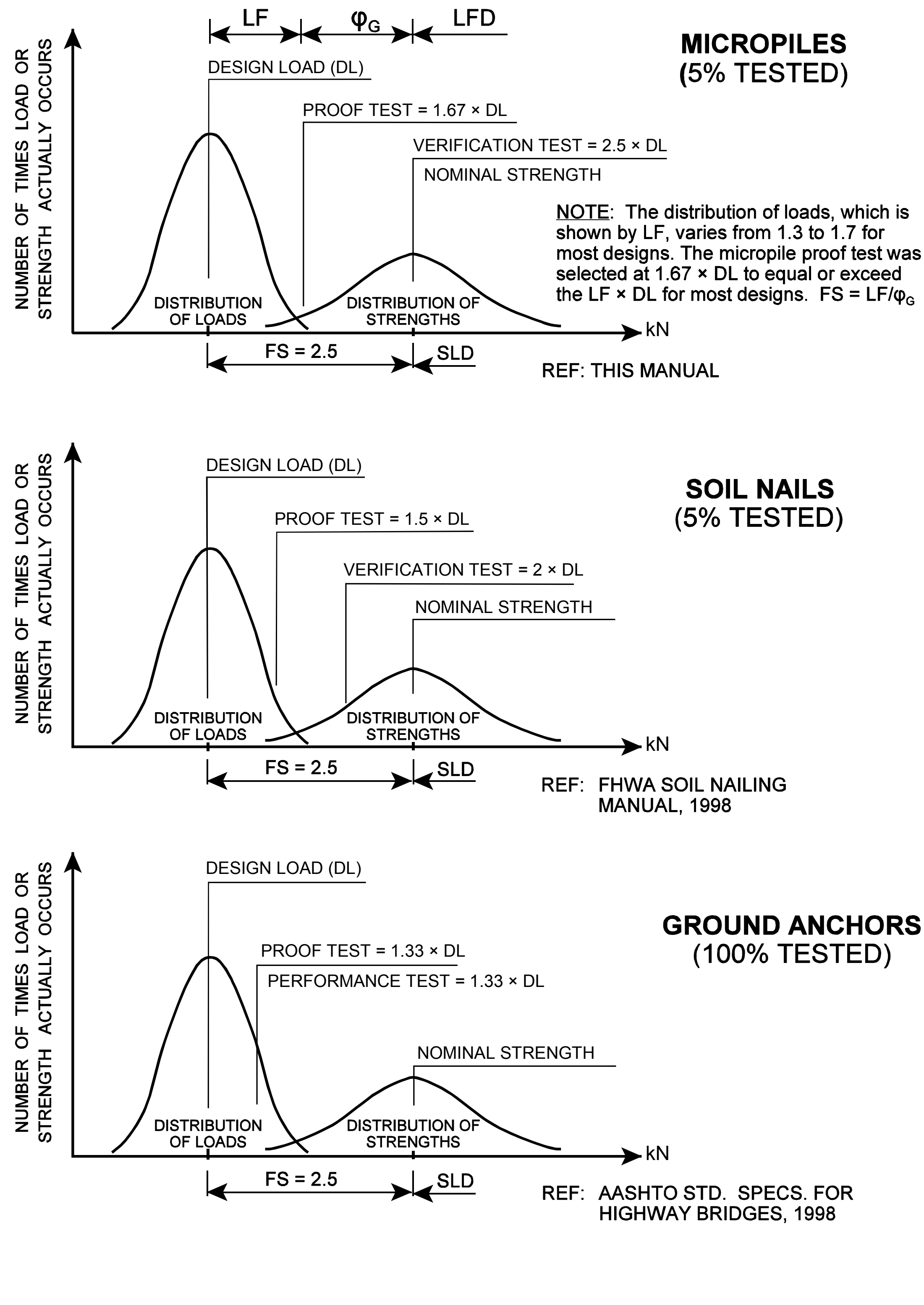


Figure 5 - 3. Comparison of Maximum Test loads for Micropiles, Soil Nails, and Ground Anchors

With permanent ground anchors, every single production anchor (i.e. 100 percent) is proof tested, therefore a lower safety factor is justified. For both soil nails and micropiles, typically only up to 5 percent of the production piles / nails are proof tested, therefore higher test loads are justified in comparison to ground anchors. Soil nails are typically of lower capacity and have greater redundancy than micropiles, especially compared to micropiles used for structural foundation designs as set forth in this manual. Thus, higher test loadings are prudent for micropiles, compared to soil nails.

Recall earlier in Section 5.D.3.1 the recommendation that a geotechnical safety factor of 2.5 (for both soil and rock) be applied to the estimated "nominal" (ultimate) grout-to-ground bond strength, to obtain the "allowable" (SLD) geotechnical grout-to-ground bond value to use for design. Section 5.C shows how to calibrate φ_G for LFD designs in order to match a SLD design with FS = 2.5.

Therefore, the recommended micropile verification test loading to 2.5 × DL is to verify that the micropile's geotechnical capacity is at least equal to the nominal grout-to-ground strength ($\alpha_{\text{bond nominal strength}}$), estimated for use in the design. The recommended proof test loading to 1.67 × DL is to provide some assurance that the geotechnical capacity of <u>all</u> the production micropiles, including the 95 percent which are not tested, is at least equal to LF × DL.

Structural

Since these tests occur under controlled conditions and have a short load duration, the design of the pile against structural failure for the verification and proof test loads may utilize a smaller factor-of-safety against steel yield in tension, steel yield or buckling in compression, or grout crushing in compression, versus the structural safety factors used in the design of the production piles. This manual recommends a structural FS = 1.25 against these structural modes of failure for the field load tests. Put another way, the micropile structural elements should not be stressed to greater than 80 percent (1/1.25) of the above structural capacities during application of the maximum test loads.

For some designs, the verification test pile may require larger pile casing and rebar than the production piles. The resulting stiffer pile can adequately confirm the grout-to-ground bond strength for the permanent production piles, but may not provide representative structural deflection behavior for the smaller permanent production piles. When this is the case, the proof tested production piles (at the 1.0 × Design Load test load increment) will have to be relied on to provide the representative structural deflection behavior of the in-service production piles. The verification tested pile would only be used to verify the grout-to-ground bond nominal strength.

Guideline criteria for the number of verification and proof tests to perform, along with suggested test acceptance criteria, are presented in Chapter 7 - "Load Testing" and in Appendix A, "Guide Construction Specifications".

This manual recommends a SLD approach for the structural design of the pile for verification and proof field test loads with the structural FS = 1.25 as previously discussed. An alternative method utilizing the transformed section of the pile could be used. It would, however, require careful consideration and documentation of the allowable strains for each of the component materials. The allowable load formulas with structural FS = 1.25 are developed next.

5.E.4.1 Pile Cased Length

$$F_{y\ steel} = \text{the minimum of } F_{y\text{-}bar} \text{ and } F_{y\text{-}casing}$$

Tension - Allowable Field Test Load

$$P_{t\text{-}allowable} = 0.80\ F_{y\ steel}\ (\text{Area}_{bar} + \text{Area}_{casing})$$

Compression - Allowable Field Test Load

$$F_a = \frac{F_{y-steel}}{FS} \quad . \; . \; . \quad FS = 1.25$$

5.E.4.2 Pile Uncased Length

$$P_{\text{transfer allowable}} = \left[\frac{\alpha_{\text{bond nominal strength}}}{FS} \right] \times 3.14 \times DIA_{\text{bond}} \times (\text{Plunge length})$$

FS = 1.25 for field test loads

Tension - Allowable Field Test Load

$$P_{\text{t-allowable}} = 0.80\, F_{\text{y-bar}}\, \text{Area}_{\text{bar}} + P_{\text{transfer allowable}}$$

Compression - Allowable Field Test Load

$$P_{\text{c-allowable}} = 0.68\, f'_{\text{c-grout}}\, \text{Area}_{\text{grout}} + 0.80\, F_{\text{y-bar}}\, \text{Area}_{\text{bar}} + P_{\text{transfer allowable}}$$

5.E.5 Grout to Steel Bond Capacity

The bond between the cement grout and the reinforcing steel allows the composite action of the pile, and is the mechanism for transfer of the pile load from the reinforcing steel to the ground. Typical ultimate bond values range from 1.0 to 1.75 MPa for smooth bars and pipe, and 2.0 to 3.5 MPa for deformed bars (ACI 318).

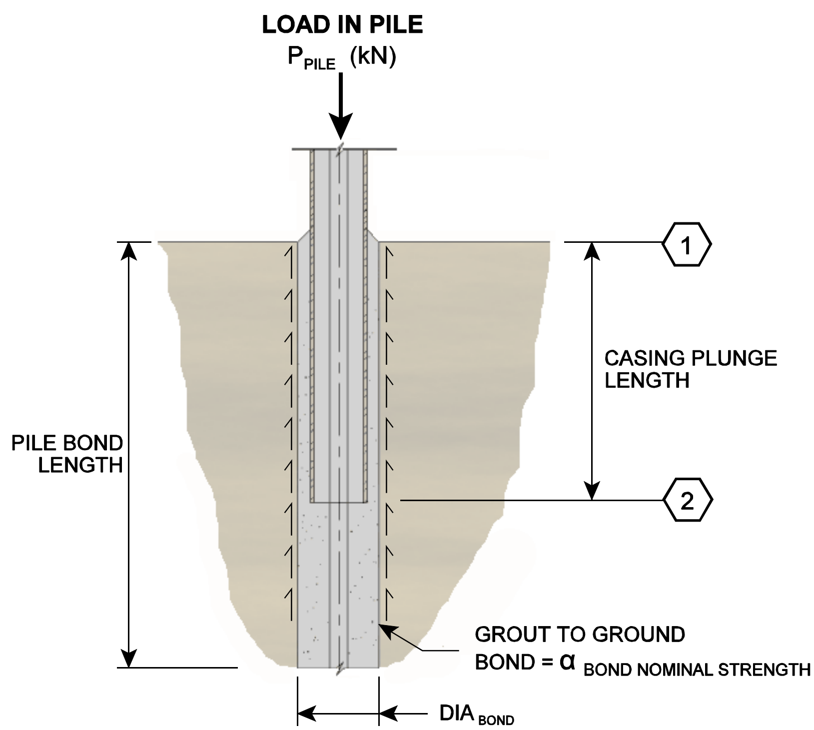
In the majority of cases, grout-to-steel bond does not govern the pile design. The structural or geotechnical pile capacity typically governs.

As is the case with any reinforcement, the surface condition can affect the attainable bond. A film of rust may be beneficial, but the presence of loose debris or lubricant or paint is not desirable. Normal methods for the handling and storage of reinforcing bars applies to micropile construction. For the permanent casing that is also used to drill the hole, cleaning of the casing surface can occur during drilling, particularly in granular soils.

5.E.6 Design of Plunge Length

As shown in Figure 5-4, a typical procedure for constructing a composite reinforced micropile is to insert the pile casing into the top of the grouted bond length. This detail accommodates the transition between the upper cased section and the uncased portion of the bond length. It also allows transfer of a portion of the pile load to the ground, reducing the load the uncased portion of the pile must support, which is typically the weakest structural portion of the pile.

This "transfer" load ($P_{transfer}$) occurring through the plunge length of the casing can be accounted for as shown in the pile structural calculations included in section 5.E.3 and as detailed in Figure 5-4. The value for this load is based on the unit grout-to-ground bond (from Table 5-2) acting uniformly over the casing plunge length.



TRANSFER LOAD:

$P_{TRANSFER}$ (kN) = ($\alpha_{BOND\ NOMINAL\ STRENGTH}$) × 3.14 × DIA_{bond} × (PLUNGE LENGTH)

LOAD CARRIED BY PILE @ DEPTH 1 = P_{PILE}

LOAD CARRIED BY PILE @ DEPTH 2 = $P_{PILE} - P_{TRANSFER}$

Figure 5 - 4. Detail of Load Transfer through the Casing Plunge Length.

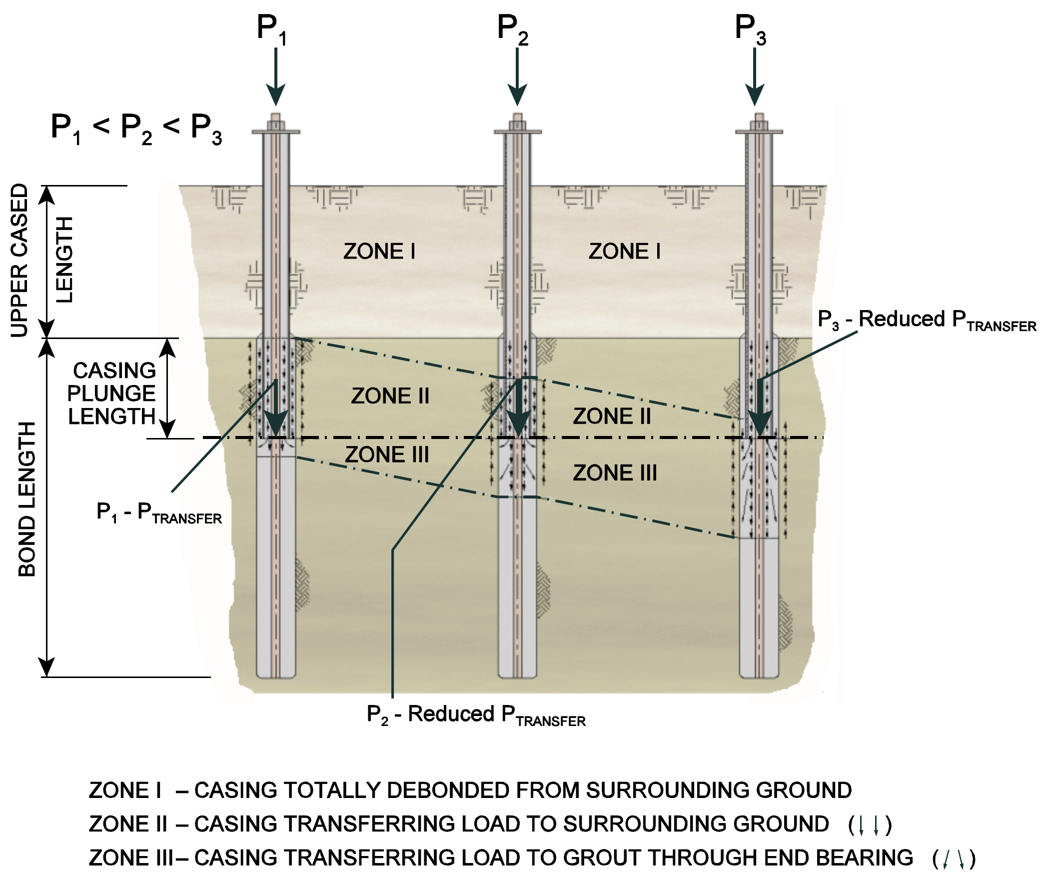


Figure 5 - 5. Change in Load Transfer through Casing Plunge Length as Load Increases (Bruce and Gemme, 1992).

Consideration must also be given to a reduction in the transfer load due to disturbance of the bond between the casing and the grout and between the grout and the ground. As shown in Figure 5-5, as the pile is subject to higher loading, the casing can debond from the grout and the ground, effectively reducing the plunge length and reducing the available transfer load. This reduction has been confirmed through field load tests on highly loaded piles. $P_{transfer}$ can be a significant component in the design of the pile uncased length and therefore its value must be carefully evaluated with the micropile specialty contractor's experience and confirmed by field tests.

5.E.7 Strain Compatibility Between Structural Components

Strain compatibility between the structural components of a composite reinforced micropile should be considered in the pile structural design, particularly when the use of high strength reinforcing bars is included. Reinforcing bars are available with yield stress up to 828 MPa (1,035 MPa ultimate strength). The strain associated with reaching 85 percent of the bar yield stress in compression at the pile ultimate strength (Load Factor Design) may exceed the strain that the grout can sustain without fracturing or crushing. Limiting the value of the yield stress used in the design may be necessary to avoid grout failure. AASHTO section 8.16.2.3 limits the maximum usable concrete compression strain to 0.003, which corresponds to a maximum steel stress of 600 MPa.

Strain compatibility between the grout and casing reinforcement is typically less of a concern due to the lower yield strength of the casing steel (typically 551 MPa max), and the confined state of the grout inside the casing section allows the grout to support higher strain values without fracturing.

Strain compatibility between a high strength bar and the casing needs to be addressed. The area of the casing section is typically much greater than that of a high strength bar. This results in the distribution of a majority of the pile load to the casing. The strain associated with reaching 85 percent of the bar yield stress at the pile ultimate strength (Load Factor Design) may result in yielding of the casing, which can reduce the casing threaded joint integrity.

In summary, strain compatibility requirements dictate the use of the smaller yield stress of the reinforcing bar and casing in the calculations, and in compression this value must not exceed 600 MPa to address the strain compatibility of the grout for the cased portion of the pile. For the uncased portion of the pile the reinforcing bar yield stress used in the calculations in compression must not exceed 600 MPa.

The strain compatibility approach used in this manual and described above is intended to be an easy to use practical approach for normal designs. A higher tension ultimate strength using a higher bar yield stress (compared to the casing) may be utilized if the strains due to the working loads are shown to not cause permanent deformations in the threaded casing joints. A higher compression ultimate capacity could be utilized for the cased portion if documentation was provided to show that the grout within the casing can sustain strains larger than 0.003.

Other approaches are acceptable with proper documentation.

5.E.8 Reinforcing Bar and Casing Connections

As discussed in chapter 3, reinforcing bar and casing are commonly installed in coupled sections. For installation of piles in very low overhead clearance conditions, the lengths of these sections can be 1 meter or shorter. Common reinforcing bar couplers can provide a minimum axial capacity of 125 percent of the bar yield strength, which is adequate for micropile applications.

A common method for coupling casing lengths is to machine a thread into the wall of the casing at the section ends. If the joint is properly designed, it can provide a compressive capacity almost equivalent to the strength of the full casing section. Having the casing filled with grout is important to the strength of this type of joint. The grout provides support, preventing the male half of the joint from deforming inward.

Tension and bending have the biggest impact on the strength of the joint due to the reduced thickness at the threaded area. The threading may reduce the casing strength in bending and tension. The casing joint detail used by a specialty contractor may be a proprietary item. Testing data that verifies adequacy of the joint detail may be a necessary component of the contractor's design submittal.

Bending in a pile is due to lateral loading and the resulting lateral displacement. The bending moment in the pile is commonly dissipated within 3 to 5 meters below the bottom of the pile cap (see section 5.F.4). The reduced capacity of the joint may be accommodated by locating the first joint at an adequate depth where the pile moment is reduced to an acceptable value.

5.E.9 Pile to Footing Connection

Unless a single micropile is used, a pile cap (footing) is necessary to spread the structure loads and any overturning moments to all the micropiles in the group. Reinforced concrete pile caps are designed in accordance with the AASHTO Standard Specifications for Highway Bridges, Sixteenth Edition, or ACI-318. The design of reinforced concrete pile caps is not addressed in this manual.

The connection between the top of the micropile and the reinforced concrete pile cap can vary depending on the required capacity of the connection, the type of pile reinforcement, and the details of the pile cap. Seven examples of the pile-to-footing connections are shown in Figures 5-6 through 5-12. Figures 5-6 through 5-9 show typical connections for piles that can have both tension and compression loads depending on load case. Figures 5-10 through 5-12 show simple connections for piles that are only in compression.

Figure 5-6 shows a composite reinforced pile connected to a new footing. The footing tension and compression load is transferred to the pile through the top plate. The stiffener plates provide bending strength to the plate, plus provide additional weld length for transferring the load from the bearing plate to the pile casing. The stiffener plates can be eliminated if the support of the top plate and additional weld length are not required. Additional considerations for this connection detail include the following:

- The portion of the tension load carried by the reinforcing bar can be transferred to the top plate through the nut, reducing the plate-to-casing weld requirement.
- The bond between the pile casing and the footing concrete can be utilized, reducing the load capacity required for the top plate and top plate to casing weld.
- A portion of the compression load can be transferred from the top plate to the casing through bearing, reducing the weld capacity requirement. This requires a higher level of quality for the fabrication of the bearing surface between the casing and the plate.

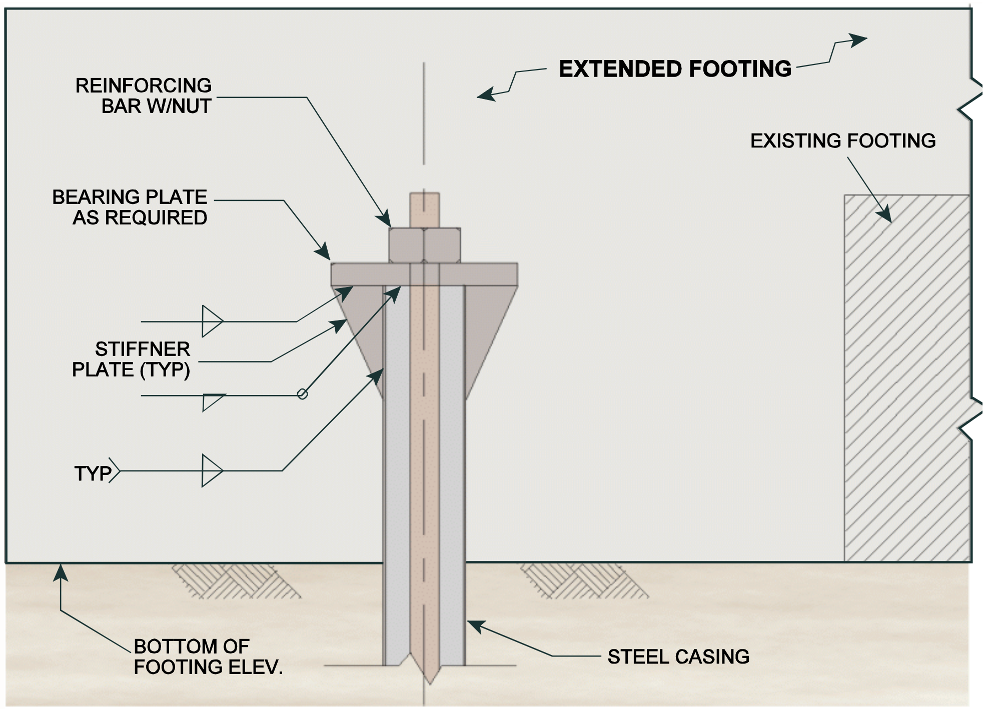


Figure 5 - 6. Pile to Footing Connection Detail.

Sample problem No. 1 at the end of this chapter includes the design of a pile connection similar to that shown in Figure 5-6. The additional considerations listed above are not included in the sample problem calculations.

Figure 5-7 shows a composite reinforced pile connected to an existing footing. The pile is installed through an oversized hole cored through the existing footing or slab. After the pile is installed, the core hole is cleaned and filled with non-shrink cement grout. Steel rings are welded to the top section of the casing prior to pile installation. These rings transfer the pile load from the casing to the non-shrink grout. Adequate spacing must be used between the rings to avoid combining bearing stresses in the concrete and grout. The total capacity of the connection is controlled by the following:

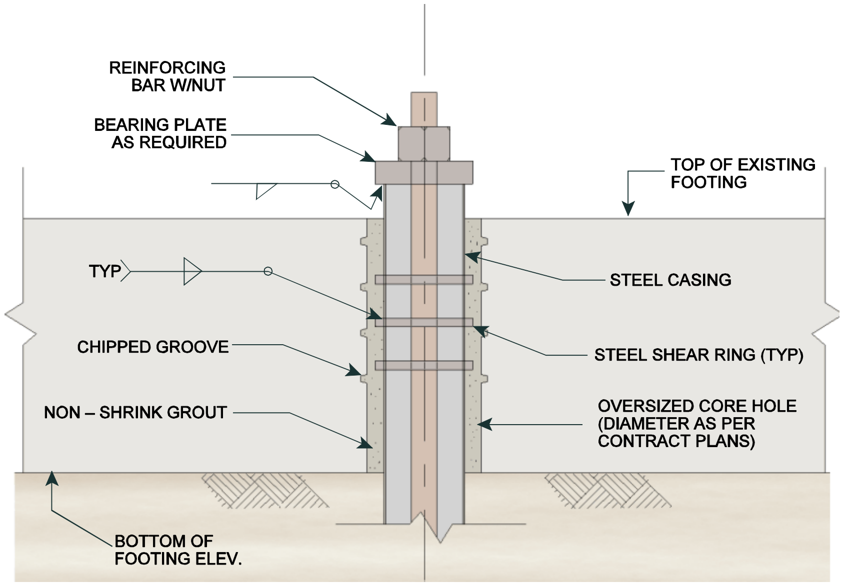


Figure 5 - 7. Pile to Footing Connection Detail.

- The sum of the bearing strength of the rings.
- The capacity of the load transfer across the interface between the non-shrink grout and the existing concrete.
- The shear capacity of the existing concrete.

Grooves may be chipped into sides of the core hole (typical dimension = 20 mm deep × 32 mm wide) to increase the capacity of the grout to existing concrete load transfer. Also, vertical reinforcing bars may be drilled and epoxied into the existing concrete around the exterior of the connection to increase the punching shear capacity.

An ultimate capacity in excess of 2,660 kN was demonstrated on a project through load testing two connections which were similar to the connection shown in Figure 5-7. The connection was installed in a 0.25-meter-diameter core hole through a 0.6-meter-thick slab. Completion of pre-production load testing for this connection type may be necessary to verify the adequate structural capacity.

For thick existing footings, the shear rings and grooves in Figure 5-7 may be eliminated. Load tests on the connections are appropriate to verify the casing to grout bond and grout to existing concrete bond for the proposed materials and methods.

Figure 5-8 shows a composite reinforced pile connected to a new footing. The footing compression load is transferred to the pile through bearing on the pile top and reinforcing bar plate, and the tension load is transferred through bearing on the reinforcing bar plate. A portion of the load transferred from the footing to the pile may be attributed to the bond between the pile casing and the footing concrete.

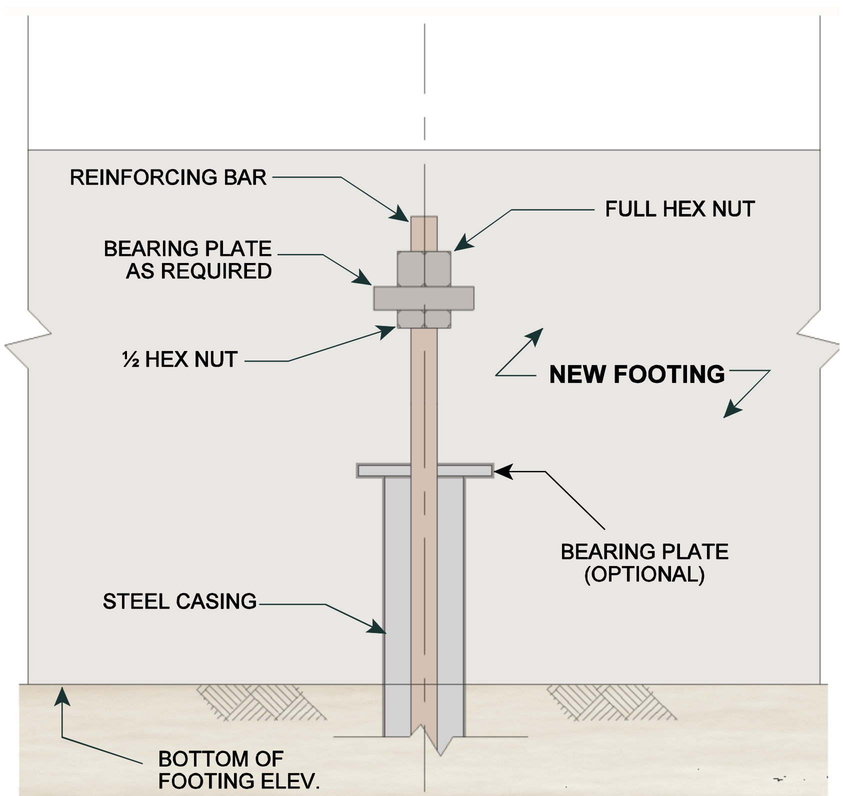


Figure 5 - 8. Pile to Footing Connection Detail.

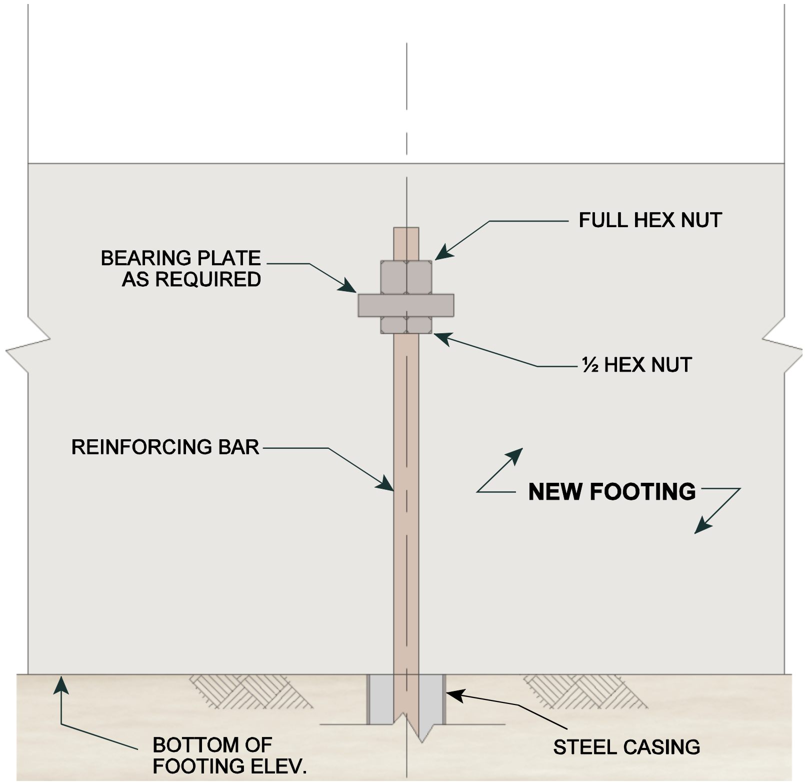


Figure 5 - 9. Pile to Footing Connection Detail.

Figure 5-9 shows a bar-reinforced pile cast into a new footing. The footing's compression and tension load is transferred to the pile through bearing on the bar plate, and through bond between the footing concrete and the reinforcing bar. Additional compression load can be transferred through footing compression on the top of the pile grout column. Competency of the construction joint between the pile grout and footing concrete is an important quality consideration for this pile type.

Figure 5-10 shows a typical compression connection for a moderately loaded micropile where concrete bearing stress on the pile top is within AASHTO/ACI limits.

Figure 5-11 shows a typical compression connection for a heavily loaded micropile where a bearing plate is required for concrete bearing stress on the pile top to be within AASHTO/ACI limits.

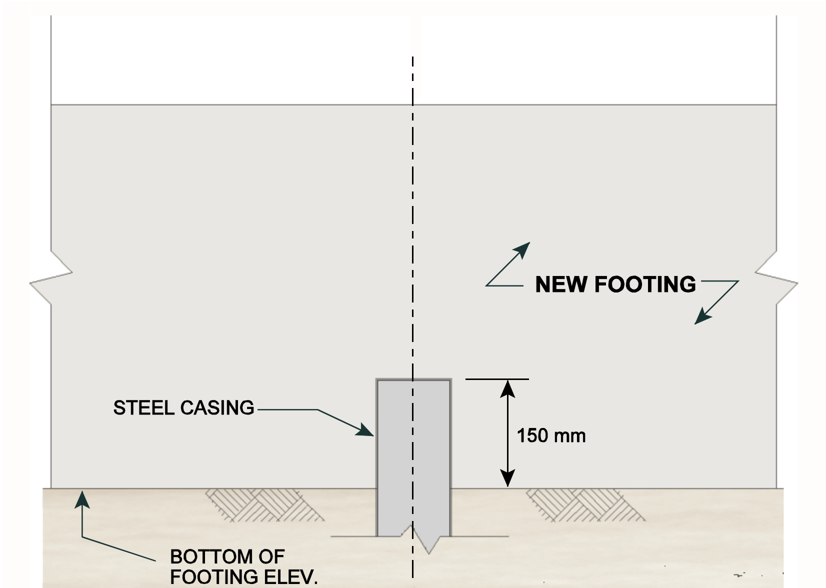


Figure 5 - 10. Pile to New Footing Connection Detail – Simple Compression, Moderate Load.

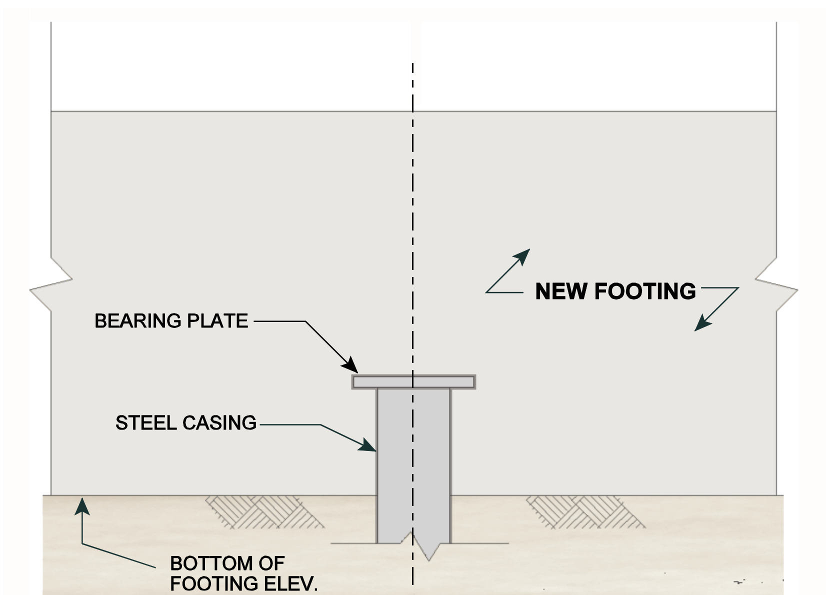


Figure 5 - 11. Pile to New Footing Connection Detail – Simple Compression, High Load.

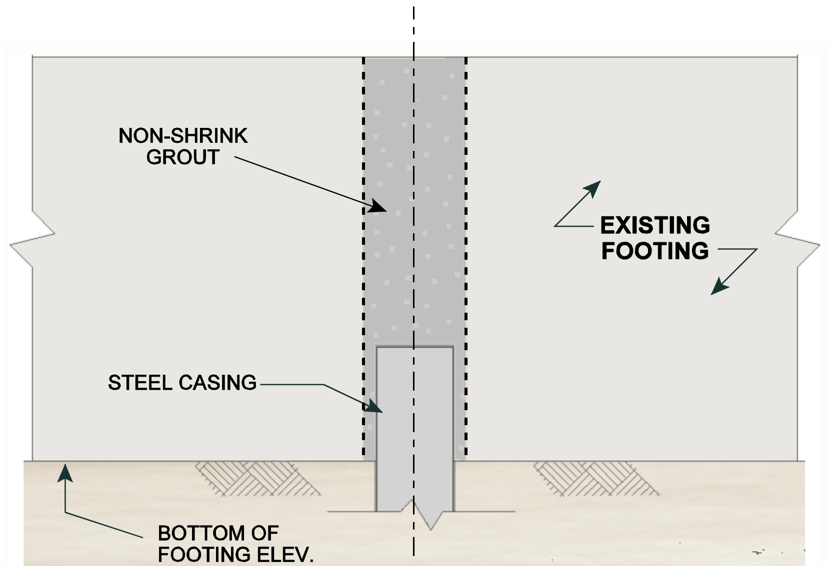


Figure 5 - 12. Pile to Existing Footing Connection Detail – Compression Only.

Figure 5-12 shows a compression connection for a pile through an existing footing. The bond of the non-shrink grout to the footing is a function of the proposed methods (cored hole, down-the-hole hammered hole, etc.) and materials (non-shrink grout, existing concrete, etc.). Connection load tests are appropriate to verify the design.

5.F ADDITIONAL GEOTECHNICAL / STRUCTURAL CONSIDERATIONS

5.F.1 Prediction of Anticipated Structural Axial Displacements

For many designs axial pile stiffness is not a concern. Earthquake foundation retrofit work requires, however, deflection compatibility between existing and new piles thereby requiring special attention to pile stiffness. New structure and earthquake retrofit designs often require determination of pile stiffness for inclusion into the overall structure model for earthquake analysis. When pile designs require stringent displacement criteria, such as the earthquake cases mentioned above, it may be necessary to predict pile stiffness and deflection limits during design and confirm through field load tests.

The design of a micropile can be controlled by stiffness requirements (allowable displacement) due to the small cross sectional area of the pile. Pile load testing is typically conducted to verify the required stiffness and load capacity. During the pile design phase, methods are available for prediction of the pile displacements under compression and tension loading, allowing initial estimation of the required pile stiffness.

One method is referred to as the "t-z" load transfer model, where t represents the interface shear stress and z the vertical displacement. This method, which was developed for predicting the displacement behavior of large diameter piles, involves modeling the pile as a series of finite elements, while the ground is represented as a series of distributed axial springs. Selection of proper design parameters and accurate prediction of pile behavior can be difficult given the effect of variable soil conditions and the effect on pile performance from installation methods. This method is not addressed further in this manual.

A more simplistic approach included in this manual is similar to the approach used for analyzing tieback anchor performance and consists of separately predicting the elastic (recoverable) and residual (non-recoverable) portion of the pile displacement.

The magnitude of the elastic displacement is dictated by the magnitude of the applied load and the length of the pile over which it acts. If a pile were acting purely in end bearing, then the length over which the pile acts elastically would be constant, and the deflection would always be proportional to the load. As the load is increased on a micropile, the bond to the soil in the upper portion of the grouted pile length is disturbed, and the location of soil bond resultant moves progressively downward. The elastic length of the pile increases with the downward movement of the soil bond resultant.

The magnitude of elastic and residual components of the displacement can be determined during load testing, with the residual movement being the zero (or alignment) load displacement reading after a load cycle, and the elastic movement being the difference between the peak load and zero load displacement readings.

For an anchor or micropile, the elastic displacement and elastic length can be approximated using the formula:

$$\Delta_{\text{Elastic}} = \frac{P\,L}{A\,E}$$

where

Δ_{Elastic} = elastic component of the total displacement (m)

P = applied load (kN)

L = elastic length (m)

AE = stiffness of section (Area (m^2) $\times$ Elastic Modulus (kPa))

For prediction of the anticipated displacement during the design phase, the pile stiffness must be determined, and values must be selected for the residual movement and elastic length. For anchors, determination of the stiffness value AE is relatively easy, with its single reinforcement and no contribution to the anchor stiffness from the grout. Determination of a micropile's stiffness value is more complex due to:

- The contribution of the grout to the pile's stiffness due to the pile acting in compression.
- The varying reinforcement used in some micropiles, with casing reinforcement in the upper portion of the pile and bar reinforcement in the lower portion.
- The stiffening effect on the grout when confined in the casing reinforcement.

The stiffness of the composite section can be determined using the formula:

$$E\,A_{\text{pile}} = \left[A_{\text{grout}} \times E_{\text{grout}}\right] + \left[A_{\text{steel}} \times E_{\text{steel}}\right]$$

Based on results from pile and material load testing, the use of E_{grout} = 31,000 MPa for grout confined in a cased length, and E_{grout} = 23,000 MPa for unconfined grout can provide reasonable results.

The length over which the pile will act elastically must be estimated based on pile installation methods, competency of the soils, and experience. A pile grouted full length in competent soils will have a short elastic length. For a pile such as shown in the example problem in this chapter, with a casing extending into a lower competent layer and the grouted length limited to that competent layer, the elastic length may extend to the bottom of the casing. If the bond length soils are less competent, the elastic length may extend below the cased length.

The magnitude of the residual movement must also be predicted based on pile installation methods, competency of the soils, and experience. The residual movement will increase with an increased applied load, and will increase with the increasing softness of the soil or decreasing geotechnical bond capacity. For a pile bonded into medium dense to dense soil with applied loads up to 1,300 kN, typical residual movement values may vary from 2 to 5 mm.

During load testing, this method can be used to examine pile performance by plotting development of the elastic length. The elastic length extending into the bottom half or third of the bond length may be an indication of pending geotechnical failure. Examination of the reasonableness of the results is also necessary. An elastic length extending beyond the bottom of the pile would indicate an inaccuracy in the stiffness value used.

5.F.2 Long Term Ground Creep Displacement

Long term displacement performance of micropiles and ground anchors depends upon the potential of the ground system to creep. Creep is a time dependent deformation of the soil structure under a constant or sustained loading. Theoretically, creep can develop in three basic components of the system: the grout, the steel, and the ground surrounding the bond length. Creep deformations of the grout and the steel material are considered to be insignificant. Creep deformations due to loss of bond between grout and steel may be greater, but are still insignificant. As the bond decreases and the load transfers deeper, the “creep” also includes elastic deflection of the pile. Fine-grained clayey soils may undergo large creep deformation that will result in significant time dependent anchor or pile displacement.

If micropiles are to be installed in creep sensitive cohesive soils, extended load testing similar to that specified for ground anchors (Post Tensioning Institute (PTI) - Recommendations for Prestressed Rock & Soil Anchors – 1996) can be performed to verify performance within acceptable limits. Test load hold duration may be extended to 100 minutes or in extreme cases 1,000 minutes or more, depending on the magnitude and type of the design loading and creep sensitivity of the soil. A maximum creep rate of 2 mm per log cycle of time is a common acceptance criteria (PTI, 1996). This criteria is included in the load testing acceptance criteria in the Appendix A guide specifications.

5.F.3 Settlement of Pile Groups

In addition to the components of axial displacement for a single pile described in the previous two subsections, arrangement of piles in a group can cause additional displacement due to the consolidation of the soil layer below the pile group. Where a single pile will transfer its load to the soil in the immediate vicinity of the pile, a pile group can distribute its load to the soil layer below the group. Consideration should be made for this group displacement when the soil below the group is cohesive in nature and subject to consolidation.

To compute pile group settlement design guidance is given in the Federal Highway Administration Design and Construction of Driven Pile Foundations Workshop Manual (Publication No. FHWA HI-97-013 -December 1996).

5.F.4 Lateral Load Capacity

The behavior of a laterally loaded micropile depends on the properties of the micropile such as diameter, depth, bending stiffness, fixity condition of the pile in the footing, and on the properties of the surrounding soils. The effects to the surrounding soil from pile installation should also be considered. These effects can include loosening of the soil due to pile drilling and densification of the soil due to grout placement. Reference is made to the following FHWA published documents and computer analysis program for discussion on the behavior and analysis of laterally loaded piles.

- Behavior of piles and pile groups under lateral load (FHWA/RD-85/106).
- Handbook on design of piles and drilled shafts under lateral load (FHWA-IP-84-11).
- COM624P - laterally loaded pile analysis program for the microcomputer (FHWA-SA-91-048).

Methods available to increase the lateral capacity provided by a micropile include:

- Installing the pile at an incline or batter.
- Installation of an oversized upper casing which increases the effective diameter of the pile, the lateral support provided by the soil, and the bending strength of the pile

Consideration must be made to the combined stresses in the micropile due to bending induced by the lateral displacement and axial loading. The ability of the pile section to support the combined stress must be checked, particularly at the casing joint locations.

The lateral stiffness and capacity of a micropile is limited due to the smaller diameter. Computer programs, such as COM624P mentioned above, are available to determine the lateral pile stiffness of a micropile, which is a complex relationship of pile deformation and the reaction of the surrounding soil, which usually is nonlinear. A linear approximation of this behavior is described in NAVFAC (1982). Also, for an illustration of the NAVFAC procedure, see the FHWA Publication No. FHWA-SA-97-010, Seismic Design of Bridges - Design Example No. 5.

The NAVFAC procedure will be demonstrated in the following computations for a 244.5 mm OD micropile.

OD = 244.5 mm; Wall thickness = 11.99 mm; ID = 244.5 - 2 x 11.99 = 220.52 mm

E_{casing} = 200,000 MPa

E_{grout} = 31,000 MPa

$$I_{casing} = \frac{\pi}{64}\ [(OD^4 - ID^4] = \frac{\pi}{64}\ [244.5^4 - 220.52^4] = 59{,}000{,}000\ \ mm^4$$

$$I_{grout} = \frac{\pi}{64}\ [ID^4] = \frac{\pi}{64}\ [220.52^4] = 116{,}000{,}000\ \ mm^4$$

$$EI = E_{casing}\, I_{casing} + E_{grout}\, I_{grout} = 15{,}396\ kN - m^2$$

Lateral load required to produce 6.35 mm (1/4") lateral displacement in a dense soil by NAVFAC is:

$$P = \left[\frac{\delta_p}{F_\delta}\right] \times \frac{EI}{\left[\ (EI/f)^{1/5}\ \right]^3}$$

Use f (modulus of subgrade reaction) = 17,600 kN/m^3 for dense soil above ground water (for this example).

δ_p = 6.35 mm (1/4")

$F_\delta = 2.6$ for $L = 3T$ $\qquad$ L = pile length and $T = \left[\dfrac{EI}{f} \right]^{1/5}$

$F_\delta = 2.25$ for $L > 5T$

$$T = \left[\frac{15{,}396}{17{,}600} \right]^{1/5} = 0.97 \text{ m}$$

For a pile length > 5T = 4.9 m use $F_\delta = 2.25$

$$P = \left[\frac{6.35/1000}{2.25} \right] \times \frac{15{,}396}{0.97^3} = 47.1 \text{ kN}$$

Displacement limits usually control the allowable load for the soil/pile interaction demonstrated by the above NAVFAC procedure. The above 6.35 mm (1/4") limit is used by many designers as the limit for non-seismic load groups. Higher displacement limits are usually used for seismic load groups, however the ultimate capacity of the pile must not be exceeded and the effects of the pile displacements to the overall structure must be evaluated.

For many designs batter piles provide sufficient lateral resistance for the lateral loads from the horizontal component of their axial load. Where batter piles are not sufficient the additional resistance from the soil/pile stiffness, as described above, is added to the batter pile resistance. Passive soil resistance against the footing is seldom used for lateral resistance since the lateral displacement required to mobilize passive resistance is large.

Table 5-3 shows pile stiffness values (Load in kN for 6.35 mm lateral displacement) for 3 different micropile sizes for 6 values of soil modulus (3 above ground water and 3 below ground water). Also shown for comparison are 3 different HP pile sections. These values can be prorated; for example the lateral load required to displace a 244.5 mm (w/ 11.99 mm wall) micropile 12.7 mm (½") in a dense soil above the ground water is 47.1 kN × 2 = 94.2 kN.

Table 5-3. P = Lateral Load for 6.35 mm (1/4 inch) Lateral Displacement in kN for Coarse Grained Soil (Pinned Pile Head Condition)

Soil f (kN/m^3)	Above Ground Water			Below Ground Water		
	Loose	Medium	Dense	Loose	Medium	Dense
	2,199	6,597	17,592	1,256	4,398	10,681
HP 10 x 42 EI = 17, 477 kN-m^2	14.2	27.5	49.5	10.2	21.6	36.7
HP 12 x 53 EI = 32,707 kN-m^2	18.3	35.3	63.6	13.1	27.7	47.2
HP 14 x 89 EI = 75, 235 kN-m^2	25.5	49.3	88.8	18.2	38.6	65.8
MP 139.7 mm, 9.17 mm wall, EI = 1,905 kN-m^2	5.9	11.3	20.4	4.2	8.9	15.1
MP 177.8 mm, 12.65 mm wall, EI = 5,237 kN-m^2	8.7	17.0	30.6	6.3	13.3	22.7
MP 244.5 mm, 11.99 mm wall, EI = 15,396 kN-m^2	13.5	26.1	47.1	9.6	20.5	34.9

HP = H-pile, MP = Micropile

$$P = \left[\frac{\delta_p}{F_\delta}\right] \times \frac{EI}{\left[(EI/f)^{1/5}\right]^3} = \left[\frac{6.35/1000}{2.25}\right] \times \frac{EI}{\left[(EI/f)^{1/5}\right]^3}$$

$F_\delta = 2.6$ for $L = 3T$ L = pile length and $T = \left[\frac{EI}{f}\right]^{1/5}$

$F_\delta = 2.25$ for $L > 5T$

Table 5-3 uses soil modulus (f) values consistent with those used in Figures 25 and 26 for steel H-piles in the AISC "Highway Structures Design Handbook," Volume I, Section 10. The following soil properties were used for the f values in Table 5-3. Refer to the AISC and NAVFAC references for more information about f values.

	γ_e (kN/m^3)	φ (degrees)	f (kN/m^3)
Coarse Grained Soil Above Groundwater			
Loose soil	14.9	28	2,199
Medium dense soil	17.3	30	6,597
Dense soil	17.3	36	17,592
Coarse Grained Soil Below Groundwater			
Loose soil	8.6	28	1,256
Medium dense soil	9.4	30	4,398
Dense soil	10.2	36	10,681

5.F.5 Lateral Stability (Buckling)

Mathematical models and experimental load testing have been applied in the examination of buckling of micropiles. Bjerrum (1957), Mascardi (1970, 1982), and Gouvenot (1975) concluded that buckling of micropiles is of concern only in soils with the poorest mechanical properties such as loose silts, peat, and soft unconsolidated clays (soils that have an elastic modulus of less than 0.5 MPa).

As part of the Caltrans pile load test program of various pile types at a deep Bay mud site in 1992, a specialty contractor installed several micropiles for load testing. The soil conditions consisted of approximately 30 meters of soft Bay mud (very soft clay) over dense sand. The piles, reinforced with 178-mm-outside-diameter casing, were tested in compression to loads exceeding 1,775 kN with no signs of buckling. Fore more information, see Caltrans (1993).

Project load testing programs will provide a conservative check of a micropiles buckling tendency. A production pile will typically perform under a fixed head condition with the pile top embedded in a concrete footing. A load test will typically be conducted under a free head condition with the pile top approximately 0.5 meter above ground.

Consideration of a pile's unsupported length can be addressed during the design phase through the inclusion of values for the effective length factor (K) and the unbraced pile length (L) in determination of the allowable or nominal (ultimate) compression of the upper pile length (sections 5.E.2.1 & 5.E.2.2). As mentioned above, most pile designs that have soil surrounding the pile will have KL = 0 and therefore no reduction for buckling. Piles that are extended above the ground or piles that are subject to scour must, for example, be checked for the buckling reduction.

For piles with an unsupported length, the following equations apply:

Service Load Design (SLD) Method

$$C_c = \left[\frac{2\pi^2 E_{steel}}{F_{y-steel}}\right]^{1/2}$$

$$\text{If} \quad \frac{KL}{r} = 0 \qquad F_a = \frac{F_{y-steel}}{FS} \qquad FS = 2.12$$

$$\text{If} \quad 0 < \frac{KL}{r} \leq C_c \qquad F_a = \frac{F_{y-steel}}{FS} \times \left[1 - \frac{(KL/r)^2 F_{y-steel}}{4\pi^2 E_{steel}}\right]$$

$$\text{If} \quad \frac{KL}{r} \geq C_c \qquad F_a = \frac{\pi^2 E_{steel}}{FS\left[KL/r\right]^2}$$

Allowable Load:

$$P_{c-allowable} = \left[0.40\, f'_{c-grout}\ \text{Area}_{grout} + \frac{F_{y-steel}}{FS}\left(\text{Area}_{bar} + \text{Area}_{casing}\right)\right] \times \frac{F_a}{\frac{F_{y-steel}}{FS}}$$

Load Factor Design (LFD) Method

For strain compatibility considerations between casing and bar (see Section 5.E.7), use the following for steel yield stress: $F_{y\text{-steel}}$ = the minimum of $F_{y\text{-bar}}$ and $F_{y\text{-casing}}$.

$$C_c = \left[\frac{2\,\pi^2\, E_{steel}}{F_{y-steel}} \right]^{1/2}$$

$$\text{If} \quad \frac{KL}{r} = 0 \qquad F_a = F_{y-steel}$$

$$\text{If} \quad 0 < \frac{K\,L}{r} \leq C_c \qquad F_a = F_{y-steel} \left[1 - \frac{(K\,L\,/\,r)^2\, F_{y-steel}}{4\,\pi^2\, E_{steel}} \right]$$

$$\text{If} \quad \frac{K\,L}{r} \geq C_c \qquad F_a = \frac{\pi^2\, E_{steel}}{(K\,L\,/\,r)^2}$$

Design strength

$$P_{c-nominal} = \left[0.85\, f'_{c-grout}\, \text{Area}_{\,grout} + F_{y-steel} \left(\text{Area}_{\,bar} + \text{Area}_{casing} \right) \right] \times \frac{F_a}{F_{y-steel}}$$

$$\text{Use } \varphi_c = 0.85 \quad \text{ and with } \quad P_{c-design} = \varphi_c \times P_{c-nominal}$$

$$P_{c-design} = 0.85 \times \left[0.85\, f'_{c-grout}\, \text{Area}_{\,grout} + F_{y-steel} \left(\text{Area}_{\,bar} + \text{Area}_{casing} \right) \right] \times \frac{F_a}{F_{y-steel}}$$

To illustrate the buckling reduction the piles for Sample Problem No. 1 will be evaluated for a predicted scour depth of 3 meters below the bottom of the footing.

Service Load Design (SLD) Method

Use K = 0.65 (fixed-fixed condition)

$L = 3$ m

$E_{steel} = 200{,}000$ MPa

$FS = 2.12$

$r_{casing} = 46$ mm

$F_{y\ casing} = 241$ MPa $\qquad$ $Area_{casing} = 3{,}224\ mm^2$

$F_{y\text{-}bar} = 520$ MPa $\qquad$ $Area_{bar} = 1{,}452\ mm^2$

$f'_{c\text{-}grout} = 34.5$ MPa $\qquad$ $Area_{grout} = 10{,}240\ mm^2$

$$\frac{KL}{r_{casing}} = 42.38 < C_c = \left[\frac{2\pi^2 E_{steel}}{F_{y-steel}}\right]^{1/2} = 128$$

$$F_a = \frac{F_{y-steel}}{FS}\left[1 - \frac{(KL/r)^2 F_{y-steel}}{4\pi^2 E_{steel}}\right] = 108\ \text{MPa}$$

$$P_{c-allowable} = \left[0.40\, f'_{c-grout}\, Area_{grout} + \frac{F_{y-steel}}{FS}\left(Area_{bar} + Area_{casing}\right)\right] \times \frac{F_a}{\frac{F_{y-steel}}{FS}}$$

$$= 636\ \text{kN}$$

Load Factor Design (LFD) Method

Similar to above except:

$$F_a = F_{y-steel}\left[1 - \frac{(KL/r)^2 F_{y-steel}}{4\pi^2 E_{steel}}\right] = 228 \text{ MPa}$$

$$P_{c-nominal} = \left[0.85\, f'_{c-grout}\, Area_{grout} + F_{y-steel}\left(Area_{bar} + Area_{casing}\right)\right] \times \frac{F_a}{F_{y-steel}}$$

$$= 1{,}349 \text{ kN}$$

And with $\varphi_c = 0.85$

$P_{c\text{-}design} = \varphi_c\, P_{c\text{-}nominal} = 1{,}147$ kN

5.F.6 Downdrag and Uplift Considerations

Piling systems may be subjected to additional compression loading due to downdrag forces from settling soils and additional tension loading due to uplift forces from expansive soils. A good discussion on the consideration of these forces for the design of drilled piers is included in the manual: *Drilled Shafts: Construction Procedures and Design Methods,* (FHWA-HI-88-042).

The use of micropiles for a foundation system on sites where downdrag or uplift forces are of concern provides several benefits. The small surface area of a micropile reduces the ability of the settling or expansive soil to transfer load to the pile. Further isolation of the pile from the moving soils can be accomplished by installation of an additional oversized outer casing through the moving soils. The use of battered piling should be avoided in such conditions where settlement or expansion will induce excessive lateral loading on the pile.

5.G SAMPLE PROBLEM NO.1 – BRIDGE ABUTMENT FOUNDATION SUPPORT

The following sample problem illustrates the design of foundation support for a bridge abutment using micropiles. This sample problem is intended to illustrate the quantification of typical abutment loads and the design of a micropile foundation for non seismic and seismic load groups. It also illustrates SLD and LFD methods (See 5.C). Practitioners typically follow different procedures around the United States in the design of bridge abutments, and therefore this sample problem is not intended to depict a "standard abutment" or a "standard abutment design procedure." For simplicity, this example problem considers only a portion of the longitudinal forces and no transverse forces. Abutment designs which include wind or seismic forces, for example, would have transverse as well as longitudinal forces. This example also does not illustrate construction load cases (e.g., full backfill prior to girder placement) that need to be incorporated into actual designs.

5.G.1 Problem Statement

The structure is a simply supported, single-span bridge, 30 meters long, supported on concrete retaining abutments. The superstructure consists of 5 AASHTO Type IV precast - prestressed concrete girders with a cast-in-place concrete deck.

The bridge abutment length is 10.5 meters. The abutment wall backfill material is medium dense sand with an angle of internal friction of 35 degrees and a unit weight of 17.5 kN/m^3. The unit weight of the concrete is 23.6 kN/m^3. Figure 5-13 shows the dimensions of the abutment. The pile details are shown on Figure 5-14.

A summary of loading applied to the bridge abutment is shown in Figure 5-15. All load values are per 1-meter length of the abutment. The seismic site design coefficient is 0.1 g.

The foundation soil conditions are described in the boring log contained in Figure 5-16. These soils consist of 2.5 meters of loose sandy gravel underlain by a moderately compressible soft, brown, fine sandy silt which is 1.5 meters thick. The silt is underlain by dense to very dense gravel with cobbles and boulders which extend to a maximum depth of 30 meters. Ground water is 4 meters below the top of footing. Type B3 (pressure grouted through the casing during casing withdrawal) micropiles will be used. The bond length will be formed in the dense gravel.

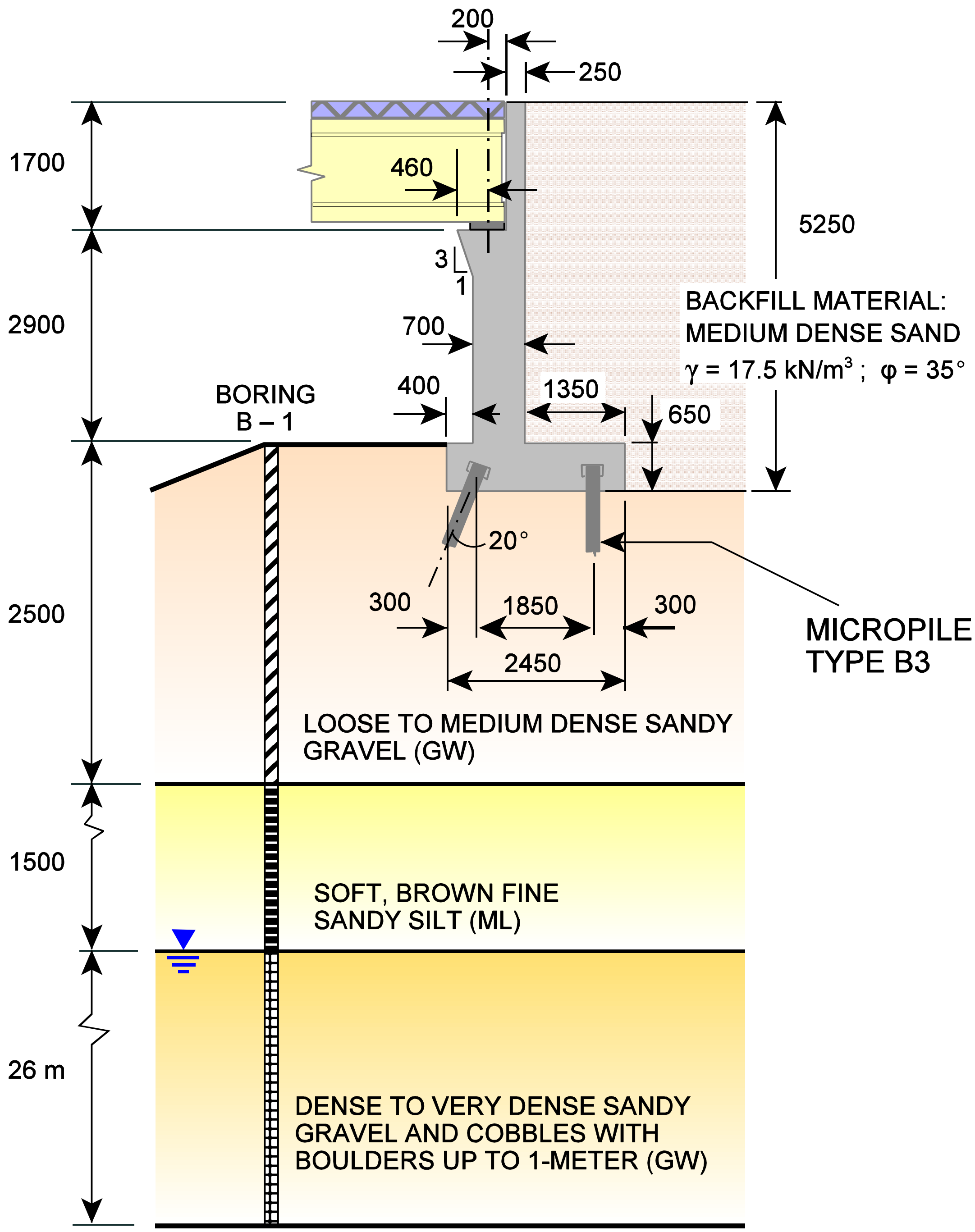


Figure 5 - 13. Sample Problem No. 1 - Abutment Section Detail.

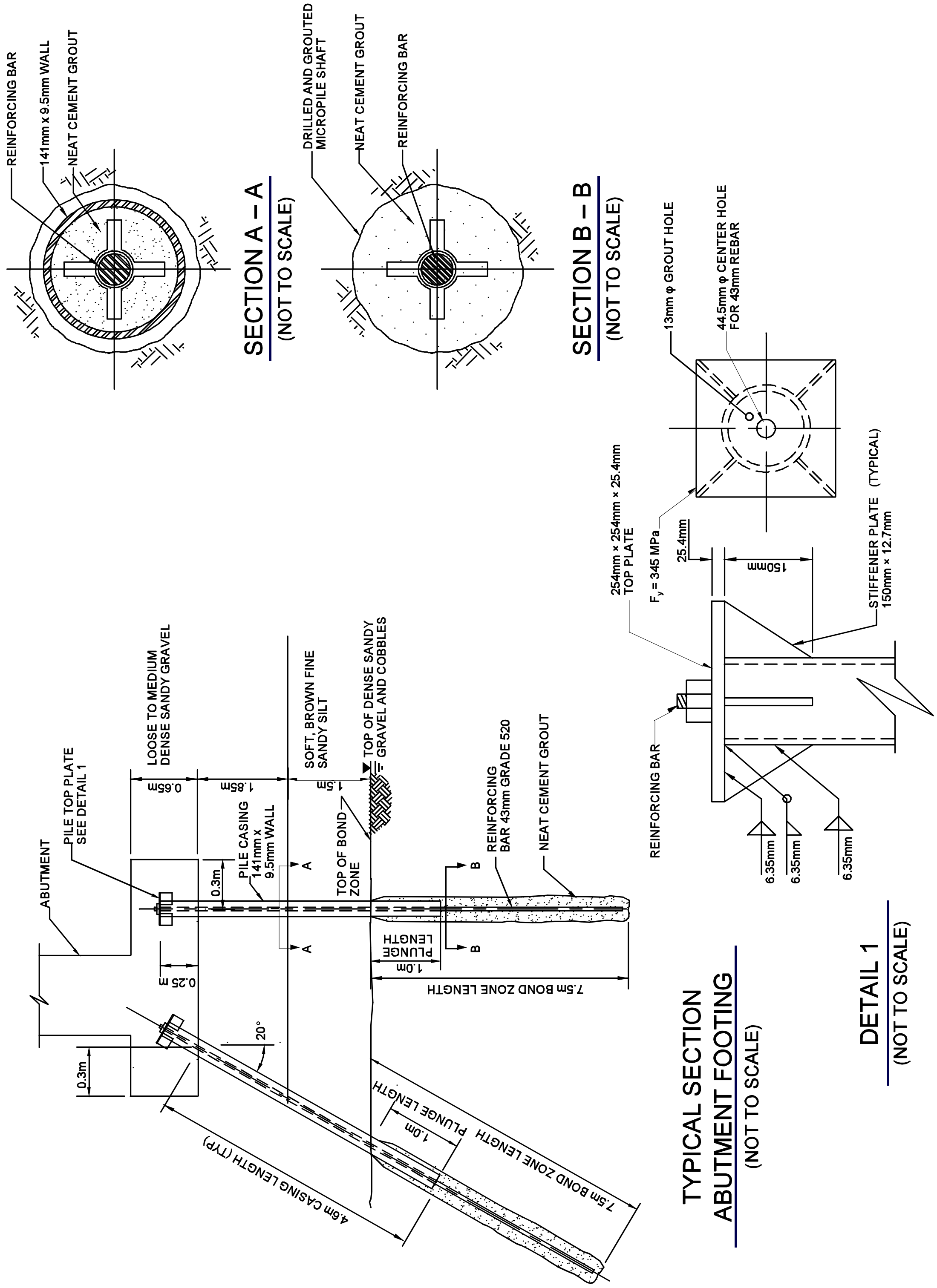
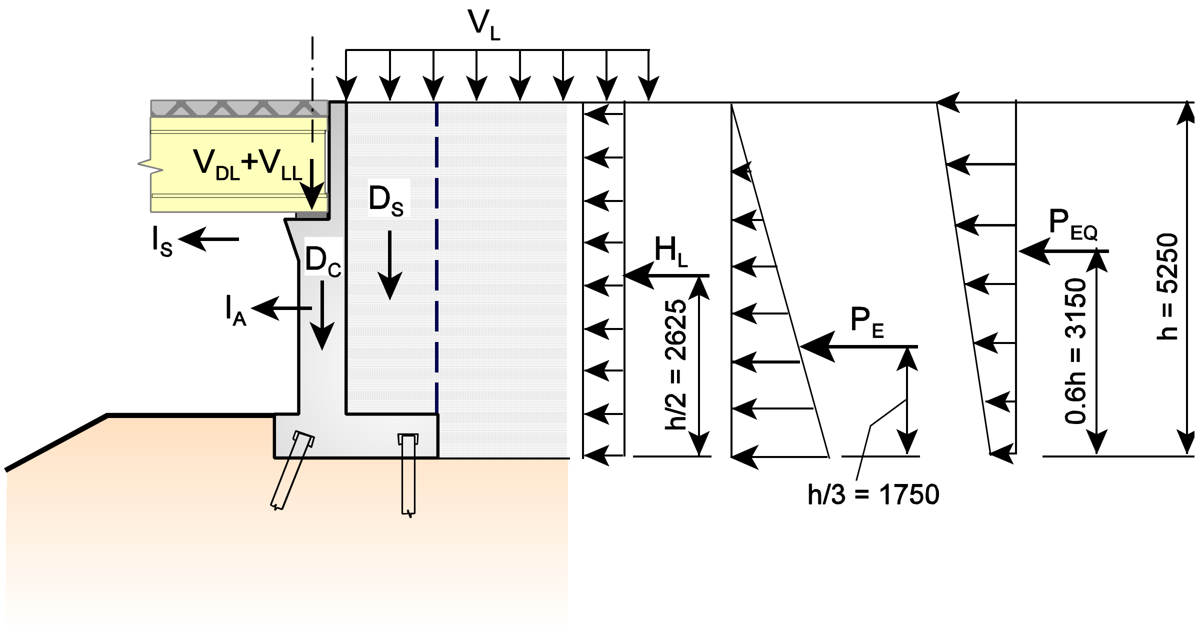


Figure 5 - 14. Sample Problem No. 1 - Pile Details



D_C = Dead load of concrete abutment

D_S = Dead load of soil

V_{DL} = 178.70 kN/m (dead load from bridge structure)

V_{LL} = 73.00 kN/m (live load from bridge structure)

H_L = Earth pressure due to live load surcharge V_L

V_L = Live load surcharge = 0.6 m thick equivalent soil surcharge (not included in this example)

P_E = Active earth pressure

P_{EQ} = Seismic earth pressure

I_A = 30.9 kN/m (seismic inertia force of concrete abutment and soil weight)

I_S = 26.8 kN/m (seismic inertia force of the superstructure)

Figure 5 - 15. Sample Problem No. 1 - Summary of Abutment Loads.

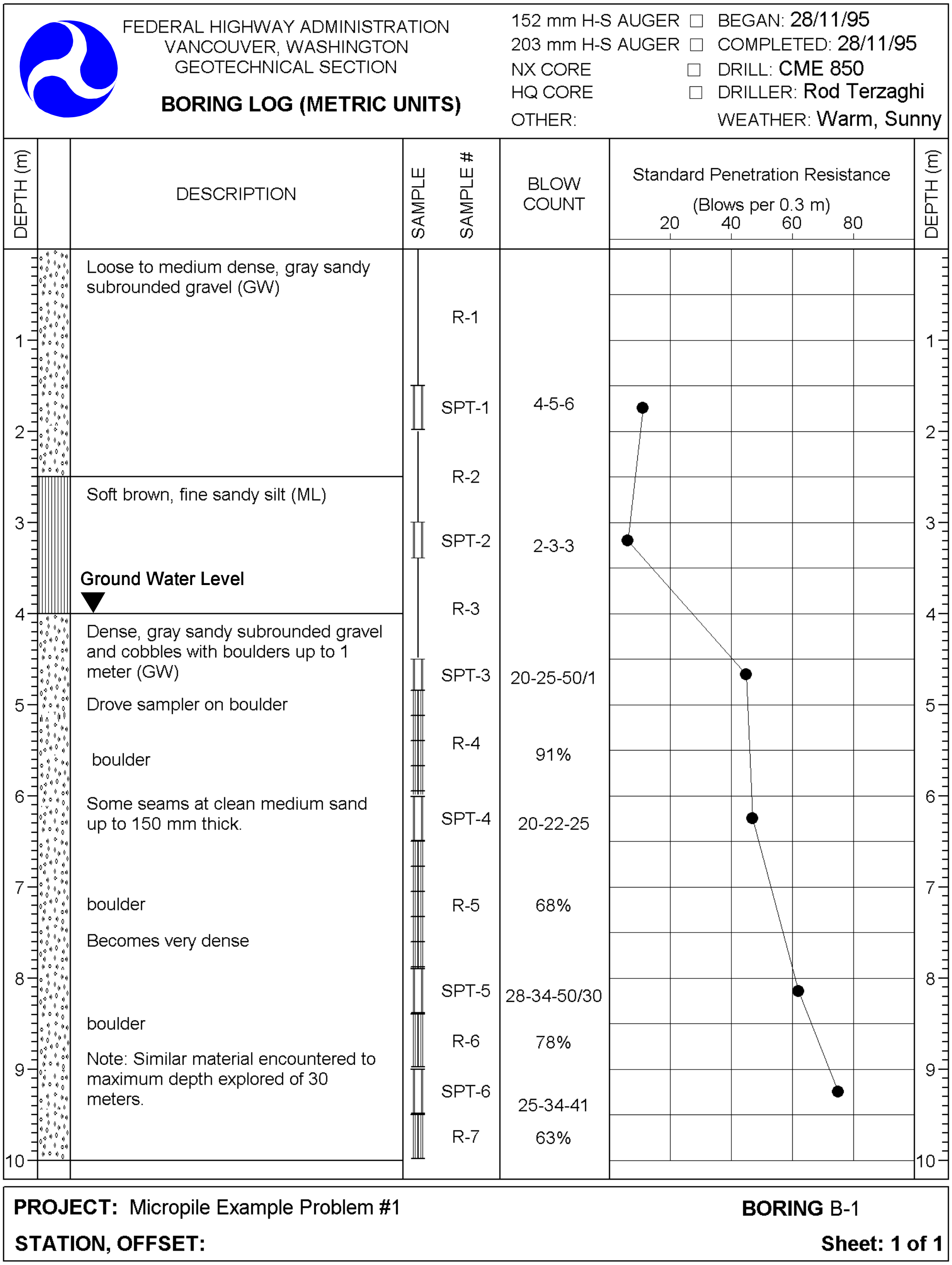


Figure 5 - 16. Sample Problem No. 1 - Soil Boring Log

The step-by-step procedure used for design of the micropiles is as follows:

Step 1 - Determine the magnitude and point of application of the design loads acting on the abutment.

Step 2 - Determine the summary of horizontal forces, vertical forces, and overturning moments acting on the abutment for each non seismic load group per AASHTO Section 3.22 and for the seismic load groups per AASHTO Division I-A. Select a pile layout and determine the front and rear pile design load (SLD) or required strength (LFD) for each load group.

Step 3 - Complete the design for the pile including:

1. The allowable load (SLD) or the design strength (LFD) of the upper cased length
2. The allowable load (SLD) or the design strength (LFD) of the lower uncased length
3. The allowable load (SLD) or the design strength (LFD) of the geotechnical bond length
4. Verify that the structural and geotechnical allowable loads exceed the design load for SLD or that the design strengths exceed the required strength for LFD as determined in step 2.

Step 4 - Determine and evaluate the anticipated displacements under service loading.

Step 5 - Complete the design for the pile top detail for connection of the piling to the abutment footing.

Step 6 - Complete drawings showing details of the pile design, pile layout, top connection detail, and general notes on materials and installation procedures.

For simplicity this example uses dead load, live load, earth pressure, and seismic loads. It illustrates a non seismic load group and a seismic load group as well as SLD and LFD methods. Steps 2, 3, and 4 are separated into two sections, one for the SLD method, and one for the LFD method.

5.G.2 Step 1 – Abutment Design Loads

The magnitude and point of application for the active earth pressure, earth pressure due to live load surcharge, and seismic earth pressure is determined in this section. Calculations for the remaining components of the abutment loading are not included. The magnitude, point of application, and resulting moment on the abutment for all of the load components are summarized in Table 5-4. The moments are taken about the center point of the base of the abutment footing.

5.G.2.1 Active Earth Pressure - P_E

Soil internal friction angle $\varphi = 35$ degrees

Unit weight of soil $\gamma_{soil} = 17.5 \text{ kN / m}^3$

Coefficient of active soil pressure $K_a = \tan^2\left[45° - \frac{\varphi}{2}\right] = 0.27$

Active earth pressure $K_a \gamma_{soil} = 4.74 \text{ kN/m}^3$

Resultant load $P_E = 0.5 \times 4.74 \frac{\text{kN}}{\text{m}^3} \times (5.25 \text{ m})^2 = 65.32 \text{ kN/m}$

Moment about centroid $65.32 \frac{\text{kN}}{\text{m}} \times \frac{5.25 \text{ m}}{3} = 114.31 \text{ kN m/m}$

Table 5-4. Sample Problem No. 1- Summary of Abutment Loads Per Meter Length

Load	Description	Type	Load		Moment Arm		Moment*
			F_x (kN)	F_y (kN)	X (m)	Y (m)	M (kN-m)
D_c	Dead load of concrete abutment	D		97.00	0.27		26.19
D_S	Dead load of soil	E		108.68	-0.55		-59.77
V_{DL}	Dead load from bridge superstructure	D		178.70	0.58		103.65
V_{LL}	Live load from bridge superstructure	L		73.00	0.58		42.34
H_L	Earth pressure due to live load surcharge	L	14.96			2.63	39.27
P_E	Earth pressure	E	65.32			1.75	114.31
P_{EQ-H}	Seismic earth pressure	EQ	15.91			3.15	50.12
I_A	Seismic inertial force of concrete & soil weight	EQ	30.9			2.35	72.50
I_S	Seismic inertial force of the superstructure	EQ	26.8			3.55	95.14

*Moment is calculated about the center of the footing at its base.

5.G.2.2 Earth Pressure Due to Live Load Surcharge - H_L

Surcharge pressure $$K_a \times \gamma_{soil} \times 0.6 \text{ m} = 2.84 \text{ kPa}$$

Surcharge load $$2.84 \text{ kN} \times 5.25 \text{ m} = 14.96 \text{ kN/m}$$

Moment about centroid $$14.96 \frac{\text{kN}}{\text{m}} \times \frac{5.25 \text{ m}}{2} = 39.27 \text{ kN m/m}$$

5.G.2.3 Seismic Earth Pressure - P_{EQ}

See "Seismic Design of Bridges - Design Example No. 3" Publication No. FHWA-SA-97-008 for a description of the following Mononobe - Okabe lateral seismic earth over pressure and the seismic inertia forces from the abutment self-weight and the soil resting on the abutment footing.

Seismic acceleration coefficient $A = 0.10$

Seismic coefficients

$$k_h = 1.5 \times A = 0.15 \quad \text{and} \quad k_v = 0 \text{ (assumed value)}$$

Reference AASHTO Sec. 6.4.3 (A) Div. 1A.

Slope of soil face $\beta = 0°$

Backfill slope angle $i = 0°$

Friction angle between soil and abutment, $\delta = \frac{1}{2} \varphi = 17.5°$

The wall friction was not used for the active earth pressure to be conservative. It was used for the seismic over pressure, however, to reduce some conservatism.

Seismic inertia angle $\theta = \operatorname{atan}\left[\dfrac{k_h}{1 - k_v}\right] = 8.53°$

Seismic earth pressure coefficient

$$\Psi = \left[1 + \sqrt{\frac{\sin(\varphi + \delta) \times \sin(\varphi - \theta - i)}{\cos(\delta + \beta + \theta) \times \cos(i - \beta)}}\right]^2 = 2.65$$

$$K_{AE} = \left[\frac{\cos^2(\varphi - \theta)}{\Psi \times \cos(\theta) \times \cos(\theta + \delta)} \right] = 0.34$$

Seismic earth pressure $K_{AE}\,\gamma_{soil} = 5.95\ \mathrm{kN/m^3}$

Resultant seismic force

$$P_{EQ} = \left[0.5 \times 5.95\ \frac{\mathrm{kN}}{\mathrm{m}^3} \times (5.25\ \mathrm{m})^2 \right] - P_E = 16.68\ \mathrm{kN/m}$$

Since this force is at angle δ to the horizontal, $P_{EQ-H} = 16.68\ \frac{\mathrm{kN}}{\mathrm{m}}\ (\cos\ \delta) = 15.91\ \mathrm{kN/m}$

To be conservative this example will neglect the vertical portion of P_{EQ} over the footing heel.

5.G.2.4 Seismic Inertia Forces

$$I_A = k_h\,(D_C + D_S) = 0.15 \times (97.0 + 108.68) = 30.9\ \mathrm{kN/m}$$

$$I_S = k_h\,(V_{DL}) = 0.15 \times 178.70 = 26.8\ \mathrm{kN/m}$$

Moment about centroid $16.68\ \frac{\mathrm{kN}}{\mathrm{m}} \times 0.6 \times 5.25\ \mathrm{m} = 52.54\ \frac{\mathrm{kN\,m}}{\mathrm{m}}$

5.G.3 Service Load Design (SLD) Method

See 5.C for a general description of the SLD method. The following section will illustrate SLD for non seismic loads. See 5.G.4 for the design for seismic loads since the LFD method is used for seismic in this manual.

5.G.3.1 Step 2 (SLD) – Determine Pile Design Loads Per Meter Length

The Group I loads per AASHTO Table 3.22.1A are calculated from the abutment loads shown in Table 5-4.

F_y = sum of vertical loads

F_x = sum of horizontal loads

M = sum of moments about the center of the footing at its base.

$$F_y = D_C + D_S + V_{DL} + V_{LL}$$

$$F_y = 97.0 + 108.7 + 178.7 + 73.0 = 457.4 \text{ kN/m}$$

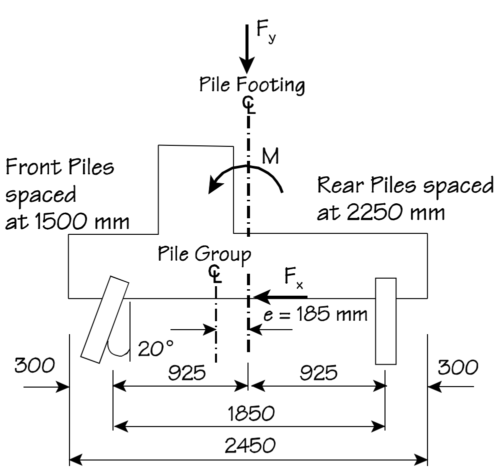
$$F_x = H_L + P_E$$

$$F_x = 15.0 + 65.3 = 80.3 \text{ kN/m}$$

$$M = D_C + D_S + V_{DL} + V_{LL} + H_L + P_E$$

$$M = 26.2 - 59.8 + 103.7 + 42.3 + 39.3 + 114.3 = 266.0 \text{ kN/m}$$

Determine Pile Group Properties Per Meter Length



Centroid of Pile Group (from front pile)					
Front piles:	1/1.5	×	0.0	=	0.0
Rear piles:	1 / 2.25	×	1.85	=	0.822
Sum:	1.111				0.822

So centroid is (0.822/1.111) = 0.74 m from front pile.

$$e = \frac{1.85}{2} - 0.74 = 0.185 \text{ m}$$

Pile Group I about centroid

$$I = \frac{1}{1.5}(0.74)^2 + \frac{1}{2.25}(1.11)^2 = 0.9127$$

$$\text{Rear Pile Vert Load} = \left[\frac{Fy}{1.111} - \frac{(M - F_y e)\ 1.11}{0.9127}\right]$$

$$\text{Front Pile Vert Load} = \left[\frac{Fy}{1.111} + \frac{(M - F_y e)\ 0.74}{0.9127}\right]$$

Group I Pile Design Loads

$$\text{Rear Pile Vert Load} = \left[\frac{457.4}{1.111} - \frac{[266 - 457.4 \times 0.185]\ 1.11}{0.9127}\right] = 191 \text{ kN}$$

$$\text{Front Pile Vert Load} = \left[\frac{457.4}{1.111} + \frac{[266 - 457.4 \times 0.185]\ 0.74}{0.9127}\right] = 559 \text{ kN}$$

$$\text{Front Pile Axial Load} = \frac{559}{\cos(20°)} = 595 \text{ kN} \quad \text{compression}$$

(Controlling Group 1, non-seismic, axial design load per pile)

A more refined analysis would utilize the vertical stiffness of the batter pile. For this example, the batter pile stiffness results in approximately a 7 percent larger front pile axial load.

5.G.3.2 Step 3 (SLD) – Determine Allowable Structural and Geotechnical Pile Loads

In the following section the material strength properties and reinforcement dimensions are selected for the pile section, and design calculations are done to determine the allowable load of the upper cased length, allowable load of the lower uncased length, and determination of the allowable geotechnical bond load.

A single pile design is used that can support the maximum compression load that acts on the front row of battered piles.

For the permanent casing reinforcement, a yield strength of 241 MPa is used for design. To maintain strain compatibility between the reinforcing bar and casing in the upper pile length, a yield strength ($F_{y\text{-}steel}$) of 241 MPa is used for both members. The use of a higher strength casing, such as API N80 (F_y = 552 MPa) would greatly increase the structural capacity of this section of the pile. These calculations do not address the capacity of the casing joint.

If the project incorporated a larger number of piles, savings could be realized by the use of separate designs for the front and rear piles. For example, a smaller casing size and a shorter bond length may be used on the rear piles due to the lower required capacity. The use of separate designs could increase the amount of load testing required at the start of the project, which the potential savings on a larger piling job could support. Consideration must also be given to maintaining simple pile construction. Mixing different reinforcing sizes and pile depths can occasionally lead to errors during pile installation. Also, if the overall stiffness (load versus pile top deflection) of the pile is changed, then the centroid and load per pile must be reevaluated.

5.G.3.2.1 Step 3 (SLD) – Pile Cased Length Allowable Load

Material dimensions and properties:

Casing - Use 141 mm outside diameter x 9.5 mm wall thickness.

Reduce outside diameter by 1.6 mm to account for corrosion.

Casing outside diameter OD_{casing} = 141 mm - 2 x 1.6 mm = 137.8 mm

Pile casing inside diameter ID_{casing} = 141 mm - 2 x 9.5 mm = 122 mm

Pile casing steel area $$Area_{casing} = \frac{\pi}{4}\left[OD^2_{casing} - ID^2_{casing} \right] = 3{,}224 \ mm^2$$

Casing yield strength $F_{y\text{-}casing}$ = 241 MPa

Radius of gyration $$r_{casing} = \frac{\sqrt{OD^2 + ID^2}}{4} = 46 \ mm$$

Reinforcing bar - Use 43 mm grade 520 steel reinforcing bar.

Bar area $Area_{bar} = 1{,}452 \ mm^2$

Bar steel yield strength $F_{y\text{-}bar}$ = 520 MPa

Cement Grout - Use neat cement grout.

Grout area $$Area_{grout} = \frac{\pi}{4} ID^2_{casing} - Area_{bar} = 10{,}240 \ mm^2$$

Grout compressive strength $f'_{c\text{-}grout}$ = 34.5 MPa

For strain compatibility between casing and rebar, use for steel yield stress:

$F_{y\text{-steel}}$ = the minimum of $F_{y\text{-bar}}$ and $F_{y\ casing}$ = 241 MPa

Tension - Allowable Load

$$P_{t\text{-allowable}} = 0.55\ F_{y\text{-steel}}\ (Area_{bar} + Area_{casing}) = 620\ kN$$

Compression - Allowable axial unit steel stress, with consideration for pile lateral stability -

FS = 2.12

$$\text{As} \quad L = 0 \qquad F_a = \frac{F_{y-steel}}{FS} = 113.7\ \text{MPa}$$

Allowable Load

$$P_{c-allowable} = \left[0.40\ f'_{c-grout}\ Area_{grout} + \frac{F_{y-steel}}{FS}\left(Area_{bar} + Area_{casing}\right)\right] \times \frac{F_a}{\dfrac{F_{y-steel}}{FS}}$$

$$= 673\ \text{kN}$$

5.G.3.2.2 Step 3 (SLD) – Pile Uncased Length Allowable Load

Material Dimensions and Properties

Soil conditions and the method of pile installation can affect the resulting diameter of the pile bond length. For this example, assume a drill-hole diameter of 50mm greater than the casing outside diameter (OD).

Using a casing OD = 141mm

Therefore, Grout DIA_{bond} = 141 mm + 50 mm = 191 mm (0.191 m)

Bond length grout area, $$Area_{grout} = \frac{\pi}{4} DIA_{bond}^2 - Area_{bar} = 27{,}200 \ mm^2$$

Based on previous experience estimate the plunge length allowable load $P_{transfer\ allowable}$ = 50 kN. This value will be verified later in the design. See section 5.E.6 for discussion about the plunge length.

Tension - Allowable load

$$P_{t\text{-}allowable} = 0.55\ F_{y\text{-}bar}\ Area_{bar} + P_{transfer\ allowable} = 465\ kN$$

Compression - Allowable load

$$P_{c-allowable} = 0.40\ f'_{c-grout}\ Area_{grout} + 0.47\ F_{y-bar}\ Area_{bar} + P_{transfer\ allowable}$$

$$= 780\ kN$$

5.G.3.2.3 Step 3 (SLD) – Allowable Geotechnical Bond Load

The pile bond length shall be located in the dense to very dense sandy gravel with cobbles and boulders starting approximately 3.35 meters deep below the bottom of footing elevation. The load capacity gained in the upper soils is ignored in the design analysis. The pile bond length shall be installed using a Type B pressure grouting methodology.

From Table 5-2 select an ultimate unit grout-to-ground bond strength $\alpha_{\text{bond nominal strength}} = 335$ kPa. An upper bound value is selected for the Type B micropile in gravel as the gravel is very dense and includes cobbles and boulders.

From 5.G.3.1, the controlling AASHTO Group 1 abutment, non-seismic, pile loading is 595 kN per pile. Therefore, an allowable geotechnical bond load $P_{\text{G-allowable}} \geq 595$ kN/pile must be provided to support the structural loading.

Provide: $P_{\text{G-allowable}} \geq 595$ kN/pile

Compute the geotechnical grouted bond length required to provide $P_{\text{G-allowable}}$ as follows:

$$P_{\text{G-allowable}} = \left[\frac{\alpha_{\text{bond nominal strength}}}{\text{FS}}\right] \times 3.14 \times \text{DIA}_{\text{bond}} \times (\text{bond length})$$

$$\geq 595 \text{ kN} \quad \text{(design load)}$$

$$\text{Bond Length} \geq \frac{595}{\frac{\alpha_{\text{bond nominal strength}}}{\text{FS}} \times 3.14 \times \text{DIA}_{\text{bond}}}$$

$$\text{Bond Length} \geq \frac{595 \text{ kN}}{\frac{335 \text{ kPa}}{2.5} \times 3.14 \times 0.191 \text{ m}} \geq 7.4 \text{ m}$$

Select Bond Length = 7.5 m

$$P_{G-allowable} = \frac{335\ \text{kPa}}{2.5} \times 3.14 \times 0.191\ \text{m} \times 7.5\ \text{m}$$

$$= 603\ \text{kN} \quad (> 595\ \text{kN})$$

Select a trial plunge length = 1m and verify $P_{transfer\ allowable}$ assumed at 50kN.

$$P_{transfer\ allowable} = \frac{335\ \text{kPa}}{2.5} \times 3.14 \times 0.191\ \text{m}$$

$$= 80\ \text{kN} \quad > 50\text{kN}$$

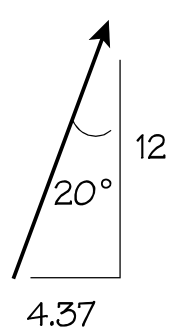
5.G.3.2.4 Step 3 (SLD) – Verify Allowable Axial Loads

From 5.G.3.1 the controlling axial design load for the pile is 595 kN (compression). The summary of the pile allowable loads is:

Structural Upper Cased Length	=	673 kN (compression)
Structural Lower Uncased Length	=	780 kN (compression)
Geotechnical Bond Length	=	603 kN (compression)

The allowable loads are all greater than the 595 kN design load, therefore the pile axial design is OK.

5.G.3.2.5 Step 3 (SLD) - Verify Allowable Lateral Loads



From 5.G.3.1 the controlling lateral design load is 80.3 kN/m. Check to see if the batter piles carry lateral loads. The front row of piles are battered at 20° from vertical and each pile carries the following ratio of the vertical pile load as a lateral resisting load.

$$\text{Lateral resisting load} = 559\ \text{kN}\left[\frac{4.37}{12}\right] = 204\ \text{kN per pile}$$

$$\text{The lateral design load per pile} = 80.3\ \frac{\text{kN}}{\text{m}} \times (1.5\ \text{m}) = 120\ \text{kN per pile}$$

The lateral resisting load is greater than the lateral design load therefore the pile lateral design is OK.

5.G.3.2.6 Step 3 (SLD) – Compute Field Test Loads

From 5.G.3.1 the controlling axial design load for the pile is 595 kN (compression).

$$\text{Verification Test Load} = 2.5 \times \text{Design Load}$$

$$= 2.5 \times 595\ \text{kN} = 1{,}500\ \text{kN}$$

$$\text{Proof Test Load} = 1.67 \times \text{Design Load}$$

$$= 1.67 \times 595\ \text{kN} = 1{,}000\ \text{kN}$$

See chapter 7 for pile load testing guidelines

5.G.3.2.7 Step 3 (SLD) – Pile Structural Design for Field Test Loads

See section 5.E.4 for more information about structural design for field test loads.

The design pile has the following properties:

141mm outside diameter × 9.5mm wall thickness

Cased Length		Uncased Length	
OD^*	= 141mm		
ID	= 122 mm		
$Area_{casing}$	= 3,925 mm^2	$Area_{grout}$	= 27,200 mm^2
$F_{y\text{-casing}}$	= 241 MPa (use for casing and bar)	$f'_{c\text{-grout}}$	= 34.5 MPa
$Area_{bar}$	= 1,452 mm^2	$Area_{bar}$	= 1,452 mm^2
$F_{y\text{-bar}}$	= 520 MPa	$F_{y\text{-bar}}$	= 520 MPa
$Area_{grout}$	= 10,240 mm^2		
$f'_{c\text{-grout}}$	= 34.5 MPa		

* Do not reduce by 1.6mm for corrosion as is done for permanent pile design.

Cased Length Allowable Compression Load

As the unsupported pile length = 0, $F_a = F_{y\text{-casing}} / 1.25$

$$P_{c-allowable} = \left[0.68\, f'_{c-grout}\, Area_{grout} + \frac{F_{y-casing}}{1.25}\left(Area_{bar} + Area_{casing}\right)\right] \times \frac{F_a}{\frac{F_{y-casing}}{1.25}}$$

$$= 1{,}277 \text{ kN}$$

Uncased Length Allowable Compression Load

$$P_{transfer\ allowable} = \left[\frac{\alpha_{bond\ nominal\ strength}}{FS\ =\ 1.25}\right] \times 3.14 \times DIA_{bond} \times (plunge\ length)$$

$$= \frac{335\ kPa}{1.25} \times 3.14 \times 0.191\ m \times 1\ m$$

$$= 160\ kN$$

$$P_{c-allowable} = 0.68\, f'_{c-grout}\, Area_{grout} + 0.80\, F_{y-bar}\ Area_{bar} + P_{transfer\ allowable}$$

$$= 0.68 \times 34{,}500\,(27.2 \times 10^{-3}) + 0.8\ (520 \times 10^{3})\,(1.452 \times 10^{-3}) + 160$$

$$= 1{,}402\ kN$$

The design pile is OK for the proof test at 1000 kN, but does not have the capacity for the verification test at 1500 kN; so try increasing the verification test pile to a 141mm OD with a 12.7mm wall and also increase the reinforcing bar to a 57mm size, grade 520.

Cased Length:

OD = 141 mm

ID = 141 - 2 × 12.7 = 115.6 mm

$$Area_{casing} = \frac{\pi}{4}\ [\,141^2 - 115.6^2\,] = 5{,}119\ mm^2$$

$F_{y\text{-}casing}$ = 241 MPa (use for casing and bar)

$\text{Area}_{\text{bar}} = 2{,}581 \text{ mm}^2$

$$\text{Area}_{\text{grout}} = \frac{\pi}{4}\left[115.6^2\right] - 2{,}581 = 7{,}915 \text{ mm}^2$$

$f'_{c\text{-grout}} = 34.5$ MPa

Uncased Length:

$\text{DIA}_{\text{bond}} = 141 + 50 = 191$ mm (0.191m)

$$\text{Area}_{\text{grout}} = \frac{\pi}{4}\left[191^2\right] - 2{,}581 = 26{,}071 \text{ mm}^2$$

$\text{Area}_{\text{bar}} = 2{,}581 \text{ mm}^2$

$F_{y\text{-bar}} = 520$ MPa

Cased Length Allowable Compression Load:

As the unsupported pile length = 0, $F_a = F_{y\text{-casing}} / 1.25$

$$P_{c-\text{allowable}} = \left[0.68\, f'_{c-\text{grout}}\, \text{Area}_{\text{grout}} + \frac{F_{y-\text{casing}}}{1.25}\left(\text{Area}_{\text{bar}} + \text{Area}_{\text{casing}}\right)\right] \times \frac{F_a}{\frac{F_{y-\text{casing}}}{1.25}}$$

$$= 1{,}670 \text{ kN} \quad > 1{,}500 \text{ kN} \quad \textbf{OK}$$

Uncased Length Allowable Compression Load:

$$P_{c-\text{allowable}} = 0.68\, f'_{c-\text{grout}}\, \text{Area}_{\text{grout}} + 0.80\, F_{y-\text{bar}}\, \text{Area}_{\text{bar}} + P_{\text{transfer allowable}}$$

$$P_{transfer\ allowable} = \left[\frac{\alpha_{bond\ nominal\ strength}}{FS = 1.25}\right] \times 3.14 \times DIA_{bond} \times (plunge\ length)$$

$$= \frac{335}{1.25} \times 3.14 \times 0.191\ m \times (1)$$

$$= 160\ kN$$

$P_{c\text{-}allowable} = 1{,}845$ kN $> 1{,}500$ kN **OK**

This upsized pile is OK for the verification test load of 1,500 kN.

5.G.3.3 Step 4 (SLD) – Anticipated Axial Displacement

Pile axial stiffness and displacement checks usually are not required for a bridge abutment but the following is shown to illustrate how it is done for the seismic pile loads.

For an approximate estimation of the anticipated axial displacement under maximum load, the pile is assumed to act elastically from the bottom of the abutment footing to the tip of the permanent casing.

Casing Length:

Front pile batter angle $\Phi_{batter} = 20°$

Max pile length from footing to top of gravel $L_{upper} = \frac{3.35\,m}{\cos(\Phi_{batter})} = 3.6\ m$

Casing insertion into bond length $L_{insert} = 1.0\ m$

Pile casing length $L_{casing} = L_{upper} + L_{insert} = 4.6\ m$

Pile stiffness values:

Steel modulus of elasticity $E_{steel} = 200{,}000 \ \text{MPa}$

Grout modulus of elasticity $E_{grout} = 31{,}000 \ \text{MPa}$

From Section 5.G.3.b.1 $Area_{grout} = 10{,}240 \ \text{mm}^2$

$$Area_{steel} = Area_{bar} + Area_{casing} = 4{,}676 \ \text{mm}^2$$

Pile tension stiffness value $AE_{tension} = \left[A_{steel} \times E_{steel}\right] = 935{,}000 \ \text{kN}$

Pile compression stiffness value

$$AE_{compression} = \left[A_{grout} \times E_{grout}\right] + \left[A_{steel} \times E_{steel}\right] = 1{,}250{,}000 \ \text{kN}$$

Elastic Displacement:

Group VII Required strength

$$P_{tension} = 56.4 \ \text{kN} \quad \text{and} \quad P_{compression} = 614.3 \ \text{kN}$$

Elastic displacement in tension $\Delta t_{elastic} = \dfrac{P_{tension} \times L_{casing}}{AE_{tension}} = 0.3 \ \text{mm}$

Elastic displacement in compression $\Delta c_{elastic} = \dfrac{P_{compression} \times L_{casing}}{AE_{compression}} = 2.2 \ \text{mm}$

Pile Total Displacements:

The total pile displacement consists of the elastic (recoverable) and residual (permanent) displacements. The magnitude of the residual displacement can be estimated based on experience from previous pile testing with consideration of the total pile length, soil type, and the magnitude of the applied loads. From previous experience, use the following values for the inelastic displacement.

Inelastic displacement $\Delta_{residual} = 0.2 \text{ mm}$ *(Tension)*

$\Delta_{residual} = 2.5 \text{ mm}$ *(Compression)*

Total displacement, tension $\Delta t_{total} = \Delta t_{elastic} + \Delta_{residual} = 0.5 \text{ mm}$

Total displacement, compression $\Delta c_{total} = \Delta c_{elastic} + \Delta_{residual} = 4.7 \text{ mm}$

These calculations illustrate how to estimate the design dispacements. When refined displacements are required, a careful evaluation of elastic lengths and residual displacements will be necessary.

5.G.3.4 Step 5 (SLD) – Pile Connection Design

Design calculations are completed for the detail shown in Figure 5-17 for connection of the pile top to the abutment footing.

Required Design Loads, Dimensions

Group I service load, compression $P_{c\text{-}service} = 594.6 \text{ kN}$

Abutment concrete compressive strength $f'_c = 27.6 \text{ MPa}$

Casing outside diameter $OD_{casing} = 141 \text{ mm}$

Pile area $Area_{pile} = \frac{\pi}{4} OD_{casing}^2 = 15{,}615 \text{ mm}^2$

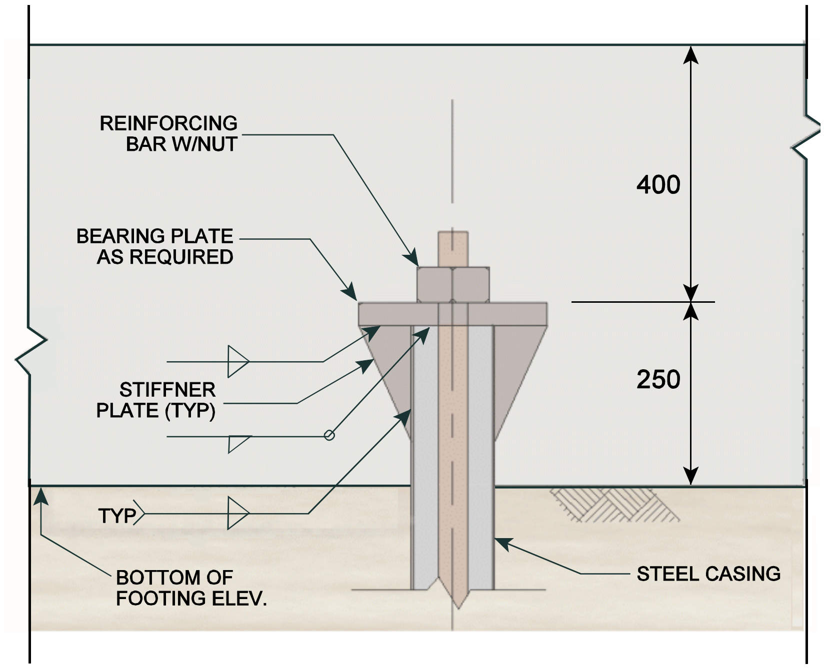
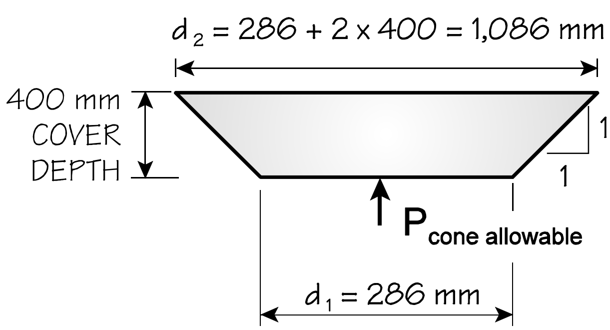


Figure 5 - 17. Pile Top to Abutment Footing Connection Detail.

Required Plate Area

Assume a 254 mm square plate $\quad \text{Area}_{plate} = \text{Plate}^2_{width} = 64{,}516 \ \text{mm}^2$

Check Cone Shear



$$P_{\text{cone allowable}} \geq P_{\text{c-service}} = 594.6 \ \text{kN}$$

Equivalent diameter for 254 mm^2 plate, $d_1 = \sqrt{\dfrac{4 \times 254^2}{\pi}} = 286 \text{ mm}$

$$A_{CP} = \frac{\pi}{4}\left[d_2^2 - d_1^2\right] = \frac{\pi}{4}\left[1086^2 - 286^2\right] = 862{,}053 \text{ mm}^2$$

Use ACI 349 Appendix B for cone shear

$$P_{\text{cone design strength}} = 4\ \varphi \sqrt{f_c' \text{ psi}} \times A_{CP}$$

$$= 4\ \varphi \sqrt{f_c' \text{ psi } (6.89476) \frac{\text{kPa}}{\text{psi}}} \times A_{CP}$$

$$= 10.5\ \varphi \sqrt{f_c' \text{ kPa}} \times A_{CP}$$

$$P_{\text{cone nominal strength}} = 10.5 \sqrt{f_c' \text{ kPa}} \times A_{CP}$$

$$= 10.5 \sqrt{27.6 \text{ mPa} \left(\frac{1000 \text{ kPa}}{\text{mPa}}\right)} \times A_{CP} \times 10^{-6}$$

$$= 1{,}504 \text{ kN}$$

$$P_{\text{cone allowable}} = \frac{P_{\text{cone nominal strength}}}{FS}$$

$$FS = \frac{LF}{\varphi}$$

$$LF = \text{Combined load factor} = \frac{\gamma\,[\,B_D Q_D + B_L Q_L + B_E Q_E\,]}{[\,Q_D + Q_L + Q_E\,]}$$

where: $\gamma = 1.3$; $B_D = 1$; $B_L = 1.67$; $B_E = 1.3$

and with $Q_D = 0.62$; $Q_L = 0.16$; $Q_E = 0.22$

$$\frac{Q_D}{Q_D + Q_L + Q_E} = 0.62$$

$$\frac{Q_L}{Q_D + Q_L + Q_E} = 0.16$$

$$\frac{Q_E}{Q_D + Q_L + Q_E} = 0.22$$

$$LF = 1.3\,[1.0 \times 0.62 + 1.67 \times 0.16 + 1.3 \times 0.22] = 1.53$$

$\varphi = 0.65$ for unreinforced shear cone per ACI 349 Appendix B

$$FS = \frac{1.53}{0.65} = 2.35$$

$$P_{conc\ allowable} = \frac{1504\ \text{kN}}{2.35} = 640\ \text{kN} > 594.6\ \text{kN}\ .\ .\ .\ .\ \ \text{OK}$$

Note - edge distance limitations and other requirements of ACI 349 Appendix B must be satisfied.

Required Plate Thickness

Actual bearing stress, compression $\quad \text{Bearing}_{\text{compression}} = \dfrac{P_{c-\text{service}}}{\text{Area}_{\text{plate}}} = 9.22 \text{ MPa}$

Plate bending moment for 10 mm width

$$M_{max} = 10 \text{ mm} \times \frac{1}{2}\left[\frac{\text{Plate}_{\text{width}} - \text{OD}_{\text{casing}}}{2}\right]^2 \times \text{Bearing}_{\text{compression}} = 0.147 \text{ kN m}$$

Note - use of this cantilever moment was determined to be a conservative approximation of the maximum moment in the plate due to a compression pile load by comparison to a more accurate formula shown in Roark and Young (1975). Plate bending from a tension pile load or with different plate support details must be further analyzed.

Allowable bending stress - $\quad F_y = 345 \text{ MPa} \quad$ and $\quad F_b = 0.55 F_y$

Note - Allowable stresses should be increased by the appropriate % factor (AASHTO Table 3.22.1A column 14) depending on which load group produces controlling load condition. For this example, the Group I load produces the controlling condition (% = 100%).

$$S_{x-req} = \frac{M_{max}}{F_b} = 775 \text{ mm}^3$$

Required plate thickness - $\quad t_{req} = \sqrt{\dfrac{6\, S_{x-req}}{10 \text{ mm}}} = 21.6 \text{ mm}$

Use 254 mm square x 25.4 mm thick top plate.

Required Weld Size:

Tensile strength of E70 electrode, $Fu_{weld} = 483$ MPa

Minimum tensile strength of connected parts (ASTM A53 Grade B Casing)

$$Fu_{part} = 414 \text{ MPa}$$

Fillet weld strength $\Phi F = 0.27\, Fu_{part} = 111.8 \text{ MPa}$

Top weld size $t_{weld-top} = 6.35 \text{ mm}$

Stiffener plate size $t_{stiff} = 12.7 \text{ mm}$ $W_{stiff} = 100 \text{ mm}$ $L_{stiff} = 150 \text{ mm}$

Top weld length $L_{weld} = \pi\, OD_{casing} - 4\, t_{stiff} + 8\, W_{stiff} = 1{,}190 \text{ mm}$

Top weld strength

$$P_{weld-top} = 0.707\, t_{weld-top}\, \Phi F\, L_{weld} = 598 \text{ kN} > 595 \text{ kN} \ . \ . \ . \ \textbf{OK}$$

Stiffener plate side weld dimensions $t_{weld-side} = 6.35 \text{ mm}$

$$L_{weld} = 8\, L_{stiff} = 1{,}200 \text{ mm}$$

Stiffener plate side weld strength

$$P_{weld-side} = 0.707\, t_{weld-side}\, \Phi F_{weld}\, L_{weld} = 603 \text{ kN}$$

$$> 0.707\, t_{weld-top}\, \Phi F \times 8\, W_{stiff} = 402 \text{ kN} \ . \ . \ . \ . \ \text{OK}$$

Use 6.35 mm fillet weld for welding top and stiffener plates.

Note - this example ignored bending stresses on the welds which is considered appropriate for a compression pile load with the weld details shown. Weld bending stress must be analyzed for other conditions.

5.G.4 Load Factor Design (LFD) Method

See 5.C for a general description of the LFD method. The following section will illustrate LFD for non seismic and seismic loads.

5.G.4.1 Step 2 (LFD) – Determine Pile Required Strengths Per Meter Length.

The Group I factored loads per AASHTO Table 3.22.1A are calculated from the abutment loads shown in Table 5-4.

F_y = sum of factored vertical loads

F_x = sum of factored horizontal loads

M = sum of factored moments about the center of the footing at its base

$$F_y = \gamma\,[B_D\,D_C + B_D\,D_S + B_D\,V_{DL} + B_L\,V_{LL}]$$

$$F_y = 1.3\,[(1.0)\,(97.0) + (1.0)\,(108.7) + (1.0)\,(178.7) + (1.67)\,(73.0)] = 658.2 \text{ kN/m}$$

$$F_x = \gamma\,[B_E\,H_L + B_E\,P_E]$$

$$F_x = 1.3\,[(1.3)\,(14.96) + 1.3\,(65.32)] = 135.7 \text{ kN/m}$$

$$M = \gamma\,[B_D\,D_C + B_D\,D_S + B_D\,V_{DL} + B_L\,V_{LL} + B_E\,H_L + B_E\,P_E]$$

$$M = 1.3\,[(1.0)\,(26.2) - (1.0)\,(59.8) + (1.0)\,(103.7) + (1.67)\,(42.3) + (1.3)\,(39.3) + (1.3)\,(114.3)] = 442.5 \text{ kNm/m}$$

The Group VII seismic loads per AASHTO Division I-A (typical section is 6.2.1) are calculated from the abutment loads shown in Table 5-4.

F_y = sum of vertical loads

F_x = sum of horizontal loads

M = sum of moments about the center of the footing at its base

Note: All load factors are 1.0 for load groups that include seismic loads.

$F_y = D_C + D_S + V_{DL}$

$F_y = 97.0 + 108.7 + 178.7 = 384.4$ kN/m

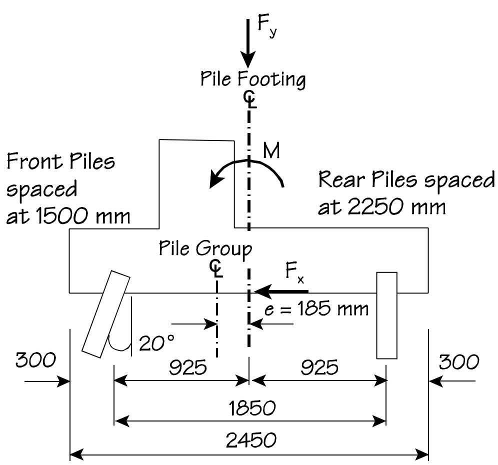
$F_x = P_E + P_{EQ\text{-}H} + I_A + I_S$

$F_x = 65.32 + 15.91 + 30.9 + 26.8 = 138.9$ kN/m

$M = D_C + D_S + V_{DL} + P_E + P_{EQ\text{-}H} + I_A + I_S$

$M = 26.2 - 59.8 + 103.7 + 114.3 + 50.12 + 72.5 + 95.1 = 402.1$ kNm/m

Determine Pile Group Properties Per Meter Length



Centroid of Pile Group (from front pile)					
Front piles:	1/1.5	×	0.0	=	0.0
Rear piles:	1 / 2.25	×	1.85	=	0.822
Sum:	1.111				0.822

So centroid is at (0.822/1.111) = 0.74 m from front pile.

$$e = \frac{1.85}{2} - 0.74 = 0.185 \text{ m}$$

Pile Group I about centroid

$$I = \frac{1}{1.5}(0.74)^2 + \frac{1}{2.25}(1.11)^2 = 0.9127 \text{ m}^4$$

$$\text{Rear Pile Vert Load} = \left[\frac{F_y}{1.111} - \frac{(M - F_y\, e)\; 1.11}{0.9127}\right]$$

$$\text{Front Pile Vert Load} = \left[\frac{F_y}{1.111} + \frac{(M - F_y\, e)\; 0.74}{0.9127}\right]$$

Group I Pile Required Strengths (Non Seismic)

$$\text{Rear Pile Vert Load} = \left[\frac{658.2}{1.111} - \frac{[442.5 - 658.2(0.185)]\; 1.11}{0.9127}\right] = 202.4 \text{ kN}$$

$$\text{Front Pile Vert Load} = \left[\frac{658.2}{1.111} + \frac{[442.5 - 658.2(0.185)]\; 0.74}{0.9127}\right] = 852.7 \text{ kN}$$

$$\text{Front Pile Axial Load} = \frac{852.7}{\cos(20^\circ)} = 907.4 \text{ kN} \quad \text{Compression}$$

(Controlling Group 1, non-seismic, required axial design strength per pile)

A more refined analysis would utilize the vertical stiffness of the batter pile. For this example, the batter pile stiffness results in approximately a 7 percent larger front pile axial load.

Group VII Pile Required Strengths (Seismic)

$$\text{Rear Pile Vert Load} = \left[\frac{384.4}{1.111} - \frac{[402 - 384.4(0.185)] \times 1.11}{0.9127} \right] = -56.4 \text{ kN} \quad \text{Tension}$$

$$\text{Front Pile Vert Load} = \left[\frac{384.4}{1.111} + \frac{[402 - 384.4(0.185)]\ 0.74}{0.9127} \right] = 614.3 \text{ kN}$$

$$\text{Front Pile Axial Load} = \frac{614.3}{\cos(20°)} = 654 \text{ kN} \quad \text{Compression}$$

(Controlling Group VII, seismic, required axial design strength per pile)

5.G.4.2 Step 3 (LFD) – Determine Structural and Geotechnical Pile Design Strengths

In the following section the material strength properties and reinforcement dimensions are selected for the pile section, and design calculations are done to determine the design strength of the upper cased length, design strength of the lower uncased length, and determination of the design geotechnical bond strength.

A single pile design is used that can support the maximum compression load that acts on the front row of battered piles and the maximum tension load which acts on the rear row of vertical piling (only required for seismic).

For the permanent casing reinforcement, a yield strength of 241 MPa is used. To maintain strain compatibility between the reinforcing bar and casing in the upper pile length, a yield strength (F_{y_steel}) of 241 MPa is used for both members. The use of a higher strength casing, such as API N80 (F_y = 552 MPa) would greatly increase the structural capacity of this section of the pile. These calculations do not address the capacity of the casing joint.

If the project incorporated a larger number of piles, savings could be realized by the use of separate designs for the front and rear piles. For example, a smaller casing size and a shorter bond length may be used on the rear piles due to the lower required capacity. The use of separate designs could increase the amount of load testing required at the start of the project, which the potential savings on a larger piling job could support. Consideration must also be given to maintaining simple pile construction. Mixing different reinforcing sizes and pile depths can occasionally lead to errors during pile installation.

5.G.4.2.1 Step 3 (LFD) – Pile Cased Length Design Strength

Material dimensions & properties -

Casing - Use 141 mm outside diameter x 9.5 mm wall thickness. Reduce outside diameter by 1.6 mm to account for corrosion.

Casing outside diameter $OD_{casing} = 141\ \text{mm} - 2 \times 1.6\ \text{mm} = 137.8\ \text{mm}$

Pile casing inside diameter $ID_{casing} = 141\ \text{mm} - 2 \times 9.5\ \text{mm} = 122\ \text{mm}$

Pile casing steel area $Area_{casing} = \frac{\pi}{4}\left[OD^2 - ID_{casing}^2 \right] = 3{,}224\ \text{mm}^2$

Casing yield strength $F_{y-casing} = 241\ \text{MPa}$

Radius of gyration $\qquad r_{casing} = \dfrac{\sqrt{OD^2 + ID^2}}{4} = 46 \text{ mm}$

Reinforcing bar - Use 43 mm grade 520 steel reinforcing bar.

Bar area $\qquad \text{Area}_{bar} = 1{,}452 \text{ mm}^2$

Bar steel yield strength $\qquad F_{y-bar} = 520 \text{ MPa}$

Cement Grout - Use neat cement grout.

Grout area $\qquad \text{Area}_{grout} = \dfrac{\pi}{4} ID_{casing}^2 - \text{Area}_{bar} = 10{,}240 \text{ mm}^2$

Grout compression strength $\qquad f'_{c-grout} = 34.5 \text{ MPa}$

For strain compatibility between casing and rebar, use for steel yield stress:

$$F_{y-steel} = \text{the minimum of } F_{y-bar} \;\&\; F_{y-casing} = 241 \text{ MPa}$$

Tension - Design Strength

$$P_{t-nominal} = F_{y-steel}\,[\,\text{Area}_{bar} + \text{Area}_{casing}\,] = 1{,}127 \text{ kN}$$

$$\varphi_t = 0.90 \qquad P_{t-design} = \varphi_t P_{t-nominal} = 1{,}014 \text{ kN}$$

Compression - Design Strength

$$\text{As } L = 0, \quad F_a = F_{y\,steel} = 241 \text{ MPa}$$

Design Strength

$$P_{c-nominal} = \left[0.85\, f_{c-grout}\ Area_{grout} + F_{y-steel}\left(Area_{bar} + Area_{casing}\right)\right] \times \frac{F_a}{F_{y-steel}}$$

$$= 1{,}427\ \text{kN}$$

$$\text{and with} \quad \varphi_c = 0.85, \qquad P_{c-design} = \varphi_c\, P_{c-nominal} = 1{,}213\ \text{kN}$$

5.G.4.2.2 Step 3 (LFD) – Pile Uncased Length Design Strength

Material Dimensions and Properties

Soil conditions and the method of pile installation can affect the resulting diameter of the pile bond length. For this example, assume a drill-hole diameter of 50mm greater than the casing outside diameter (OD).

Using a casing OD = 141mm

Therefore, Grout DIA_{bond} = 141 mm + 50 mm = 191 mm (0.191 m)

Bond length grout area $\qquad Area_{grout} = \frac{\pi}{4} DIA^2_{bond} - Area_{bar} = 27{,}200\ \text{mm}^2$

Based on previous experience estimate the plunge length design strength $P_{transfer\ design}$ = 110 kN. This value will be verified later in the design. See section 5.E.6 for discussion about the plunge length.

Tension - Design Strength

$$P_{t\text{-}design} = (0.90)\, F_{y\text{-}bar}\ Area_{bar} + P_{transfer\ design} = 790\ \text{kN}$$

Compression - Design Strength

$$P_{t\text{-}design} = (0.75)\,(0.85\, f'_{c\text{-}grout}\ Area_{grout} + F_{y\text{-}bar}\ Area_{bar}) + P_{transfer\ design} = 1{,}275\ \text{kN}$$

5.G.4.2.3 Step 3 (LFD) – Geotechnical Bond Design Strength

The pile bond length shall be located in the dense to very dense sandy gravel with cobbles and boulders starting approximately 3.35 meters deep below the bottom of footing elevation. The load capacity gained in the upper soils is ignored in the design analysis. The pile bond length shall be installed using a Type B pressure grouting methodology. From Table 5-2 select an ultimate unit grout-to-ground bond strength, $\alpha_{\text{bond nominal strength}}$ = 335 kPa. An upper bound value is selected for the Type B micropile in gravel as the gravel is very dense and includes cobbles and boulders.

From section 5.G.4.1, the controlling AASHTO Group 1, non-seismic, required pile axial strength is 907 kN per pile. Therefore, a geotechnical bond axial strength, $P_{\text{G-design strength}} \geq 907$ kN per pile, must be provided to support the structural loading. The required geotechnical grouted bond length may be computed as follows:

Provide: $P_{\text{G-design strength}} \geq$ 907 kN/pile

$$P_{\text{G-design strength}} = \varphi_G \left[\alpha_{\text{bond nominal strength}} \right] \times 3.14 \times DIA_{\text{bond}} \times (\text{Bond Length})$$

$$\geq 907 \text{ kN} \quad (\text{required strength})$$

$$\text{Bond Length} \geq \frac{907}{\varphi_G \, [\alpha_{\text{bond nominal strength}}] \times 3.14 \times DIA_{\text{bond}}}$$

$$\text{Bond Length} \geq \frac{907}{0.6\,(335 \text{ kPa}) \times 3.14 \times 0.191 \text{ m}} \geq 7.5 \text{ m}$$

Select Bond Length = 7.5 m

For Group I (non-seismic) use φ_G = 0.60

$$P_{G-\text{design strength}} = 0.60 \times \ 335 \text{ kPa} \ \times 3.14 \times 0.191 \text{ m} \times 7.5 \text{ m}$$

$$= 904 \text{ kN} \quad (\approx 907 \text{ kN})$$

For Group VII (seismic) use φ_G = 1.00

$$P_{G-\text{design strength}} = 1.0 \times \ 335 \text{ kPa} \ \times 3.14 \times 0.191 \text{ m} \times 7.5 \text{ m}$$

$$= 1507 \text{ kN}$$

Select a trial plunge length = 1m and verify $P_{\text{transfer design}}$ assumed at 110 kN.

For Group I (non seismic) use φ_G = 0.60

$$P_{\text{transfer design}} = 0.60 \times (335 \text{ kPa}) \times 3.14 \times 0.191 \text{ m} \times (1 \text{ m})$$

$$= 120 \text{ kN} \ > 110 \text{ kN} \ . \ . \ . \text{ OK}$$

For Group VII (seismic) use φ_G = 1.0

$$P_{\text{transfer design}} = 1.0 \times (335 \text{ kPa}) \times 3.14 \times 0.191 \text{ m} \times (1 \text{ m})$$

$$= 200 \text{ kN} > 110 \text{ kN} \ . \ . \ . \text{ OK}$$

5.G.4.2.4 Step 3 (LFD) – Verify Axial Design Strengths

From 5.G.4.1, the controlling axial required strengths for the pile are:

Group I (non seismic) = 907 kN (compression)

Group VII (seismic) = 654 kN (compression) and −56 kN (tension)

The summary of the pile design strengths is:

	Non Seismic	Seismic
Structural Upper Cased Length	1213 kN (C)	1213 kN (C) 1014 kN (T)
Structural Lower Cased Length	1275 kN (C)	1275 kN (C) 790 kN (T)
Geotechnical Bond Length	904 kN (C)	1507 kN (C) 1507 kN (T)

The pile design strengths are all equal to (904 ≈907) or greater than the required strengths; therefore the pile axial design is OK.

Figure 5-18 shows the SLD (at position 1) and LFD (at position 2) to illustrate that both methods (for this example) provide the same factor-of-safety (2.5) for the nominal strength of the geotechnical grout-to-ground bond. As discussed in 5.C this condition was obtained by calibrating φ_G for LFD to SLD with FS = 2.5. See 5.C for more discussion about SLD, LFD, and φ_G calibration. Figure 5-18 also shows that both SLD and LFD methods produce the same requirements for the verification field test load and the proof field test load.

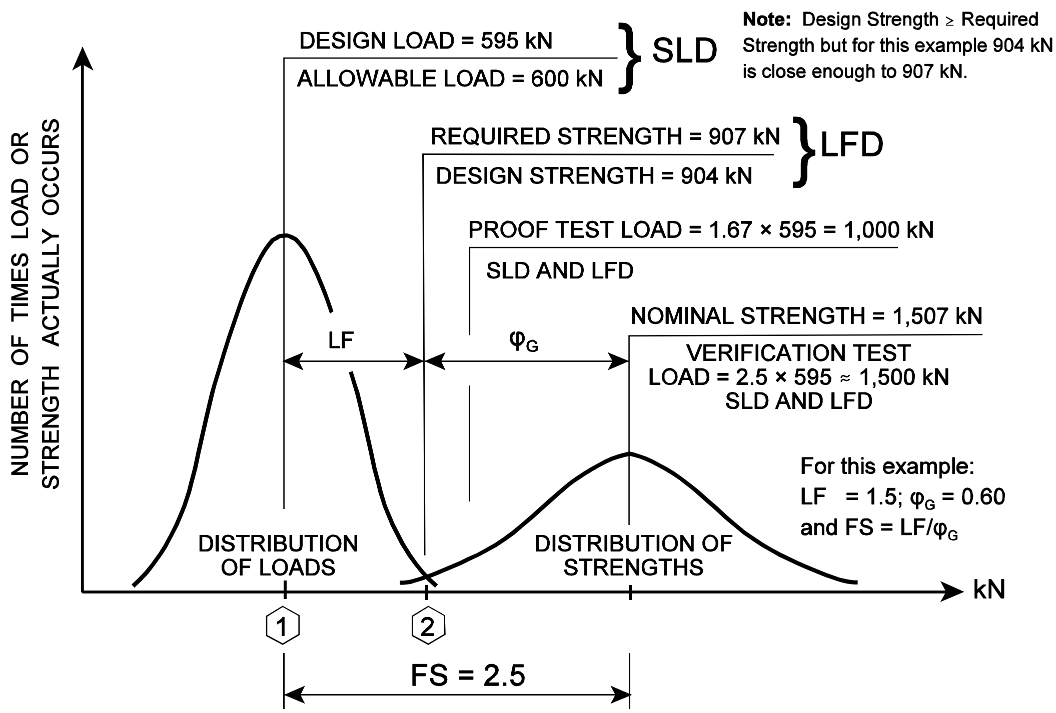


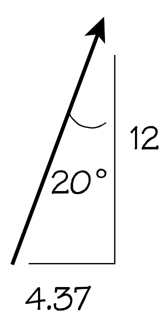
Figure 5 - 18. Comparison of SLD and LFD for Geotechnical Bond Length Design.

5.G.4.2.5 Step 3 (LFD) – Verify Lateral Design Strength

From 5.G.4.1, the controlling lateral required strengths for the pile per meter are:

Group I (non seismic) = 135.7 kN/m

Group VII (seismic) = 138.9 kN/m



Check to see if the batter piles have sufficient strength to carry the lateral required strengths. The front row of piles are battered at 20° from vertical and each pile carries the following ratio of the vertical pile required strength.

Group I (non seismic)

Lateral design strength = $853\ \text{kN} \times \frac{4.37}{12} = 311\ \text{kN per pile}$

The lateral required strength per pile = $135.7\ \frac{\text{kN}}{\text{m}} \times (1.5\ \text{m}) = 204\ \text{kN per pile}$

Pile OK for Group I

Group VII (seismic)

Lateral design strength = $614\ \text{kN} \times \frac{4.37}{12} = 224\ \text{kN per pile}$

The lateral required strength per pile = $(138.9\ \frac{\text{kN}}{\text{m}}) \times (1.5\ \text{m}) = 208\ \text{kN per pile}$

Pile OK for Group VII

5.G.4.2.6 Step 3 (LFD) – Compute Field Test Load.

From 5.G.4.1, the controlling axial required strengths for the pile are:

Group I (non seismic) = 907 kN (compression)

Group VII (seismic) = 654 kN (compression) and −56 kN (tension)

The verification test load is equal to the nominal strength required for the controlling required strength which is 907 kN for this example.

φ_G (Nominal Strength) = 907 kN

φ_G = 0.60 for Group I, so Nominal Strength $= \frac{907\ \text{kN}}{0.6} = 1500\ \text{kN}$

$$\text{The proof test load is} \left[\frac{1.67}{2.5}\right] \times (\text{Nominal Strength})$$

$$\text{so the proof test load} = \frac{1.67}{2.5} \times 1500 = 1000 \text{ kN}.$$

Note that these are the same as the SLD since the LFD was calibrated to SLD. See chapter 7 for pile load testing discussion and requirements.

5.G.4.2.7 Step 3 (LFD) – Pile Structural Design for Field Test Loads

Section 5.E.4 describes a SLD method with a FS = 1.25 for the pile structural design for the field test loads. Section 5.G.3.2.7 illustrates this method for this Sample Problem.

5.G.4.2.8 Step 4 (LFD) – Anticipated Axial Displacement

See section 5.G.3.3 for discussion of displacement checks and sample calculations for this Sample Problem.

5.G.4.2.9 Step 5 (LFD) – Pile Connection Design

Design calculations are completed for the detail shown in Figure 5-19 for connection of the pile top to the abutment footing.

Required Design Strength, Dimensions

Required strength, tension	$P_{t-required} = 56 \text{ kN}$	Group VII (seismic)
Required strength, compression	$P_{c-required} = 907 \text{ kN}$	Group I (non seismic)
Abutment concrete compressive strength	$f_c' = 27.6 \text{ MPa}$	
Casing outside diameter	$OD_{casing} = 141 \text{ mm}$	
Pile area	$Area_{pile} = \frac{\pi}{4} OD_{casing}^2 = 15{,}615 \text{ mm}^2$	

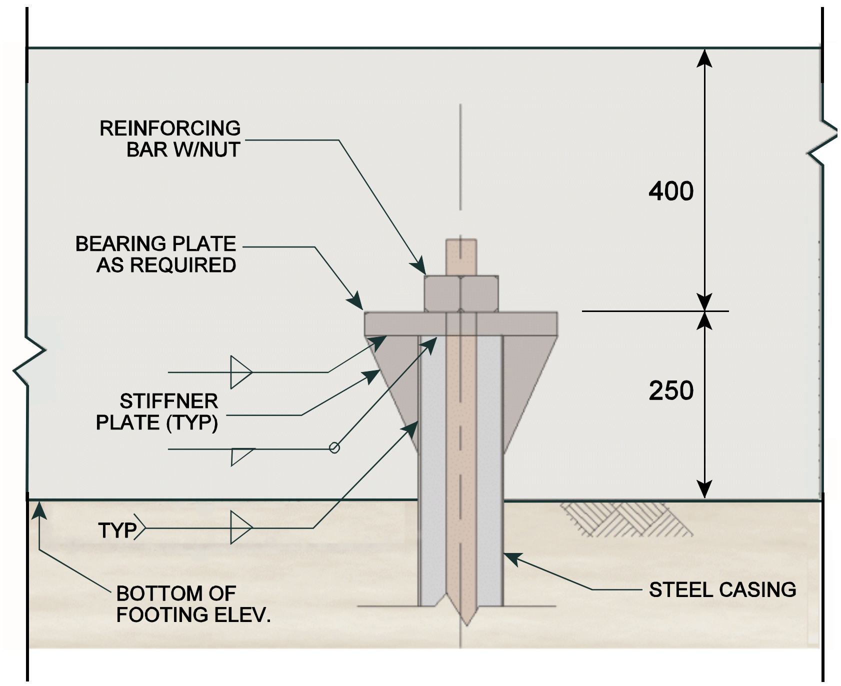
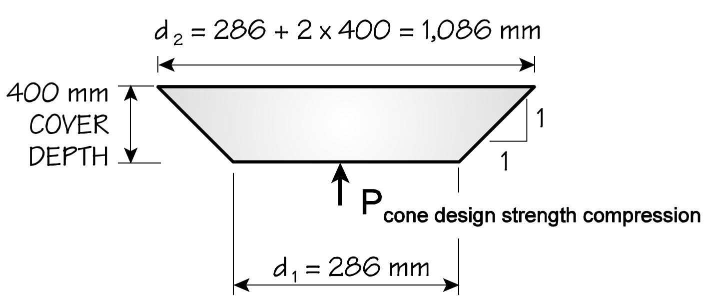


Figure 5 - 19. Pile Top to Abutment Footing Connection Detail.

Required Plate Area

Assume a 254 mm square plate- $\text{Area}_{plate} = \text{Plate}^2_{width} = 64{,}516 \text{ mm}^2$

Check cone shear for compression loads:



Equivalent diameter for 254 mm² plate

$$d_1 = \sqrt{\frac{4 \times 254^2}{\pi}} = 286\text{mm}$$

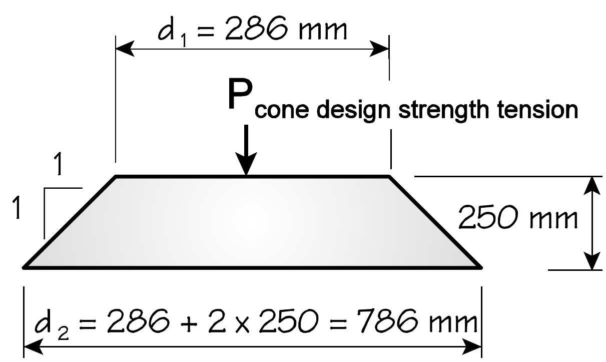
$$A_{CP} = \frac{\pi}{4}\left[d_2^2 - d_1^2\right] = 862{,}053 \text{ mm}^2$$

Use ACI 349, Appendix B

$$P_{\text{cone design strength compression}} = 10.5 \times (0.65) \sqrt{27.6 \times 1000} \times [A_{CP}] \times 10^{-6}$$

$$= 977 \text{ kN} > 907 \text{ kN} \ldots\ldots \text{ OK}$$

Check cone shear for tension loads:



$$A_{CP} = \frac{\pi}{4}\left[d_2^2 - d_1^2\right] = 536{,}000 \text{ mm}^2$$

$$P_{\text{cone design strength tension}} = 10.5 \times (0.65) \sqrt{27.6 \times 1000} \times [A_{CP}] \times 10^{-6}$$

$$= 608 \text{ kN} > 56 \text{ kN} \ldots\ldots \text{ OK}$$

Note - edge distance limitations and other requirements of ACI 349 Appendix B must be satisfied.

Required Plate Thickness

Actual bearing stress, tension $\text{Bearing}_{\text{tension}} = \dfrac{P_{t-\text{required}}}{\text{Area}_{\text{plate}} - \text{Area}_{\text{pile}}} = 1.15 \text{ MPa}$

Actual bearing stress, compression $\quad \text{Bearing}_{\text{compression}} = \dfrac{P_{\text{c-required}}}{\text{Area}_{\text{plate}}} = 14.06 \text{ MPa}$

Plate bending moment for 1 cm width

$$M_{\max} = 10 \text{ mm} \times \frac{1}{2}\left(\frac{\text{Plate}_{\text{width}} - \text{OD}_{\text{casing}}}{2}\right)^2 \times \text{Bearing}_{\text{compression}} = 0.224 \text{ kNm}$$

Note - use of this cantilever moment was determined to be a conservative approximation of the maximum moment in the plate due to a compression pile load by comparison to a more accurate formula shown in “Formulas for Stress and Strain,” 5th Edition, 1975, by Roark and Young. Plate bending from a tension pile load or with different plate support details must be further analyzed.

Design bending stress $F_y = 345 \text{ MPa}$

$$S_{\text{x-req}} = \frac{M_{\max}}{F_y} = 649.3 \text{ mm}^3$$

Required plate thickness $\quad t_{\text{req}} = \sqrt{\dfrac{6\, S_{\text{x-req}}}{10 \text{ mm}}} = 19.7 \text{ mm}$

Use 254 mm square x 25.4 mm thick top plate.

Required Weld Size:

Tensile strength of E70 electrode $Fu_{weld} = 483$ MPa

Minimum tensile strength of connected parts (ASTM A53 Grade B Casing)

$Fu_{part} = 414$ MPa

Fillet weld strength $\Phi F = 0.45\,Fu_{part} = 186\ \text{MPa}$

Top weld size $t_{weld-top} = 6.35\ \text{mm}$

Stiffener plate size $t_{stiff} = 12.7\,\text{mm}$ $W_{stiff} = 100\ \text{mm}$ $L_{stiff} = 150\ \text{mm}$

Top weld length $L_{weld} = \pi\,OD_{casing} - 4\,t_{stiff} + 8\,W_{stiff} = 1{,}190\ \text{mm}$

Top weld strength

$$P_{weld-top} = 0.707\,t_{weld-top}\,\Phi F\,L_{weld} = 995\ \text{kN} > P_{c-required}\ .\ .\ .\ .\ \textbf{OK}$$

Stiffener plate side weld dimensions $t_{weld-side} = 6.35\ \text{mm}$

$L_{weld} = 8\,L_{stiff} = 1{,}200\ \text{mm}$

Stiffener plate side weld strength -

$$P_{weld-side} = 0.707\,t_{weld-side}\,\Phi F_{weld} = 1{,}004\ \text{kN}$$

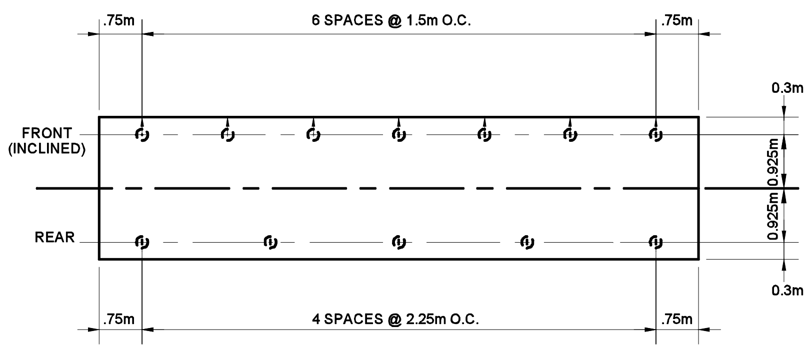
$$> 0.707\,t_{weld-top}\,\Phi F \times 8\,W_{stiff} = 669\ \text{kN}\ .\ .\ .\ \text{OK}$$

Use 6.35 mm fillet weld for welding top and stiffener plates.

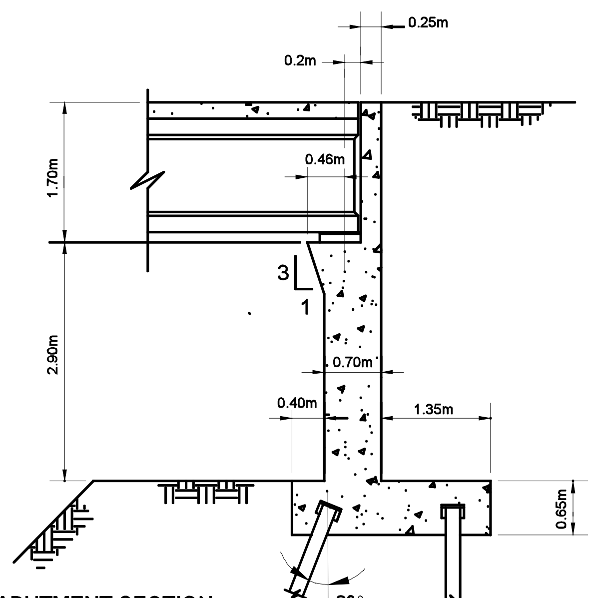
Note - this example ignored bending stresses on the welds which is considered appropriate for a compression pile load with the weld details shown. Weld bending stress must be analyzed for other conditions.

5.G.5 Step 6 (SLD & LFD) – Complete Detail Drawings

The following detail drawings (Figures 5-20 and 5-21) show the pile design determined by the Service Load and Load Factor design methods in the previous sections. (The two methods yielded the same design in this case.)



ABUTMENT FOOTING – PLAN VIEW
NOT TO SCALE



ABUTMENT SECTION
NOT TO SCALE

GENERAL NOTES

MICROPILE INSTALLATION PROCEDURE

1. Secure area for work and survey locations for piling installation.
2. Advance 141 mm outside diameter casing to full pile depth required (7.5 meters minimum into dense gravel soils or 11.2 meters total below bottom of footing whichever is greater) utilizing rotary drilling techniques.
3. Tremie casing full with neat cement grout
4. Place 43 mm reinforcing threadbar with centralizers
5. Reattach drill head to the top of the casing and pressure grout the 7.5 meter-long pile bond length by pumping neat cement grout under pressure while extracting casing. Minimum grout pressure should be 0.35 MPa.
6. Upon completion of pressure grouting, reinsert the casing 1.5 meters into the top of the bond length.
7. Trim top of casing to its proper elevation, and weld the top bearing plate with stiffener plates.
8. The quality of the grout shall be monitored by collecting grout cubes for later compression testing and by measuring the grout specific gravity from one batch per day.
9. Consistency of pile installation shall be monitored and recorded as described in the pile installation quality control document. Monitored and recorded data shall include total pile depth, grout pressures and quantities, soils /rock encountered during installation and any obstructions or irregularities.

PILE LOAD TESTING

1. The pile load test program shall be conducted as described in the specifications. Testing procedures and results will be inspected and reviewed by DOT representative, and are subject to DOT approval. An expeditious response to the load test submittal is needed from DOT so as not to delay the progress of the Contractor.

MATERIAL SPECIFICATIONS

Grout – A neat mix of Portland Cement (Type I / II) conforming to ASTM C150 with a water cement ratio of approximately 0.45. The minimum 28 day compressive strength of the grout shall be 34.5 MPa.

Reinforcing Bar – The reinforcing bar shall be a 43 mm Grade 520 Dywidag Threadbar (or equivalent) conforming to ASTM A – 615 (F_y = 520 MPa). Length of couple bar sections shall be determined based on the overhead clearance available at each pile location.

Bearing Plate – Steel for the top bearing plate with side stiffeners shall conform to AASHTO M270 Grade 350 (F_y = 345 MPa).

Casing – The steel casing shall be 141 mm outside diameter, 9.5mm wall thickness conforming to ASTM designation: A106 Grade B, A252 Grade 2, A519 with a minimum yield strength of 241 MPa, or A53 Grade B.

CONTRACTOR DESIGN / BUILD
WORKING DRAWING SUBMITTAL
CHAPTER 5, ABUTMENT MICROPILES
SAMPLE PROBLEM #1

Figure 5 - 20. Page 1 of Sample Problem No. 1 – Working Drawing Submittal (SLD & LFD Designs)

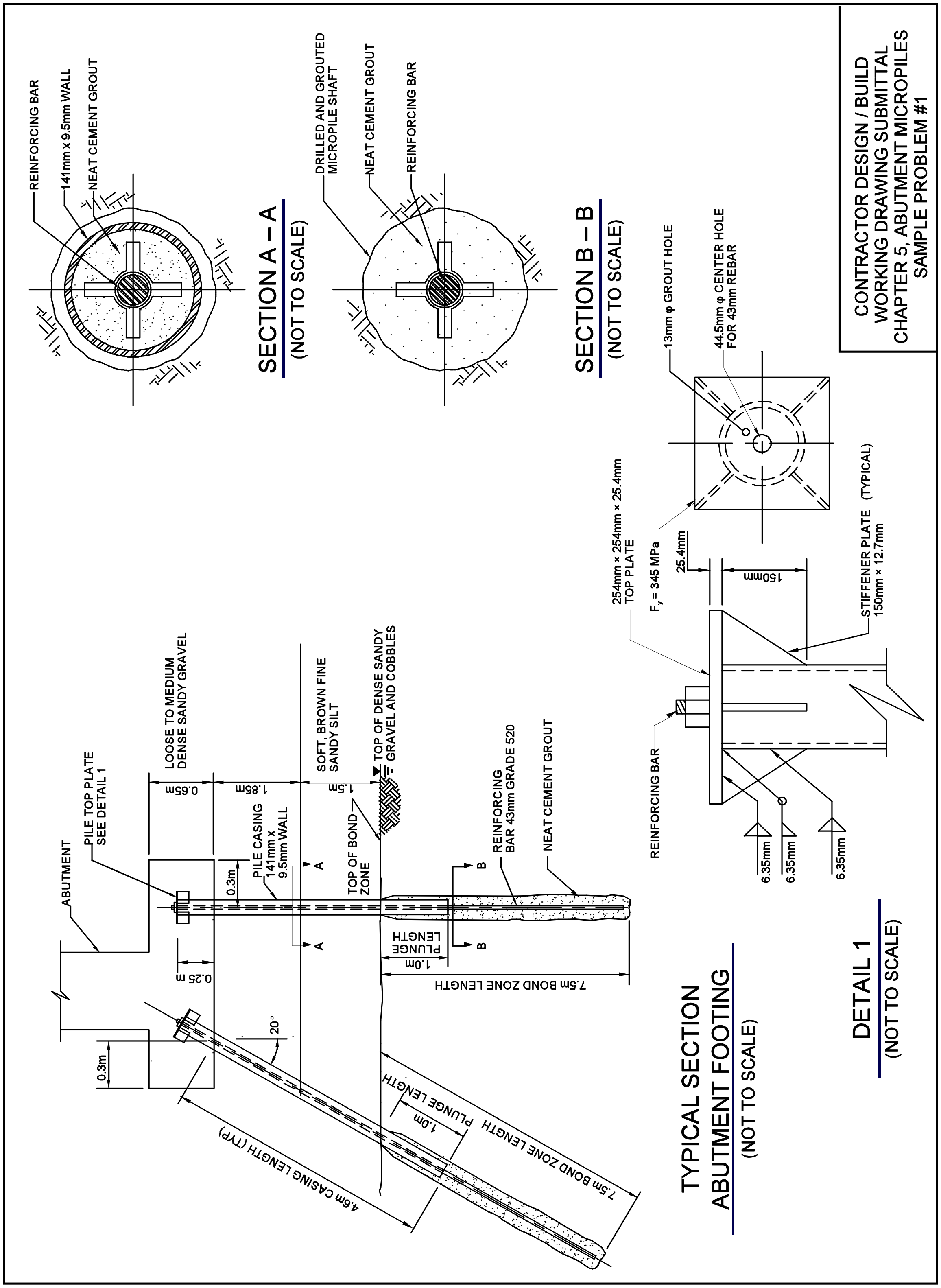


Figure 5 - 21. Page 2 of Sample Problem No. 1 – Working Drawing Submittal (SLD & LFD Designs)

CHAPTER 6

DESIGN METHODOLOGY: MICROPILES FOR SLOPE STABILIZATION AND EARTH RETENTION

6.A SPECIAL NOTE

Chapter 6 was intended to cover slope stabilization applications. Due to a lack of consensus on design procedures, Chapter 6 is still under preparation. When completed, Chapter 6 will be made available as a supplement to this manual.

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CHAPTER 7

PILE LOAD TESTING

7.A INTRODUCTION

It is industry standard, as a first order of work on micropile foundation support projects, to perform load testing of at least one micropile to the ultimate design load in order to verify the design assumptions and the adequacy of the contractor's installation methods. Load testing may consist of installation and verification load testing of one or more piles prior to commencement of production pile installation. Additional confirmation of the pile load capacity may be obtained through (proof) testing on production piles during the course of construction.

Micropiles are tested by the static axial load testing of individual piles. These tests usually feature incremental axial loading until the pile either sustains a predetermined maximum test load, reaches a predetermined structural axial displacement limit, or reaches a predetermined ground creep threshold. With the trend towards higher capacity CASE 1 piles, failure may occur in the form of the sudden loss of load and increase in displacement associated with structural failure. This aspect requires careful consideration in design of higher capacity micropiles (see Chapter 5).

In this manual, the close link between micropiles, ground anchors and soil nails has been noted regarding installation methods and geotechnical performance. Please refer to the more detailed discussion in Chapter 5, section 5.E.4. This link is also reflected in micropile load-testing methods and acceptance criteria. Many references address the load testing of driven piles, drilled piers, ground anchors and soil nails, but none specifically address micropiles. The load-testing procedures presented in this section conform to the requirements of ASTM D 1143 and D 3689 for testing individual piles under static axial compression and tension load, with modifications that reflect micropile testing practices.

7.B TYPES AND PURPOSE OF LOAD TEST

Micropiles are field tested to verify that the micropile design loads can be carried without excessive movements and with an adequate factor of safety for the service life of the structure. In addition, testing is used to verify the adequacy of the contractor's drilling, installation and grouting operations prior to and during construction of production piles. Therefore, the soil/rock conditions, as well as the method, equipment and operator used for installing production piles <u>must</u> be similar to those used for installing test piles. If ground and/or installation procedures change, additional testing may be required. If test results indicate faulty construction practice, or grout-to-ground load capacities less than required, the contractor is required to alter the micropile installation/construction methods. In the event that the required design grout-to-ground bond capacities are still not achievable, redesign may be necessary.

Testing criteria will be part of the specifications and may include *ultimate* and/or *verification* tests which are conducted to validate the contractors installation methods and to verify compliance with the micropile load carrying capacity and grout-to-ground bond values used in design. These tests usually require loading to a maximum test load that includes the factor of safety assigned to the design grout-to-ground bond and/or that which results in failure (i.e. inability to maintain constant test load without excessive micropile movement). The number of tests will vary, depending on the size of project and the major different ground types in which micropiles will be installed. On smaller projects, one or two ultimate, or verification, tests are commonly conducted prior to beginning production pile installation, and then one or more additional such tests may be conducted in each major different ground type encountered as construction proceeds. A larger number of ultimate, or verification, tests may be specified for larger projects. *Ultimate and verification tests are typically performed on "sacrificial" test piles.*

During production installation, "proof" testing may be conducted on a specified percentage of the total production piles installed. Some specifications allow proof testing to be performed on production piles that will be incorporated into the structure while other specifications require the proof-tested piles to be extra “sacrificial” piles that will not be incorporated into the structure. For economy, the first approach is strongly recommended.

Creep tests are typically performed as part of ultimate, verification, and proof tests. Creep is a time dependent deformation of the soil structure under a sustained loading. Creep is primarily of concern in organic and cohesive (clayey) soils. The creep test consists of measuring the movement of the pile at constant load over a specified period of time. This test is done to ensure that the pile design loads can be safely carried throughout the structure’s service life (typically 75 to 100 years) without causing movements that could damage the structure.

Micropile testing is conducted by incrementally loading (and if specified, unloading) the pile and measuring the movement of the pile head at each load increment. Typically, the pile-head movement reading is recorded just after the next load increment has been applied. The loading increments, the time that each load increment is held and the number of measurements for each load increment are determined by the type of test being performed and will be specified in the contract documents. If not specified, recommended practice is to obtain a pile-head movement reading just after the load has been applied, and a second reading after the load has been maintained for a sufficient amount of time to ensure that pile-head movement has stabilized.

Testing procedures were not “standardized” at the time this manual was written and vary among different Highway Agencies. Check specifications for test procedures applicable to your projects.

Most micropile load tests completed to date have been performed in accordance with ASTN D1143 “Quick” test procedures.

7.B.1 Ultimate Test

Ultimate tests (if used) are performed on non-production, “sacrificial” micropiles and provide the following information:

- Determination of the ultimate grout-to-ground bond capacity (if carried to failure).

- Verification of the design grout-to-ground bond factor of safety.
- Determination of the load at which excessive creep occurs.

A true “ultimate” test is performed by loading the micropile until failure takes place along the grout-ground interface. Failure is the inability to maintain constant test load without excessive movement. Excessive movement is often taken as the slope of the load-deflection curve exceeding 0.15 mm/kN. True ultimate tests, taken to failure, are usually only specified as part of a research project or on very large projects where a design phase test program can be justified. The design phase test program will allow the micropile design to be optimized.

7.B.2 Verification Test

Verification tests are conducted to verify that installation methods will provide a micropile capable of achieving the specified grout-to-ground bond capacity with a specified factor of safety. Verification maximum test loading will be defined by the grout-to-ground bond factor of safety and the chosen design grout-to-ground bond capacity. If the design grout-to-ground bond factor of safety is 2.5, the maximum test load will verify 250 percent of the design bond value. Verification tests are generally completed on non-production, “sacrificial” micropiles as a first order of work prior to construction of production piles. In addition, “verification” testing may be required during production to verify capacities for different soil/rock conditions and/or drilling/installation methods. Verification tests may or may not test the micropile to the point of failure. The test is a rigorous multiple cycle test with the test load progressively increased during each loading cycle until the final specified maximum test load is reached.

7.B.3 Proof Test

A proof test is typically performed on a specified number of the total number of production micropiles installed. The test is usually a single cycle test in which the load is applied in increments until a maximum test load, recommended to be 167 percent, or more, of the design grout-to-ground bond value is reached. Proof tests provide information necessary to evaluate the ability of production micropiles to safely withstand in-service design loads without excessive structural movement or long-term creep over the structure’s service life.

7.B.4 Creep Test

If micropiles are to be bonded in creep susceptible cohesive soil, creep tests are typically performed as part of the ultimate, verification, or proof test. Creep testing is conducted at a specified, constant test load, with movement recorded at specified time intervals. The deflection versus log-time results are plotted on a semi-log graph, and are compared with the acceptance criteria presented in the contract documents. A maximum creep rate of 2 mm per log cycle of time is a common acceptance criteria. Creep tests should utilize a calibrated load cell during the creep test load hold increment to monitor and adjust for small changes in load caused by jack-bleed, ram friction, or other factors.

7.C DETERMINATION OF PROJECT LOAD TESTING REQUIREMENTS

Load testing of foundation support elements confirms the bond capacities of the underlying soils, pile structural design capacities, and the movement characteristics of the micropile itself. Procedures used for installation of successfully tested pre-production piles establish the procedures to be used for installation of the production piles. The factors to be considered when determining the project testing requirements include the following:

1. Total number of production micropiles.
2. Magnitude and type of design loading.
3. Sensitivity and importance of the supported structure.
4. Variance in ground subsurface conditions across the installation site.
5. Types of subsurface conditions.
6. Site access and headroom/installation constraints.
7. Contractor experience.

A project specification for foundation support elements should include requirements for the following components of a load test program. These requirements are discussed in detail below, and include:

- The number of pre-production verification load tests.
- The number of production pile proof load tests.
- The magnitude of the test loads.
- The method of load application (single vs. multiple cycle, order of testing).
- The duration for which the test loads are applied.
- Acceptance criteria for maximum total displacement and maximum creep displacement at a specified load.

7.C.1 Number of Load Tests

For a smaller micropiling project, the load testing required will usually consist of testing at least one pre-production verification pile to some factor times the design loading. This pile may be a sacrificial (non-production) pile or a production pile. Loading beyond the required minimum capacities may be conducted in an attempt to determine the pile's ultimate capacity. For large projects, additional pre-production tests and production pile proof testing will usually be required. For example, load testing specified for Caltrans seismic retrofit micropiles typically consists of at least one pre-construction performance (verification) test on a non-production pile

For a specific large commercial project involving strengthening the foundations under existing grain silos with 800 high-capacity micropiles, the required load testing initially included three successful sacrificial preproduction tests and 12 production pile tests, all to 200 percent of the service design compression loading. The required number of production load tests was later reduced to six after consistent and successful performance of the piles tested.

Testing requirements can add considerable cost to a project. Sacrificial pile compression tests, including reaction piles, can add between $10,000 and $30,000 per test to the project cost.

Production pile tests that do not involve high-capacity ultimate testing and where adjacent piles may be used to provide reaction to compression test loads will add between $2,000 to $10,000 per test. Considering these costs, efforts should be made to provide a load-test specification that ensures that all project piles have adequate capacity while keeping total testing costs to a reasonable amount.

The magnitude of the maximum test load may be reduced for production pile proof testing, therefore reducing the cost. Performance at the lower test loads can be compared to results of the pre-production verification testing. Alternatively, production pile testing may consist of tension only tests, which are less expensive because the ground may be used for reaction to the test load rather than adjacent piles or reaction tie-down anchors. This is reasonable if the pile's critical load condition is in tension, or if results of the production tension testing can be correlated to the pre-production tension and compression testing results.

For a project involving a very small number of piles, load testing may be foregone if the structural and geotechnical design capacities provide sufficient redundancy (factor of safety) over the required loading. Some verification of the pile capacity could be provided from results on previous projects completed in similar conditions. The functional importance of the supported structure should also be considered.

7.C.2 Micropile Load-Testing Guidelines

7.C.2.1 Micropile Load-Testing Guidelines for New Users (Owners/Contractors)

The following are suggested guidelines to establish micropile load-testing requirements for a structural foundation project. **These guidelines are based on the following assumptions:**

- Micropiles are required for foundation support of bridge structures. (e.g., critical structures).
- Project specifications require pre-qualified micropile specialty contractors.
- Micropile proof load testing is in critical loading direction only.

Table 7-1. Suggested Micropile Load Testing Guidelines

Number of Piles Proposed for Project	Number of Tests	
	Pre-Production Pile **Verification** Testing	Production Pile **Proof** Testing
1 – 249	1	5 %
250 – 499	2	5 %
more than 500	3	5 %

Additional factors to be considered when determining a project's pre-production bond testing requirements are listed in Table 7-2. To establish the testing requirements for each project, multiply the assigned testing amplification factor for each category by the number of above referenced number of tests (i.e. from Table 7-1) according to size of project. The final verification testing requirements will be the summation of the number of pile tests required by Table 7-1 added to the number required by Table 7-2 based on the size of the project and each of the applicable pile-testing factors. For economy, round *down* to the nearest whole number of load tests when computing the required number of verification and proof tests.

Verification tests may be performed on production piles provided that:

- the piles are designed with a structural factor of safety of at least 1.25 at maximum test load,
- the piles are not failed or overloaded during testing, and
- the pile can be replaced if the pile fails.

Previous tests by the Owner/Contractor on Micropiles may be used to reduce the number of verification and proof tests provided the previous tests had:

- similar ground conditions
- similar construction methods (drilling and grouting)
- similar design loads and/or design bond stresses

Table 7-2. Amplification Factors for Micropile **Verification** Load Testing

Pile Load-Testing Factor		Amplificatio n
1. Criticality of Structure Supported		
Lifeline structure*		2.0
Non-lifeline structure		0.0
2. Micropile Ultimate (Nominal) Capacity		
1 – 1299 kN		0.0
>1300 kN		0.5
3. Subsurface Material Type (Pile Bond Zone)		
Rock		0.0
Soil	sands, gravels	0.0
. . .	clays, silts	0.5
4. Variance in Subsurface Conditions		
Little to no variance		0.0
Mild variance		0.5
Substantial variance		1.0

*Lifeline structure is a structure on a designated lifeline transportation route which must remain in service after an earthquake or other extreme design event.

7.C.2.2 Micropile Load-Testing Guidelines for Experienced Users (Owners/Contractors)

Many projects have been done with less testing than presented in Section 7.C.2.1. Reference [26] by Pearlman, et al., presents a summary of bridge projects and number of load tests performed. This list shows that:

- Typically only 1 or 2 pile load tests have been performed, especially for piles founded in rock.
- More tests have been performed on larger jobs with variable ground (e.g., Williamsburg Bridge and Brooklyn-Queens Expressway)

For experienced users and/or contractors, fewer tests than suggested in Section 7.C.2.1 may be appropriate and should be evaluated on a project-by-project basis.

Verification tests may be performed on production piles provided that:

- the piles are designed with a structural factor of safety of at least 1.25 at maximum test load,
- the piles are not failed or overloaded during testing, and
- the pile can be replaced if the pile fails.

Previous tests by the Owner/Contractor on Micropiles may be used to reduce the number of verification and proof tests provided the previous tests had:

- similar ground conditions
- similar construction methods (drilling and grouting)
- similar design loads and/or design bond stresses

7.C.3 Sample Problem No. 1 (Chapter 5) – Pile Load Testing Requirements

Given: Single-span bridge (not lifeline structure) located in mountainous terrain approximately 125 km northwest of Denver, Colorado. Each abutment is to be supported on 12 micropiles (a total of 24), approximately 12 meters in length. The controlling axial design load for the pile is 595 kN in compression Access to each abutment work bench must be established. Ground conditions are as shown in the Chapter 5 example problem.

Table 7-3 summarizes the verification testing multipliers that have been selected for this example.

Table 7-3. Sample Problem No. 1 – Pile Testing Requirements

Load Test Factor	Amplification Factor	Number of Verification Tests
24 Micropiles (from Table 7-1)	n/a	1
Non-lifeline Structure	0.0	1 × 0.0 = 0
1,500 kN (ultimate capacity)	0.5	1 × 0.5 = 0.5
Sands and Gravels	0.0	1 × 0.0 = 0
Little Variance in Subsurface Conditions	0.0	1 × 0.0 = 0
	Total:	**1.5**

Therefore, for this project *one* initial verification test is recommended on a non-production sacrificial test pile, to verify the contractor's design and installation procedure. Also, *one* proof test (5% of 24) will be required on the production piles. The verification test can be conducted at abutment one and the proof test at abutment two. Test in compression to a maximum test load of 1,500 kN (2.5 × DL) for the verification test pile and 1,000 kN (1.67 ×DL) for the proof test pile.

7.C.4 Test Load Magnitude

The factor applied to determine the test-load magnitude is typically controlled by the geotechnical factor of safety desired. This geotechnical factor is usually greater than factors applied for ultimate structural capacity considerations. Therefore, excess structural capacity may be required for the pile to support the test loads based on geotechnical capacities.

For magnitude of test loading, this manual recommends:

Verification load testing to . . . . . . 2.5 × Design Load, and

Proof load testing to . . . . 1.67 × Design Load.

Refer to Section 5.E.4 for a detailed discussion of the rationale for these recommended test loads.

For ground-anchor testing tension, extra reinforcement is sometimes added to the initial verification test anchors, saving cost on the less-reinforced production anchors. It is much more difficult to correlate the test results of piles with differing reinforcement, particularly for composite-reinforced piles acting in compression. Also, a pile's geotechnical capacity can have an effect on the ultimate structural capacity, calling for the same factor for both geotechnical and structural considerations. Whether or not the reinforcement can be varied for the test piles should be considered carefully, and should be addressed in the project specifications. Refer to Chapter 5 for more detailed coverage of this design aspect.

7.D MICROPILE LOAD TESTING METHODS AND PROCEDURES

7.D.1 Method of Load Application

If the micropiles are designed for tension and compression loads, then both loading conditions should be tested. If the same micropile is to be tested in both tension and compression, it is suggested that the tension test be conducted first. This will allow the pile to be reseated during compression testing in the event some net upward residual movement occurs during the tension test.

The method of applying the load can vary, either in one cycle, incrementally advancing to the required capacity, or in multiple cycles where the load increments are applied and removed gradually until the maximum load is attained. The use of multiple cycles may be preferable if an attempt is made to reach the ultimate capacity of the pile. As discussed in Chapter 5, the analysis of the elastic and residual displacements measured during cyclic loading will provide valuable insight into the pile performance and mode of failure, and the extra costs are minimal.

It is usually not necessary to conduct the load test on an inclined pile, even if the project includes them. Installing the pile on an incline has little, if any, effect on difficulty of construction and resulting capacity, particularly for piles installed by the cased hole method. However, testing an inclined pile can be difficult and increase the testing costs, particularly for compression testing.

7.D.2 Load-Hold Duration

The duration for which the applied loads are held (testing the tendency of a pile to creep) is another important consideration. If a pile is installed in non-creep-sensitive soils, such as sands, gravels, or rock, the maximum test load may be held for only ten minutes, with the hold duration extended if the acceptance criterion is not met. For piles in a creep-sensitive soil, such as plastic silt or clay, the maximum load hold duration may range from 100 minutes to as long as 24 hours, depending on the type and magnitude of design loading, nature of the soil, and sensitivity of the supported structure.

7.D.3 Load Test Acceptance Criteria

The magnitude and direction of load a pile must support is determined in the design of the structure. Maximum pile displacement is also determined during design of the structure, considering its sensitivity to movement, or considering the allowable displacements and footing rotations specified during seismic analysis. Criteria for allowable creep displacements can be based on standard criteria for ground-anchor testing (PTI, 1996) and soil nail testing (FHWA, 1996), which is based on an allowable displacement of 2 mm per log cycle of time in minutes.

Load test acceptance criteria is structure specific and typically includes the following:

- The verification test pile shall support a load in tension and/or compression equal to 250 percent of the specified service design loading (i.e. 2.5 × DL) without failure. The proof test pile shall support a load in tension and/or compression equal to 167 percent of the specified service design loading (i.e. 1.67 × DL) without failure. Pile failure is defined as continued pile top displacement without supporting an increase in applied load.

- The test pile shall support the service design load values with a total pile top displacement of not greater than _____ mm. For combined tension and compression testing, the total displacement shall be measured relative to the pile top position at the start of initial testing. *(**Commentary:** Structural designer should determine the* *maximum allowable total pile-head displacement based on structural design requirements. Refer to Chapter 5 for more detailed design guidance.)*

- While holding the applied test load at 133 percent of the design service load, increase in the pile top displacement (creep) measured between 1 minute and 10 minutes shall not exceed 1 mm. If the increase in displacement exceeds the criterion value, the load hold duration shall be extended to 60 minutes or longer. The creep rate between 6 and 60 minutes shall not exceed 2 mm per log cycle of time in minutes.

The load test acceptance criterion is structure specific and must be verified by the designer. Seismic retrofit applications may allow higher displacements for compatibility with the existing pile system.

7.E LOAD-TEST SETUP AND INSTRUMENTATION

For axial and lateral testing, the most common and convenient system includes a hydraulic jack and reaction arrangement. The reaction system may be adjacent micropiles, ground anchors, and/or footings or mats providing ground-bearing reaction for tension testing. Test setup arrangements are shown in Figures 7-1, 7-2, and 7-3 for compression, tension, and lateral testing. Additional test setups are shown in Photographs 7-1 through 7-3. The load is applied with the hydraulic jack, with the magnitude of the load determined through the correlation with the hydraulic pressure read from the gauge attached to the jack. This correlation is established through calibration of the jack and gauge, which is usually required within 90 days prior to the date of the load test. Figure 7-4 shows the load versus gauge pressure relationship established by jack calibration.

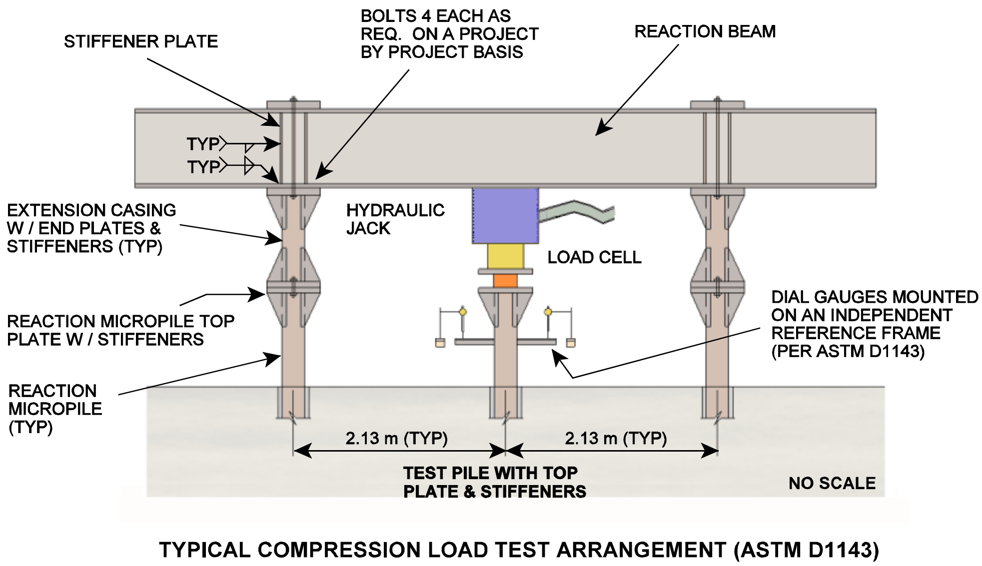


Figure 7 - 1. Compression Load Test Arrangement

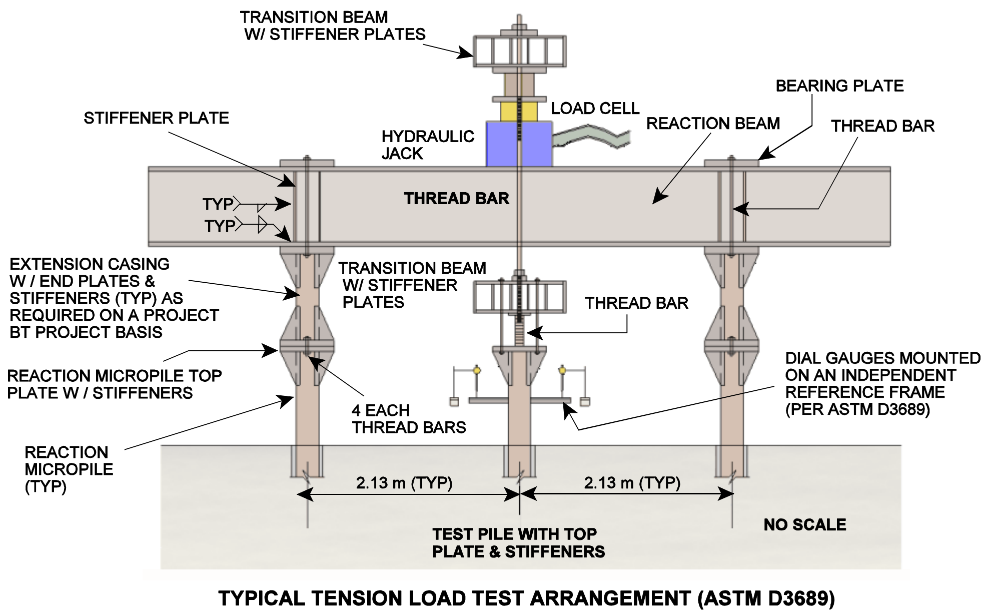
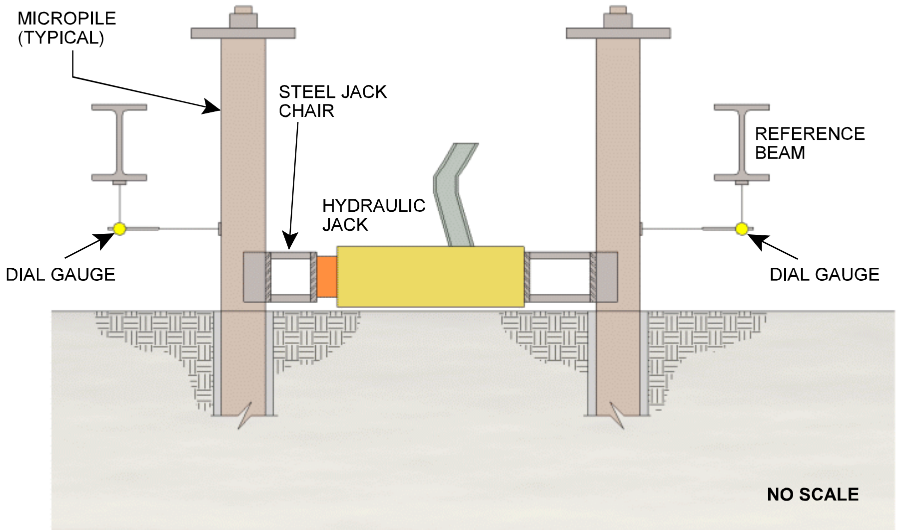
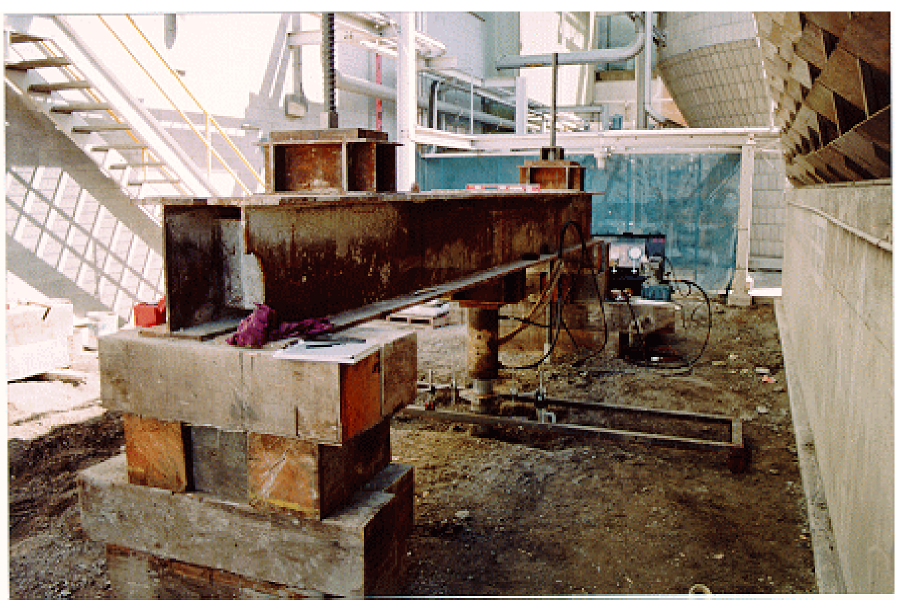


Figure 7 - 2. Tension Load Test Arrangement



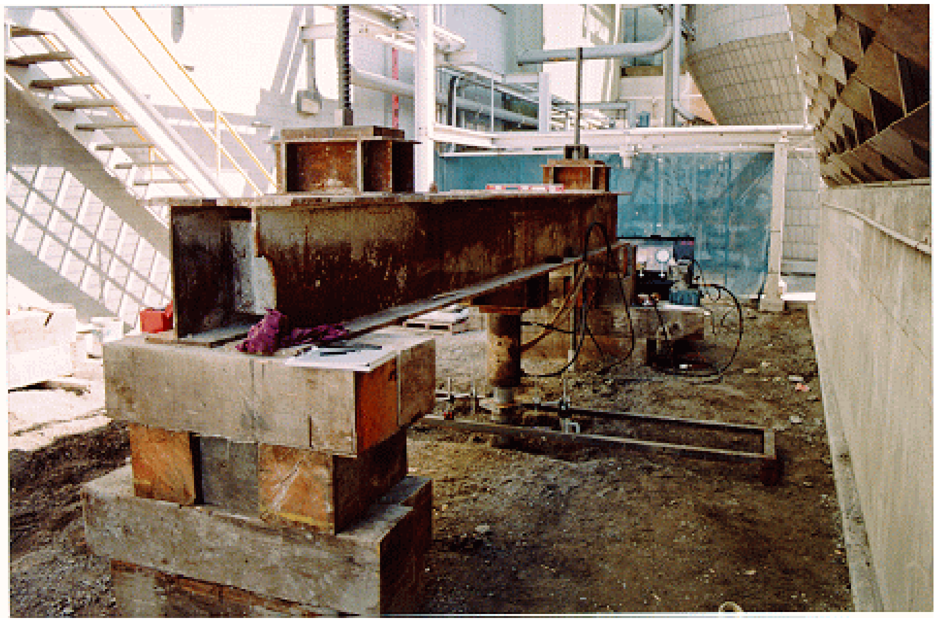
LATERAL LOAD TEST SETUP

Figure 7 - 3. Lateral Load Test Arrangement



Photograph 7 - 1. Compression Load Test Setup

Photograph 7 - 2. Tension Load Test Setup



Photograph 7 - 3. Typical Load Test Jack

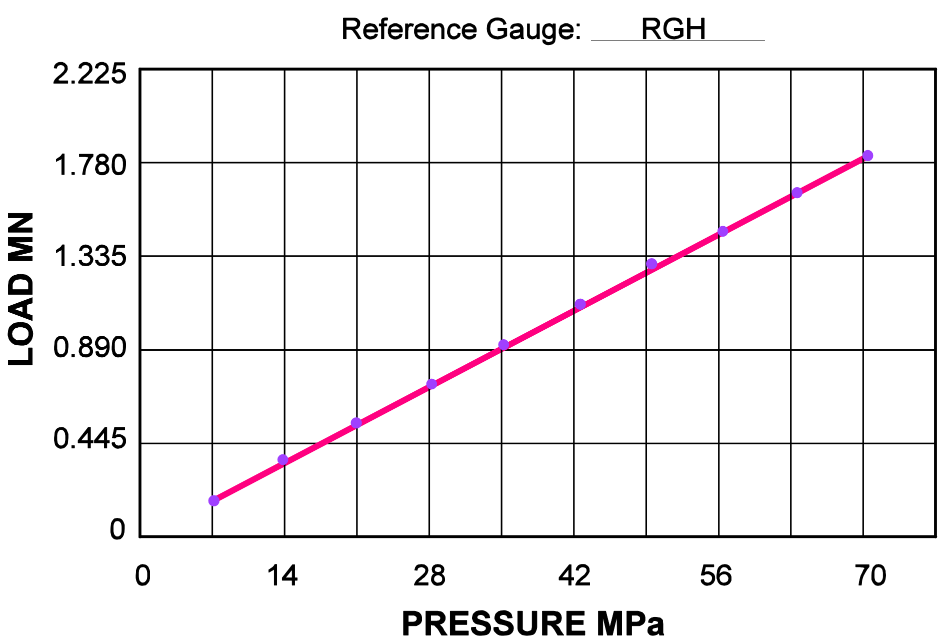


Figure 7 - 4. Typical Jack Calibration Curve.

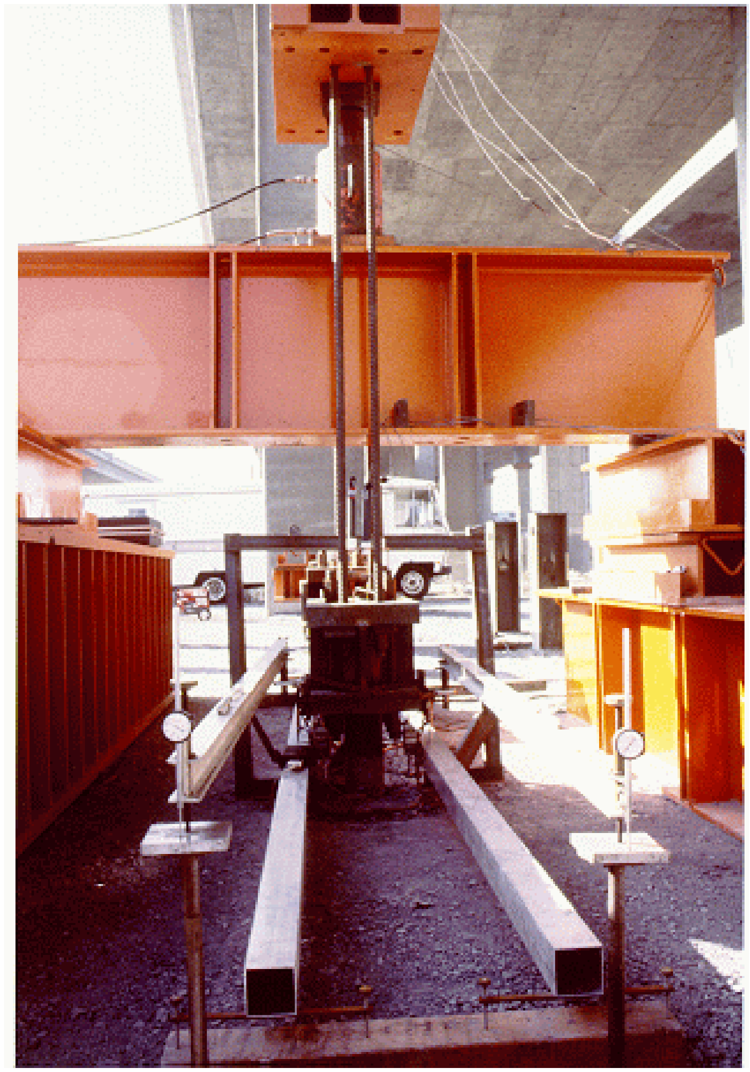
Instrumentation for recording pile performance may include the following (refer to Photographs 7-4 to 7-6):

- **Dial Gauges -** The vertical and lateral displacement is usually measured with dial gauges, with reading sensitivity to 0.025 mm. For an axial load test, the gauges are mounted on an independent reference beam whose supports are located a minimum of 2.13 meters from the test pile. The averaged readings of three gauges are used to compensate for tipping of the pile head. The gauges are placed around the pile at an equal distance from the pile center.

- **Wire with Mirror and Scale -** Pile top displacement may be determined using a mirror with a scale mounted on the pile, and a wire mounted on a separate reference strung in front of the scale. The scale is read by adjusting the line of sight until the wire lines up with its reflection. This method may be used as a backup to the dial gauge system, but it is less sensitive.

- **Survey Method -** The displacement of the pile may be gauged using survey instruments (level, theodolite). This method may be used as a backup to the dial gauge system, but it is less sensitive.

- **Load Cell -** Load cells may be used as an additional backup method of measuring the load applied to the pile. They can be used to more accurately maintain a constant test load throughout the creep-test load hold for verification test piles. When load cells are used, care should be taken to ensure that the cell is properly aligned with the axis of the micropile and jack. Load cells are used mainly to detect small changes in load and allow load adjustment and maintenance of constant holding load during creep testing. As an example, assuming that the load cell reads "440" once the creep test load is reached, it is important that the "440" reading on the load cell be maintained through jack pressure adjustments for the duration of the test. This provides assurance that a constant load was indeed maintained throughout the creep test.



Photograph 7 - 4. Micropile Load Test Instrumentation

Photograph 7 - 5. Micropile Load Test Instrumentation

Photograph 7 - 6. Micropile Load Test Instrumentation

- **Telltales -** Telltales are metal or fiberglass rods anchored into the pile grout at discrete elevations to measure axial displacement at these points in the pile. The rod tip is embedded into the grout at the point of interest, with the remainder sheathed to allow free movement of the pile. Movement at the top of the rod is measured with a dial gauge or electronic measuring device. Multiple telltales can be installed to measure movement at several points. Care must be taken to avoid damage to the telltales during pile installation, such as the rods becoming wrapped and damaged during drill string rotation, or the sheath becoming partially filled with grout.

- **Strain Gauges -** Strain gauges may be mounted on the reinforcing steel, allowing measurement of the level of stress in the reinforcement at various levels in the pile. Again, care must be taken to avoid damage to the gauges and connecting wires during installation. If used, vibrating-wire-type strain gauges are recommended.

- **Inclinometer -** An inclinometer may be installed in a pile to measure the deflected shape of the pile during lateral load testing.

7.F DATA RECORDING AND PRESENTATION

Shown in Tables 7-4, 7-5, and 7-6 are schedules for cyclic tension and compression tests. The schedules shown are arranged to show testing that may be appropriate for Sample Problem No. 1, included in Chapter 5. The figures that follow present data from completed micropile projects in several formats.

Figure 7-5 – Pile Top Displacement vs. Load Displacement data for axial and lateral tests can be presented as shown. The data shown is for a five-cycle compression test. The pile was well grouted into an upper layer of concrete rubble and dense sand, resulting in a very stiff pile.

Figure 7-6 – Pile Top Elastic/Permanent Displacement vs. Load The elastic and permanent displacement for a cyclic compression and tension test are shown. The permanent displacements are to the left, and the elastic to the right. The readings plotted are for the peak load of each cycle. The tension test included three cycles, and the compression test five.

Table 7-4. Example Tension Cyclic Load Test Schedule (Verification Test) for a 322 kN, Seismic Service Design Load

Initial Tension Test

Service Design Tension	Load Cycle	Tension Load (% Design)	Tension Load (kN)	Gage Pressure (MPa)	Hold Duration (minutes)	Dial Gage "1" Reading (mm)	Dial Gage "2" Reading (mm)	Dial Gage "3" Reading (mm)	Average Displacement
322 kN	TI-1	AL	45.0	2.64	0				
		25%	80.5	4.72	1				
					3				
					5				
		50%	161.0	9.44	1				
					3				
					5				
		AL	45	2.64	0				
					2				
	TI-2	25%	80.5	4.72	0				
					2				
		50%	161.0	9.44	0				
					2				
		75%	241.5	14.17	1				
					3				
					5				
		100%	322.0	18.89	1				
					3				
					5				
		AL	45	2.64	0				
					2				
	TI-3	25%	80.5	4.72	0				
					2				
		50%	161.0	9.44	0				
					2				

Table 7-4. . . . . . . continued

Initial Tension Test

Service Design Tension	Load Cycle	Tension Load (% Design)	Tension Load (kN)	Gage Pressure (Mpa)	Hold Duration (minutes)	Dial Gage "1" Reading (mm)	Dial Gage "2" Reading (mm)	Dial Gage "3" Reading (mm)	Average Displaceme nt
		75%	241.5	14.17	0				
					2				
		100%	322.0	18.89	0				
					2				
		125%	402.5	23.61	1				
					3				
					5				
		150%	483.0	28.34	1				
					3				
					5				
					6				
					10				
					15*				
					20*				
					25*				
					30*				
					45*				
					60*				
		AL	45.0	2.64	0				
					2				

*Note - Hold durations not necessary unless the displacement creep measured from 1 to 10 minutes exceeds 1 mm.

Table 7-5. Example Compression Cyclic Load Test Schedule (Verification Test) for the 721 kN Non-seismic Service Design Load

Initial Compression Test

Service Design Tension	Load Cycle	Tension Load (% Design)	Tension Load (kN)	Gage Pressure (MPa)	Hold Duration (minutes)	Dial Gage "1" Reading (mm)	Dial Gage "2" Reading (mm)	Dial Gage "3" Reading (mm)	Average Displacement
721	CI-1	AL	45.0	2.64	0				
		25%	180.3	10.57	1				
					3				
					5				
		50%	360.5	21.15	1				
					3				
					5				
		AL	45	2.64	0				
					2				
	CI-2	25%	180.3	10.57	0				
					2				
		50%	360.5	21.15	0				
					2				
		75%	540.8	31.72	1				
					3				
					5				
		100%	721	42.30	1				
					3				
					5				
		AL	45	2.64	0				
					2				
	CI-3	25%	180.3	10.57	0				
					2				
		50%	360.5	21.15	0				

Table 7-5. . . . . . Continued

Initial Compression Test

Service Design Tension	Load Cycle	Tension Load (% Design)	Tension Load (kN)	Gage Pressure (MPa)	Hold Duration (minutes)	Dial Gage "1" Reading	Dial Gage "2"	Dial Gage "3"	Average Displacement (mm)
					2				
		75%	540.8	31.72	0				
					2				
		100%	721	42.30	0				
					2				
		125%	901.3	52.87	1				
					3				
					5				
		150%	1081.5	63.45	1				
					3				
					5				
		AL	45	2.64	0				
					2				
	CI-4	25%	180.3	10.57	0				
					2				
		50%	360.5	21.15	0				
					2				
		75%	540.8	31.72	0				
					2				

Table 7-5. . . . . . Continued

Initial Compression Test

Service Design Tension	Load Cycle	Tension Load (% Design)	Tension Load (kN)	Gage Pressure (MPa)	Hold Duration (minutes)	Dial Gage "1"	Dial Gage "2"	Dial Gage "3"	Average Displacement
	CI-4	100%	721.0	42.30	0				
					2				
		125%	901.3	52.87	0				
		150%	1081.5	63.45	0				
					2				
		175%	1261.8	74.02	1				
					3				
					5				
		200%	1442	84.6	1				
					3				
					5				
					6				
					10*				
					15*				
					20*				
					60*				
		AL	45	2.64	0				
					2				

* Note - Hold durations not necessary unless the displacement creep measured from 1 to 10 minutes exceeds 1 mm.

Table 7-6. Example Ultimate Compression Load Test Schedule (Verification Test) for the 721 kN, Non-seismic Service Design Load

Ultimate Compression Test

Service Design Tension	Load Cycle	Tension Load (% Design)	Tension Load (kN)	Gage Pressure (MPa)	Hold Duration (minutes)	Dial Gage "1"	Dial Gage "2"	Dial Gage "3"	Average Displacement (mm)
721	C2-1	AL	45.0	2.64	0				
		25%	180.3	10.57	0				
					2				
					5				
		50%	360.5	21.15	0				
					2				
		75%	540.8	31.72	0				
					2				
		100%	721.0	42.30	0				
					2				
		125%	901.3	52.87	0				
					2				
		150%	1081.5	63.45	0				
					2				
		175%	1261.8	74.02	0				
					2				
		200%	1442	84.60	0				
					2				
		225%	1622.3	95.18	1				
					3				
					5				
		250%	1802.5	105.75	1				
					3				
					5				
					6				
					10				
		AL	45.0	2.64	0				
					2				

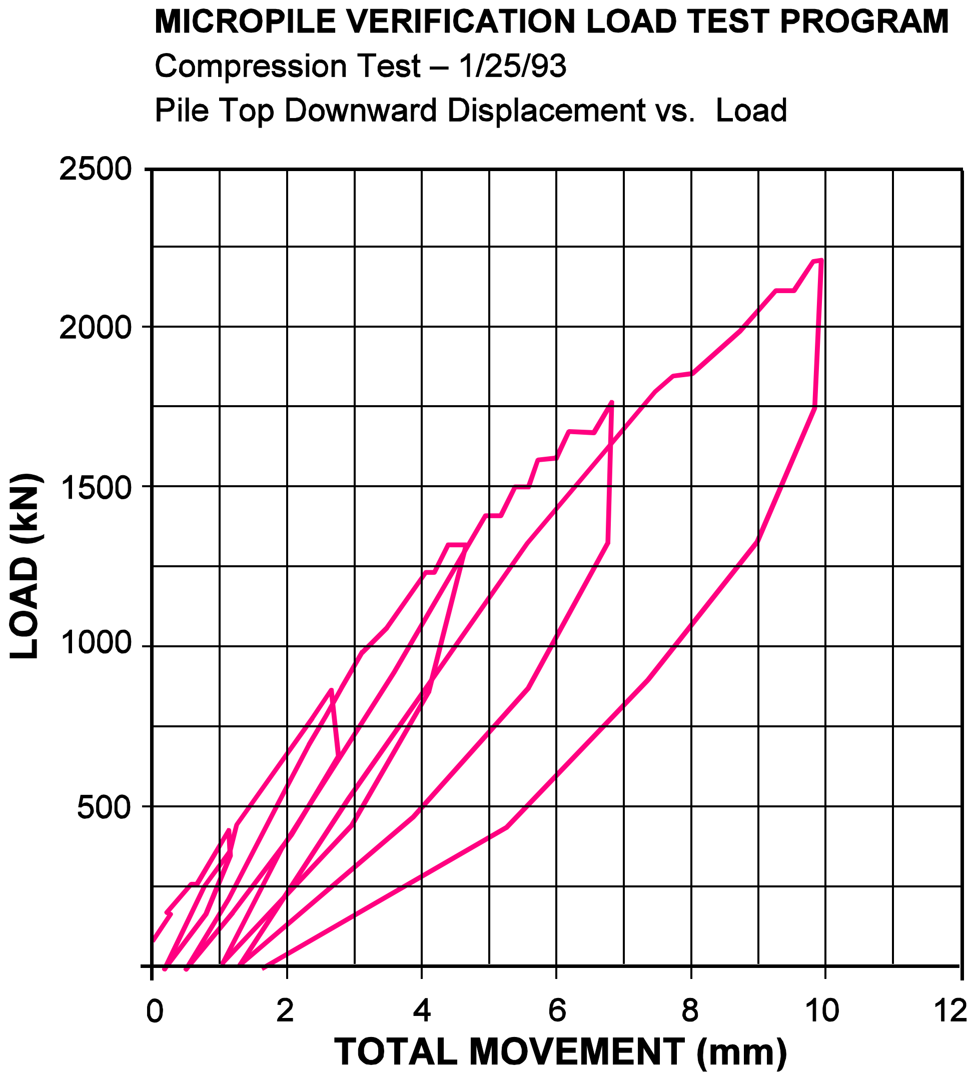


Figure 7 - 5. Pile top displacement vs. pile load curve

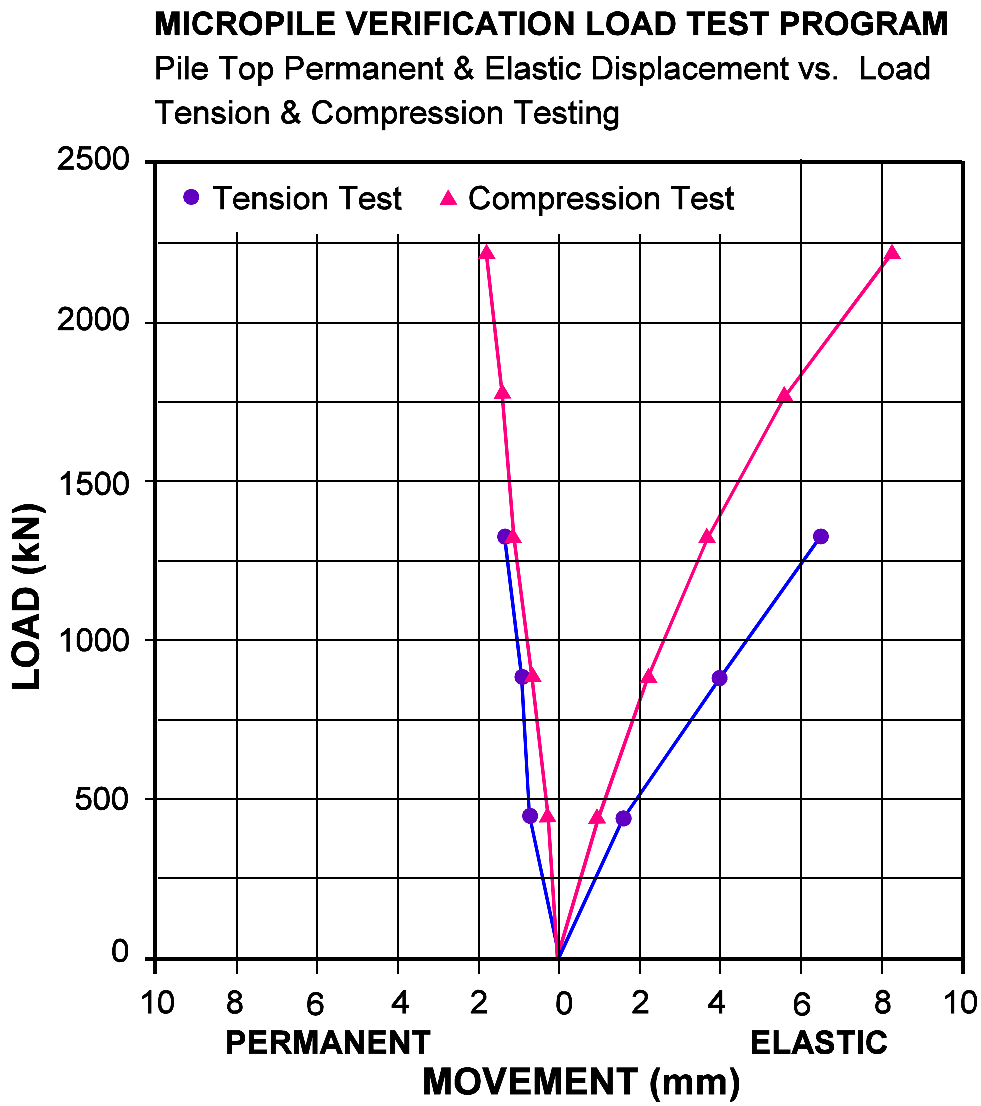


Figure 7 - 6. Pile Top Elastic/permanent Displacement Vs. Pile Load Curve

Figure 7-7 Displacement Creep vs. Time The increase in displacement (creep) at constant load versus time is shown in this figure, with the time shown on a logarithmic scale. The criterion line for creep of 2 mm per log cycle is also shown. The long duration tests shown were for high-capacity piles installed in medium-dense to dense sands and gravels.

Figure 7-8 – Elastic Length vs. Load The pile elastic length is determined based on the measured elastic displacement per load cycle and the calculated stiffness (EA) value. Again, note the stiffness of the pile due to it being well grouted into the upper layers.

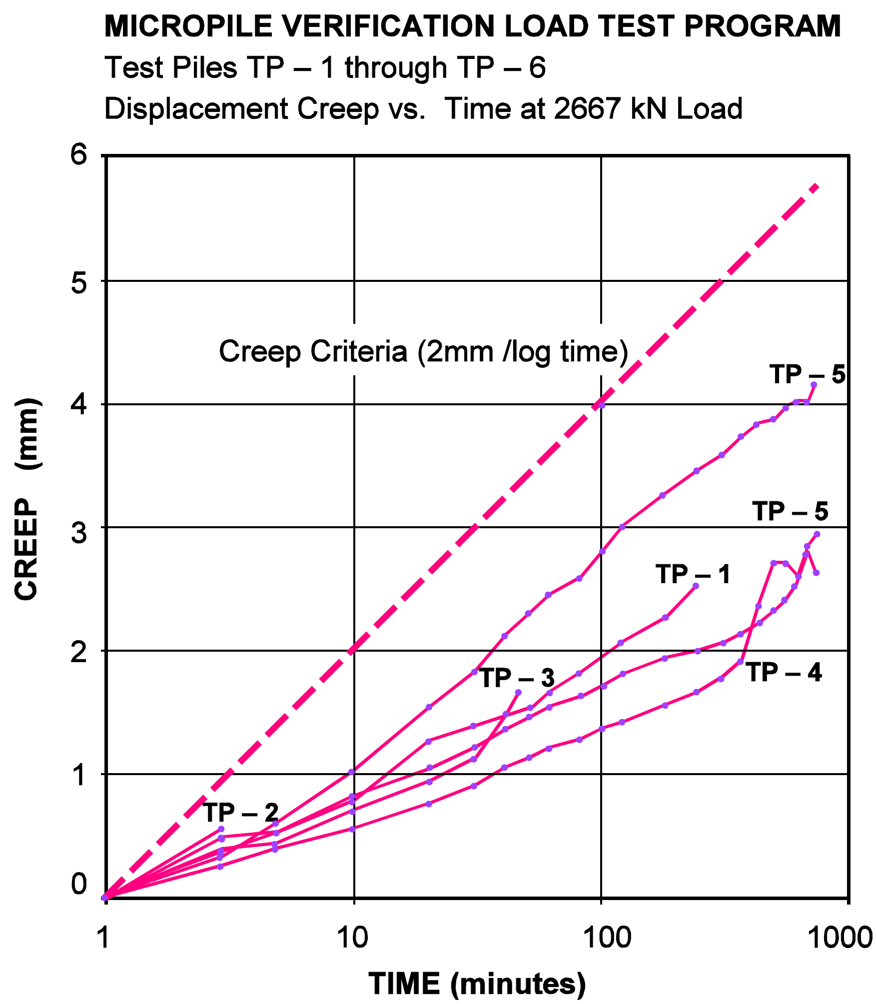


Figure 7 - 7. Displacement Creep Vs. Time Curve

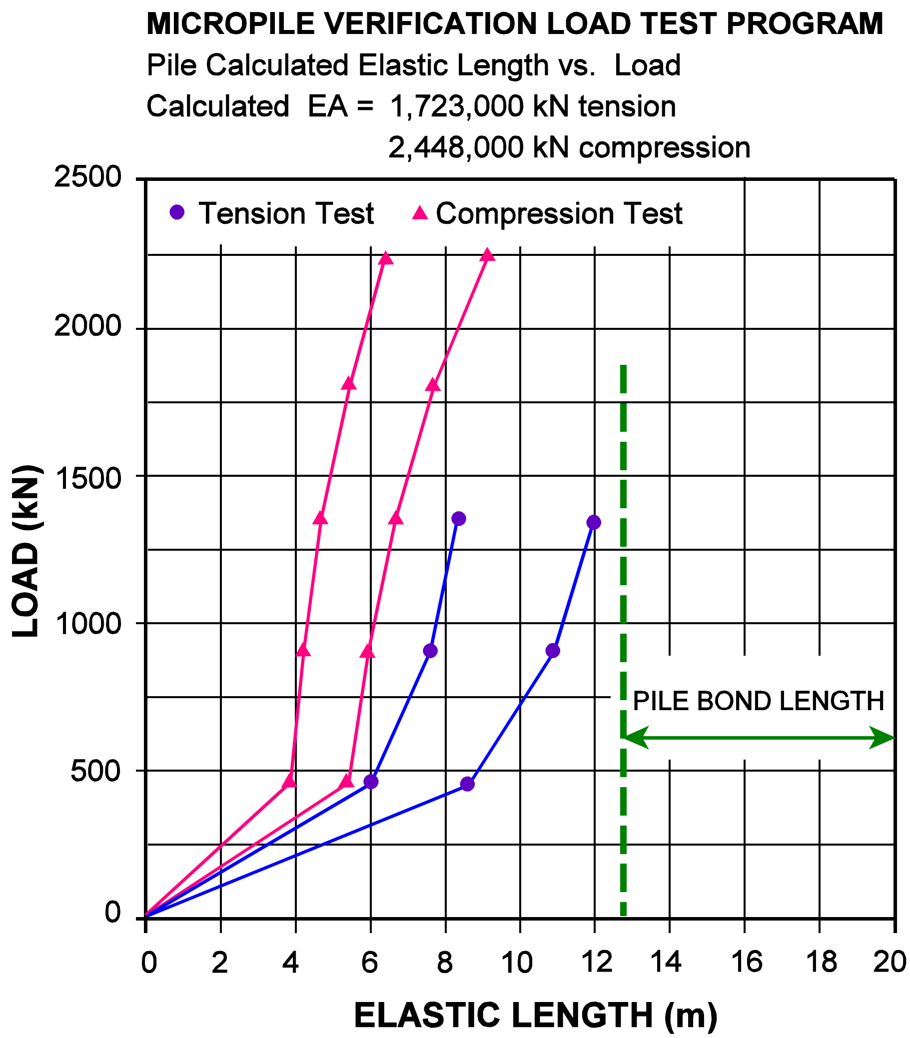


Figure 7 - 8. Elastic Length vs. Pile Load Curve

7.G PILE LOAD-TEST REPORT

For each load test, a report must typically be written and submitted to the Owner, usually within 24 to 48 hours of the load test completion. Suggestions for the contents of this report are as follows:

1. Brief project description.
2. Description of site and subsurface conditions.
3. Key personnel.
4. Pile installation data.
5. Results of load test, including data and data presentation.
6. Statement of load-test requirements and acceptance criteria.
7. Comparison of load-test requirements, acceptance criteria.
8. Summary statement on the load test results.
9. Hydraulic jack/load cell calibration report.
10. Material certification, including grout compressive strength testing, steel mill certification.

Pile installation data includes detailed information about the test micropile, such as length of the pile (cased and uncased), number of bags of cement, size and type of casing and reinforcement, material encountered during installation, and actual drilling and grouting records. The description of site and subsurface conditions restates the anticipated ground conditions at the location of the load test, and compares actual with foreseen conditions. Personnel listed should include the drill rig operator, the superintendent, the grout plant operator, and any other key personnel involved in the installation and testing of the micropile. Results of the load test should include the actual sheets used during the load-test as well as tabulated and graphical presentations of the load-test results. A summary of the load-test requirements and acceptance criteria, in addition to the load-test results, should also be included for comparison.

7.H TEST PILE FAILURE

In the event a micropile fails to meet the specified acceptance criteria during pre-production testing, the contractor can modify the design of the micropile and/or the construction procedure in order to provide the required capacity. These modifications may include modifying the installation procedures, increasing the pile bond length, or changing the micropile type. If failure occurs during load testing of a production pile, the contractor shall modify the design, the construction procedure, or both. These modifications may include installing replacement micropiles, modifying installation methods, increasing the bond length, or changing the micropile type. Additional testing may be required to verify adequacy of the piling system depending on the details of the failure, results of the previous testing, and specifics of the load capacity requirements.

CHAPTER 8

CONSTRUCTION INSPECTION/QUALITY CONTROL

8.A INTRODUCTION

A major consideration in promoting new construction technology is implementing and achieving quality assurance and control (QA/QC). The importance of this construction control is magnified with specialty construction techniques, as in the use of micropiles. In such contractor- designed and built situations, the owner's project engineer and inspector must have a clear understanding of the items to be controlled by specification and the items left to the discretion of the specialty contractor. Recommended contracting methods and construction specification guidelines are discussed in Chapter 9. This chapter is devoted to the operations and procedures that require the attention of the owner's project engineer and/or construction inspector. Its purpose is to provide field inspectors with the knowledge necessary to effectively monitor and document the construction of micropile support systems.

The QA/QC process begins before the specialty micropile contractor appears on the job site. QA/QC begins in the plans and specification development stage, and is ensured by adherence to the intended specifications. **The proper installation of micropiles is a process that requires the expertise of an experienced specialty contractor. The most important section of the specifications to be enforced by the owner deals with the experience qualifications of the micropile contractor and personnel hired to perform the project work.** Failure to enforce the specified qualifications opens the doors for inexperienced contractors trying to cut costs, often with negative consequences to the owner. The results often are inferior workmanship, project delays, and project claims that often substantially increase project costs. Results like these often discourage project owners from implementing new technology, and draws them back to more traditional methods at any cost. This can be avoided with the proper specification implementation and, as importantly, enforcement to ensure a mutually successful project.

In addition to the expertise of the specialty contractor, the quality of the individual construction elements is directly related to the final products overall quality. As with other drilled pile systems, the actual ultimate capacity of a micropile can only be definitively proven by pile load tests. It is not practical or economical to test every element installed. It is therefore imperative that close attention is paid to the quality of the materials, including their storage, handling, and preinstallation preparation, and the construction at all stages of the work. Inspection begins with the arrival on the job site of the micropile materials.

8.B QUALITY CONTROL INSPECTION

8.B.1 Material Handling and Storage

Grout

High-quality grout starts with proper material storage (Photograph 8-1). This primarily consists of preventing hydration by keeping the cement dry. Cement is typically supplied either in bagged (43 kg/sack) or bulk (1320 kg/bulk bag) form, depending on site conditions, job size, local availability, and cost. Cement should be shipped on wooden pallets, and handled and stored in a protected location to avoid moisture absorption. It should not be stored directly on the ground or anywhere it may be subject to significant moisture absorption. Avoid stacking the bags too high to prevent overcompaction of the cement. Prior to use, the cement should be visually checked for lumps or other indications of hydration or contamination by foreign matter.

All cement shall conform to the proper standards (Chapter 4, Section 4.C). Handling of admixtures should follow a similar course, with primary attention paid to manufacturer's recommendations. Admixtures that have exceeded the manufacturer's recommended shelf life shall not be used.

Water used for grout mixing must be potable, and not contain impurities harmful to the steel or the grout. Potable water is typically obtained from adjacent hydrants and/or other sources.

Photograph 8 - 1. Cement Storage

Reinforcing Steel

The micropile contractor should use care in unloading and storing all reinforcing steel material. The storage area should be away from construction traffic and in a protected location. When reinforcing steel material is delivered to the job site, it should be visually checked for size and any signs of corrosion. Reinforcing steel elements should be accompanied by their mill certificates. Mill secondary pipe material, for which mill certificates are not available, require verification of steel quality through tensile and chemical testing of steel samples. *For Buy America Contracts, United States manufacturing of materials must be verified.* All steel reinforcing materials shall conform to the proper standards (Chapter 4, Section 4.D.2), as identified in the project specifications.

Photograph 8 - 2. Storage of Reinforcing Steel

Properly stored steel reinforcing materials (see Photograph 8-2) prevents corrosion or contamination by dirt, oils, and/or organics. Wood dunnage placed between the ground and the steel materials will prevent slight rusting that can occur if steel is exposed to the ground. A light coating of surface rust on the steel is normal and indicates that oil and grease are not present. Deep flaky corrosion or deep pitting of the steel is cause for rejection.

Double corrosion-protected and/or epoxy-coated steel bars should be delivered to the job site pre-manufactured (Photograph 8-3). Extra care in handling and storage shall be given to these materials. Epoxy-coated bars shall be properly wrapped with padded bands and placed on dunnage. Pre-manufactured corrosion-protected steel bars must be stacked with care to prevent any damage to the corrugated tube and/or epoxy coating.

Photograph 8 - 3. Double Corrosion Protected (Encapsulated) Reinforcing Steel

Mechanical couplers, hex nuts, centralizers, and spacers are typically delivered to the job site in boxes. Mill certificates will be provided for the steel couplers and hex nuts. Storage of these materials shall follow the same procedures as the other materials.

Inspection of all associated reinforcing elements should include:

- Checking for damage.
- Checking for corrosion of steel.
- Checking proper size.
- Checking proper corrosion protection.
- Obtaining mill certificates and any independent material test results.

8.B.2 Construction Monitoring

Prior to the commencement of any construction, the micropile contractor's plan of work should be established by submittal of their project-specific working drawings. The work plan should describe all anticipated aspects of construction, including any anticipated difficulties. Following is a compilation of installation submittal requirements.

1. Contractor and employee qualifications.
2. Description of Contractor's understanding regarding materials anticipated to be drilled from geotechnical report. (List any potential problems).
3. Performance criteria and tolerances.
4. Location and orientation of the micropiles.
5. Micropile size and configuration.
6. Micropile capacity.
7. Drilling equipment, including manufacturer and model numbers, flushing media, and precautions against drilling deviation.
8. Anticipated equipment loads on structure or adjacent ground during construction.
9. General installation plan, including proposed sequence of installation, phasing, and scheduling.
10. Grout design mix, along with batching, mixing, and injection techniques.
11. Reinforcement, including sizing, configuration, and corrosion protection.
12. Postgrouting methods, procedure, and equipment (if any).
13. Documentation and protection of existing utilities and other sensitive elements in the built-environment, and proposed measures to anticipate the conditions.

14. Plan to accommodate low headroom or nearby obstructions.

15. Testing criteria including: maximum design and proof loads, allowable deformations under test loads, and testing procedures.

16. Location of load cells, proper gauge calibration, and any other testing or monitoring devices.

17. Details of connection to existing structures.

18. Criteria for implementing remedial procedures.

19. Spoil handling (typically performed by general contractor).

Most importantly, protection of the existing environment, including underground utilities, must be considered. The existing site conditions should be examined by the micropile contractor with the owner's inspector. The contractor is responsible for furnishing equipment suitable for the specific site conditions of each project.

Drilling

As outlined in Section 4.B (Drilling), a wide range of methods can be utilized for micropile drilling. Most drilling techniques are likely to be acceptable, provided they can form a stable hole of the required dimensions within the permitted tolerances, and without detriment to the surroundings.

The physical nature of drilling and forming a hole may disturb the surrounding ground for a certain time and over a certain distance. It is important that the owner's inspector is aware of this and that he/she is assured the micropile contractor's equipment and drilling and grouting methods and procedures will produce the desired results. Ground disturbance is an important issue; the use of high pressures in poorly controlled flushing operations should always be undertaken with care.

The inspector must ensure that drilling operations are not causing unacceptable loss of ground. Obvious signs are the inability to withdraw drill casing, large quantity of soil removal for little or no casing advancement, and subsidence of the ground above drilling location.

The inclination and position of micropiles may deviate somewhat from their designed specifications. The following normal tolerances are typical of those proposed for larger diameter elements.

In plan	75 mm in any direction
Verticality	1 in 50
Between vertical or inclined up to 1:6 .	1 in 25
Inclined greater than 1:6	1 in 15

Although the pile tolerances above are realistic for most sites, tolerances may have to be relaxed where the ground contains obstructions and appropriate allowances made in the design of micropiles and their associated structural members.

In all cases, drilling must not allow collapse of the borehole. When temporary casing is to be fully extracted during the grouting process, it should be removed in such a way that the pile reinforcement is not disturbed, damaged, or allowed to contact the soil. It should also be kept full of grout during the extraction operation, to minimize the danger of pile hole collapse. *(Note: Fluid levels in the hole must always be kept above the ground water level so that the internal fluid pressure adequately balances external ground water pressure and active earth pressure throughout all potentially unstable soils.)*

The drilling, installation of reinforcement, and grouting of any given micropile should be completed in a series of continuous processes as expeditiously as possible. Some materials, such as consolidated clays and weak rock, can deteriorate and soften on exposure. A good rule of thumb in these conditions is to make sure drilling of the pile bond zone be undertaken on the same day as reinforcement installation and grouting.

Unless pile construction is continuous, the pile hole should be temporarily plugged or covered at the surface to prevent debris from falling into the borehole and for safety of the personnel working on the site. It is also recommended that any casing (temporary or permanent) project a minimum of 0.3 m above the head elevation to avoid drill spoils and flushing water from other operations flowing down and contaminating the completed hole.

The micropile contractor shall provide for proper disposal and containment of the drilling spoils from the site in accordance with the approved working plan. (*Note: Handling and disposal of the drill spoils may be the responsibility of the general contractor.)* Beware of uncontrolled drill water causing erosion or freezing on/off the site.

Any of a great number of problems may develop during the drilling process, and thereby require close supervision. It is imperative that comprehensive pile installation logs be maintained. This is discussed later in the chapter.

Grouting

Before any grouting begins, the condition of the equipment should be checked and the approved grout-mix design verified. Care in execution of the grouting process can greatly impact pile load performance. Some guidelines to proper grouting practices follow.

1. Select equipment to ensure continuous grout placement.
2. Select a mixer capable of colloidally mixing the cement, and a storage tank that will then permit continuous grout agitation.
3. Prevent the presence of air in the grout lines by expelling all air and checking that the suction circuit is airtight.
4. During grouting, do not draw down the level of grout in the supply tank below the crown of the exit pipe.
5. Ensure the exclusion of any foreign matter during grout placement for the full length of the pile.

6. Use tremie methods.
7. Prevent heaving in horizontally stratified soil by limiting grout pressures or by limiting the quantity of grout pumped. Both may be necessary.
8. Prevent the soil at the bottom of the hole from blowing in.

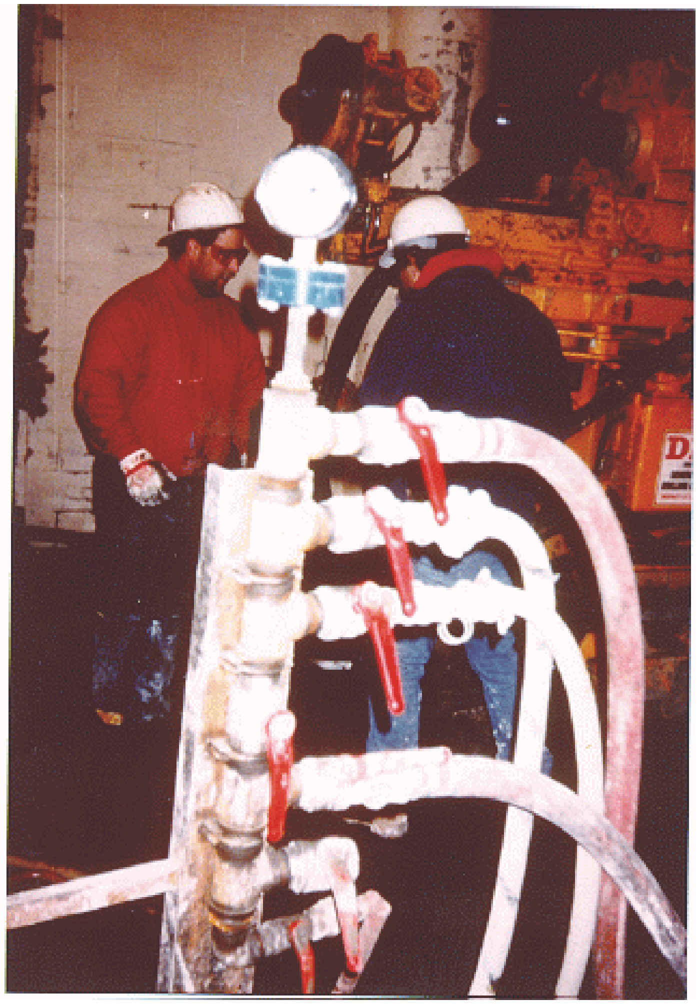
Prior to grouting, the diameters of micropiles drilled without casings should be verified. Ideally, micropiles should be grouted immediately after drilling the bond zone. If this is not possible, then grouting should be performed before degradation of the drillhole.

The primary grouting operation involves injecting cement grout at the lowest point of the borehole so the hole will fill evenly without air voids. A tremie tube is typically tied to the reinforcing steel before insertion in the borehole. Care must be taken to ensure that the tube is tied loose enough to be removed from the borehole on completion of the grouting. Contractors typically use duct tape to wrap the tremie tube and reinforcing steel loosely together. After placement of the reinforcement steel and tremie tube, the grout is pumped to the bottom of the pile until grout of suitable quality (of the same consistency as that being injected) returns to the top of the pile. The tremie tube is then removed from the pile hole.

Grouting should be performed in one continuous operation. Care should be taken to maintain a positive head at the grout holding tank, to avoid drawing air into the injected grout.

Pressure grouting may be applied to the primary grout during casing withdrawal. Care must be taken with such pressures to avoid distress to the ground or adjacent structures. The pressure grouting process should be stopped if grout is observed escaping at the ground surface.

Grout pressures are measured as close to the point of injection as possible to account for line losses between the pump and the borehole. Typically, a pressure gauge is mounted on the drill rig and monitored by the drill rig operator as a guide during pressure grouting and casing withdrawal (Photograph 8-4).



Photograph 8 - 4. Drill Rig Pressure Gauge

For micropiles installed with a hollow-stem auger, the grout should be pumped under continuous pressure with a grout head maintained above the tip of the augers at controlled and monitored withdrawal rates to ensure that no "necking" of the borehole occurs.

If postgrouting is performed, it is very important to monitor grout pressures and volumes throughout the injection of each sleeve. This will help prevent dangerous or needless overinjection and progressively verify the effectiveness of the treatment. Postgrouting pressures have been documented in excess of 6.5 MPa. Typical range during injection is 2 to 4 MPa.

Grouting records are of vital importance on every project and are described in detail later in this chapter.

Installation of Reinforcement

Reinforcement may be placed either prior to grouting, or placed into the grout-filled borehole before temporary casing (if used) is withdrawn. In any case, the inspector shall record the total pile length and bond zone length for each installed micropile. The reinforcement shall be capable of being inserted to its prescribed length in the hole without force. Inability to achieve this penetration in uncased holes usually means caving of soil into the hole has occurred, and withdrawal of the reinforcement and redrilling is necessary. *(Note: Inspectors should be cautioned to prevent the micropile contractor from driving the reinforcement to full length or cutting off the reinforcement.)*

Care must be taken not to damage any corrosion protection or centralizers during installation. Neither epoxy-coated, double corrosion-protected, or uncoated steel reinforcements should be dragged across abrasive surfaces or through the surface soil. All splices and couplings should be checked for proper seating. The reinforcement must be clean of deleterious substances such as surface soil, oil, and mud, which may contaminate the grout or coat the reinforcement. Centralizers and spacers shall be checked to ensure placement at specified intervals, typically 2.5 to 3 meters (Photograph 8-5.) Attachment of the centralizers to the reinforcement should also be checked to ensure that the reinforcement remains centered in the borehole. Typically centralizers are attached with tie-wire or duct tape. *(Note: Inspectors should be cautioned to make sure tremie grout tubes can be placed to the bottom of the borehole without interference with the centralizers, if placed after bar installation.)*

Photograph 8 - 5. Reinforcement Centralizers

8.C QUALITY CONTROL DOCUMENTATION

8.C.1. Pile Load Testing

Quality assurance and control often begins with the verification pile load testing and continues during production pile installation and production pile proof load testing. During the verification load testing, the inspector must understand the reasons for requiring careful control of such testing and the practical meaning of the acceptance criteria. Verification pile testing is done to verify the design capacity and construction processes of the micropiles.

The inspector must ensure the test pile installation method reflects the method planned for the installation of the production piles, the pile load test setup is in accordance with the approved working drawings, and the pile is load tested in accordance with the project specifications. Items to check include testing apparatus alignment, use of properly calibrated jacks and gauges, dial gauge arm alignment and travel length, dial gauge point of contact, application of the proper loads, and potential interference points due to pile deflection.

It is important for the owner's project engineer and the micropile design engineer to be prepared to evaluate potential pile load test failure. Solutions for alternative pile types and/or layouts will need to be developed and implemented quickly so as not to delay the project schedule.

8.C.2 Production Piles

Comprehensive records of the pile installation and grouting operations are of vital importance in establishing the basis of payment and highlighting any deviation that may be significant to pile performance at a later stage. Table 8-1 provides a recommended micropile installation log, to be used by the micropile contractor and the owner's inspector.

Because the grout is such a vital component of the micropile, close attention should be paid to the control and quality of the product. Grout production and consumption records must be kept daily. It is important that the actual pressure and volume of grout pumped in each pile be recorded. The grout take itself will tell a lot about the success of the procedure.

Compressive Strength: Unconfined compressive grout strength is determined according to AASHTO T106/ASTM C-109. Compressive strength is checked through a set of three each, 50-mm grout cubes (Photograph 8-6.) Tests are typically performed at 3, 7, and 28 days after grouting. Seven-day tests are considered the most crucial, as the grout will typically attain the required design strength within this time period. Anticipated strength will depend upon project need, but compressive cube strengths can be very high, as much as 24 MPa after only 24 hours of set time. The unconfined compressive strength is largely dependent on project needs, but a 28-day strength requirement between 25 and 40 MPa is considered common for micropiles. The inspector needs to verify that the grout cube break strengths comply with the project specifications. Grout samples should be taken directly from the grout plant. It is recommended that one set of three grout cubes be taken for every 10 piles installed, or every day for each grout plant in operation, whichever occurs more frequently.

Table 8-1. Micropile Installation Log

Micropile Installation Log

Project Name:	
Contract No.:	

Pile Designation #		Time @	
Installation Date		Start of Drilling	
Drill Rig/Drill Method		Start of Grouting	
Drill Rig/ #, Operator		Pile Completion	
Grout Plant #, Operator		Total Duration	

Drill Bit Type and Size		Cement Type	
Casing Dia./Wall Thickness		Admixtures	
Pile Inclination		w/c Ratio	
Reinforcement Size/Length			

Pile Length Above B.O.F.		Tremie Grout Quantity (bags)	
Upper Cased Length		Pressure Grout Quantity (bags)	
Cased and Bond Length (Plunge)		Grouting after plunge (bags)	
Bond Length Below Casing		Total Grout Quantity (bags)	
Total Pile Length		Grout Ratio (bags/m bond)	

Comments - Pile Drilling

Depth from B.O.F (m)	Soil / Rock Description	Flush Description	Comments

Comments - Pile Grouting

Depth from B.O.F (m)	Pressure Range Max/Average (MPa)	Comments

B.O.F. = Bottom of Footing (ref. FHWA-SA-97-070)

Table 8-2. Completed Micropile Installation Log

Micropile Installation Log

Project Name:	Keeley Palace
Contract No.:	0064

Pile Designation #	Pile 1	Time @	
Installation Date	12-25-96	Start of Drilling	8:00 a.m.
Drill Rig/Drill Method	Klemm 806, Cased Rotary	Start of Grouting	9:00 a.m.
Drill Rig/ #, Operator	10-1 / Kilian	Pile Completion	9:20 a.m.
Grout Plant #, Operator	8-1 / Holder	Total Duration	1:20

Drill Bit Type and Size	Casing Teeth – 175 mm	Cement Type	I / II
Casing Dia./Wall Thickness	175 mm / 50 mm	Admixtures	None
Pile Inclination	0 degrees	w/c Ratio	0.45
Reinforcement Size/Length	57 mm / 15.5 m		

Pile Length Above B.O.F.	0.5 m	Tremie Grout Quantity (bags)	12
Upper Cased Length	8 m	Pressure Grout Quantity (bags)	13
Cased and Bond Length (Plunge)	2 m	Grouting after plunge (bags)	2
Bond Length Below Casing	6 m	Total Grout Quantity (bags)	27
Total Pile Length	16.5 m	Grout Ratio (bags/m bond)	12/16.5 + 15/8 = 2.6

Comments – Pile Drilling

Depth from B.O.F (m)	Soil / Rock Description	Flush Description	Comments
0 – 5	Gravel and Cobbles	Brown, full return	
5 – 10	Sand Gravel w/Cobbles	Brown, full return	
10 – 15	Cobbles w/Gravel	Gray, full return	Occasional Sand Seams

Comments – Pile Grouting

Depth from B.O.F (m)	Pressure Range Max / Average (MPa)	Comments
15 – 10	0.55 / 0.45	Grouted first 3 m under pressure, pulled remainder under static head
10 – 5	0.55 / 0.47	Grouted first 3 m under pressure, pulled remainder under static head
Plunge, 5 – 6.5	1.2	After plunging casing, pumped additional 2 bags at max pressure = 1.2 MPa

B.O.F. = Bottom of Footing

(ref. FHWA-SA-97-070)

Photograph 8 - 6. Grout Cubes for Compressive Strength Testing

Specific Gravity: The water content in grout is the prime control over grout properties and is most frequently checked through specific gravity measurements. Neat cement grout density can be determined per the *API Recommended Practice 13B-1* by the Baroid Mud Balance Test (Photograph 8-7). The test is extremely quick and inexpensive. Frequent checking is recommended - at least once per pile. By monitoring the water/cement ratio during grouting, the inspector can ensure that the grout is being prepared according to the specified project mix design. A common specific gravity for many types of micropiles is between 1.8 and 1.9 kg/m^3. This equates to a 0.45 water/cement ratio.

Tests that permit quality assessment prior to installation are more valuable since they permit immediate reaction in case of anomalies. Unfortunately, other factors like compressive strength values may only be established 48 hours after the sampling at the earliest. It is good practice that such initial tests be made on the design grout mix in pre-construction trials to give the owner's representative confidence prior to production drilling. The design mix of the grout can be verified prior to and/or during the micropile verification load testing.

Photograph 8 - 7. Baroid Mud Balance Test

CHAPTER 9

CONTRACTING METHODS

9.A INTRODUCTION

To date, methods of design, specification, and installation of micropile systems in the United States have been developed by specialty geotechnical contractors. Most public agencies and consulting engineers presently have little or no knowledge regarding micropiles and their application. The goal of this manual is to provide guidelines that will help owners and engineers implement the most technically and economically feasible application of micropile technology in everyday use.

As discussed at the beginning of Chapter 8, preparation and enforcement of the contract documents are important steps in the introduction of new technologies. For innovation to flourish, there must be a need and a reward for those who take the risk of funding and implementing the technology. One of the biggest constraints on this is our most commonly used delivery system for construction projects, the traditional “low-bid” owner-designed system. This process limits innovation, promotes the use of unqualified contractors and poor quality, may result in an increase in end-product costs, and retards the implementation of new technology. With this in mind, the implementation of project specifications incorporating the use of alternative contracting methods is vital to the adoption of micropile technology.

In order to insure quality micropile construction, it is strongly recommended that projects utilizing micropiles be specified to place a large amount of responsibility on the specialty micropile contractor. This is the basis for the remaining portions of this chapter. Alternative contracting methods are described and compared to provide guidelines for owners and consulting engineers when specifying or allowing micropiles. Also, information to be included on the contract plans and the specialty contractor working plan submittal is discussed.

9.B SPECIFICATIONS

As most specialty geotechnical work is performed in the public sector, fair contracting practices are always an issue. The U.S. construction industry is very strongly geared to the owner- designed low-bid contracting process to promote these fair contracting practices. Some owners are reluctant to move away from this system, for fear of potential litigation caused by unfair contracting. However, there are proven alternative contracting methods that can be used to encourage good quality innovation and still protect the owners' interest, even within the confines of the traditional low-bid system.

The specifications can be used to mandate methodology or allow alternative designs. The degree of detail will be based on the designer's experience with micropile installations, owner confidence with the pre-qualified micropile contractors, and the critical nature of the application. An important note to always remember when preparing the plans, specifications, and cost estimate for micropile projects is that the specialty contractor installing the micropiles is often a subcontractor on the project. Support services are necessary from the general contractor and will have to be included in the micropile pricing (e.g., access, spoils handling, footing excavation and backfill, etc.).

Two general options for micropile design are set forth in Appendix A-1 and A-2 Guide Specifications of this manual. These are summarized in Table 9-1.

As stated earlier in this chapter, micropile specifications must place a large amount of responsibility on the specialty geotechnical contractor. With this in mind, pre-qualification of micropile contractors is highly recommended for both owner-controlled design and contractor design/build specifications. It is not the intent of this document to develop a pre-qualification system for every project owner, but it is recommended that the pre-qualification process be performed prior to contract advertisement. The pre-qualified micropile contractors (minimum of two or as determined by state law or owner agency policy) can be listed in the contract specifications. If the micropile work is a subcontracted item, each general contractor must be required to provide the name of its micropile subcontractors with the bid submittal.

Table 9-1. Contractor Design/Build Options

	Information provided by Owner Engineer	Design by Micropile Specialty Contractor
Option 1: Contractor Design/Build of Micropiles	1. Footing Design & Pile Layout 2. Pile working load (axial tension, axial compression and lateral) and ultimate/seismic load and tolerable deflections 3. Geotechnical information (preferably Geotechnical Baseline Report) 4. Ground zones which can not be used for pile capacity (weak soil, scour and liquefiable zones)	1. Pile structural design 2. Pile geotechnical design (length, diameter etc.) 3. Pile connection design
Option 2: Contractor Design/Build of Foundations (Micropiles and Footing)	1. Load combination (vertical, lateral and moments) for working and seismic cases at top of footing 2. Geotechnical information (preferably Geotechnical Baseline Report) 3. Ground zones which can not be used for pile capacity (weak soil, scour and liquefiable zones)	1. Footing design 2. Pile structural design 3. Pile geotechnical design (length, diameter etc.) 4. Pile connection design

The following minimum pre-qualification requirements are recommended for each micropile contractor:

1. The company and personnel's previous successful experience – evidenced by owner references-in the design and installation of micropiles of similar scope to those proposed for the application project. Documentation shall include at least five successful projects performed in the last five years.

2. The contractor shall assign an engineer to supervise the work who has at least three years of experience in the micropile design and construction of at least 3 successfully completed projects over the past 5 years.

3. Micropile contractor must have previous drilling experience in soil/rock similar to project conditions. Contractor must submit at least three successful load-test reports from different projects of similar scope to project.

4. Project superintendents and drill operators responsible for installation of micropile system must have micropile installation experience on at least 3 successfully completed projects over the past 5 years.

Alternative specifications for both owner controlled design and contractor design/build projects are described in the following sections. *(Note: All specification recommendations assume pre-qualified micropile contractors are listed in the contract specifications and the general contractors are required to name their pre-qualified micropile subcontractor of choice at bid time.)*

9.B.1 Owner-Controlled Design Methods

Owner-controlled design specifications vary in the amounts of the design to be performed by the owner's design engineer and the micropile contractor. The owner typically establishes the following:

- Scope of work.
- Micropile design loadings and spacing.
- Footing details.
- Corrosion protection.
- Micropile testing procedures and requirements.
- Instrumentation requirements.
- Special design consideration (e.g., scour and liquefaction potential).
- Performance criteria.

The micropile contractor specifies the following:

- Micropile construction process.
- Micropile type.
- Micropile design.
- Pile top footing connection design.

This division of work allows the pre-qualified specialty contractors to provide an economical micropile design, while satisfying the owner's engineer's design requirements. This method also allows responsibility for the work to be shared between the owner and the micropile contractor. While this is slightly more restrictive than the contractor design/build specification method, it is still flexible enough to allow for some innovation and cost savings. Various types of owner-controlled design specifications include the following.

Standard Design

The contract documents for the Standard Design Type are prepared to allow for various pre-qualified micropile designs. The owner's engineer provides the micropile design loadings, footing design, and pile layout for foundation support projects. In addition, the owner provides the following:

- Geotechnical reports and data.
- Micropile design parameters, including (foundation support projects only):
  - Maximum pile sizes.
  - Axial pile loads.
  - Lateral pile loads.
  - Displacement requirements.
  - Ductility requirements.
- Existing utility plans.
- Site limitations, including:
  - Access limitations.
  - Right of way.
  - Scour.
  - Liquefaction.
  - Noise requirements.

Vibration requirements.

Hazardous/contaminated soils.

- Contractor working drawing/design submittal and review requirements, including:

    Time frame.

    Penalties.

- Material specifications.
- Testing requirements.
- Instrumentation requirements (if any).
- Acceptance criteria.
- Method of measurement and payment.

It is recommended that micropile measurement and payment be performed on a unit-per-micropile basis separately for 1) furnished and installed production piles, 2) verification testing, and 3) proof testing, except for piles founded in rock, for which an add/deduct footage price should be included (see Appendix A, *Guide Specifications Commentary*).

During the bidding process, the pre-qualified micropile contractors prepare a preliminary design and a firm cost proposal based on the owner's plans and specifications. If the project is to be subcontracted, general contractors will receive bids from each pre-qualified specialty subcontractor and includes the best offer in the proposal. The name of the recommended micropile subcontractor is included in the general contractor's bid, which is then submitted. Once the contract has been awarded, the selected specialty contractor prepares their working drawings and design calculations and submits them to the engineer for review and approval. After acceptance of the design, construction begins.

Appendix A-1 contains sample contract plans and guide construction specifications utilizing this method.

Alternate Micropile Design

The contract documents for the Alternate Micropile Design Type are also prepared to allow for various pre-qualified micropile designs. The major difference in this method is the owner provides a design in the contract documents utilizing a more traditional foundation support system. The contract documents allow alternate micropile designs to be submitted by the listed specialty contractors on a one-for-one pile replacement of the owner-designed pile system. The information described under Standard Design Type is also required.

An attractive option to this one-for-one pile replacement method is where the owner provides three alternative pile-supported footing designs in the contract documents. Alternate 1 would be one-for-one pile replacement, Alternate 2 could be one-for-two pile replacement, and Alternate 3 could be one-for-one-and-one-half pile replacement. This allows the pre-qualified micropile contractor to provide fewer higher capacity micropiles than the conventional pile footing design requires, and allows the owner's engineer to maintain control of the footing design.

The information necessary in the contract documents is similar to that previously mentioned, except the owner provides two to three alternative pile footing designs on the plans with the associated pile design and load criteria.

Cost Reduction Incentive Proposal (Value Engineering)

The Cost Reduction Incentive Proposal is another long-established form of alternate proposal used in the United States.

When Cost Reduction Incentive Proposals are permitted, it is important that the owner specify any additional restrictions that may apply, such as right-of-way restrictions. Foundation support elements are usually the first order of work, so the approval time required for review and approval of the Cost Reduction Incentive Proposal requires quick response by the Contractor and commensurate responsive review by the Owner.

9.B.2. Contractor Design/Build Methods

According to the owner-controlled design specification method, when the use of micropiles is stipulated for a project, the owner's engineer defines the scope of work and shares responsibility in the design and installation of the micropile system. With the design/build method, by contrast, the owner outlines the project's ultimate needs and the specialty contractor is responsible for the micropile system detailed design and installation. The owner's engineer typically establishes the following:

- Scope of work.
- Total structure loads.
- Footing details.
- Corrosion protection.
- Micropile testing procedures and requirements.
- Instrumentation requirements.
- Special design consideration (e.g., scour and liquefaction potential).
- Performance criteria.

The micropile contractor specifies the following:

- Micropile construction process.
- Micropile type and quantity.
- Micropile design.
- Revisions to footing details to accomodate micropile design.
- Pile top footing connection design.

Based upon specified limitations and requirements, a design/build proposal is submitted, either before the bid advertisement (pre-bid), or after contract award (post-bid). Measurement and payment is typically made on a lump-sum basis.

In design/build construction, the owner is committed to a team approach whereby the specialty contractor becomes an important part of the team, contributing to all foundation and ground-support aspects of the project. Risk sharing is integral: the micropile contractor is responsible for the adequacy of the design and its construction, the owner is responsible for the accuracy of the information upon which the design is based. Costs are reduced, as the contractor includes fewer contingencies. Innovation is encouraged, since the contractor is rewarded for economies of design and installation. Lastly, quality is enhanced due to pre-qualified contractors working with the project owner in a partnering approach. The two recommended types of contractor design/build specifications follow.

Postbid

The contract documents for the postbid design/build method are prepared to allow for various pre-qualified-contractor-designed alternatives. The owner's engineer provides the design and detailing of ancillary structures, and the performance criteria and objectives necessary for the micropile system design and installation. This information includes, as a minimum, the following:

- Geotechnical reports and data.
- Structure loadings (axial, lateral and moment)
- Existing utility drawings..
- Design criteria and parameters.
- Site limitations (e.g., right of way).
- Design and details of ancillary structures.
- Contractor working drawing/design submittal and review requirements.
- Acceptance criteria.

During the bidding process, the pre-qualified micropile contractors prepare a preliminary design and a firm cost proposal based on the owner's plans and specifications. For subcontracted items, general contractors will receive bids from each pre-qualified specialty subcontractor and include the best cost proposals in their bids. The name of the recommended micropile subcontractors are included in the general contractor's bid, which is then submitted. Once the contract has been awarded, the selected specialty contractor prepares their detailed design calculations and working drawing and submits them to the engineer for review and approval. After acceptance of the design, construction begins.

Appendix A-2 contains sample contract plans and specifications utilizing this method.

Prebid

The contract documents for the prebid design/build method are also prepared to allow for pre-qualified-contractor-designed micropiling alternatives. The major difference in this method is the timing of the design and the bidding. Performance criteria and necessary project design information are usually made available 60 to 90 days prior to the contract advertisement date. Pre-qualified micropile contractors prepare and submit final design calculations and working drawings for the owner's review and approval. Once the designs (typically two to three total) are approved, a list of pre-qualified specialty contractors with approved designs are included in the contract documents. Often the specialty contractors' proprietary working drawings are included in the contract bid documents. These drawings illustrate the proposed construction and assist the general contractors in understanding and coordinating their other project tasks with the proposed micropile construction. General contractors then receive bids from each pre-qualified subcontractor, bidding only on their own proprietary design, and include the best cost proposal in their bid. The name of the recommended micropile subcontractor is included in the general contractor's bid, which is then submitted. Once the contract is awarded, the selected general contractor and specialty micropile contractor can begin work immediately.

9.B.3 Other Methods

Technical and Cost Proposals

Weighted technical cost proposals require the contractor to submit two very detailed documents, one for the price to perform the work and the other for the proposed work plan. Each is independently assessed. The first portion is a plan of how the contractor proposes to perform the work. This portion contains the contractor's design and construction experience, proposed scope of work and work plan, preliminary design and construction schedule, preliminary design calculations and drawings and proposed quality control and safety plans. This document is evaluated for technical competence, personnel, and corporate experience and safety. The second portion is a lump-sum cost proposal. Each element is given a rating; 70/30 or 60/40 for the technical-to-price ratio is not unusual. The contract award is made to the contractor who provides the best overall proposal to the owner.

This two-pronged contracting procedure is slowly gaining popularity with Federal and State agencies. In the technical cost proposal process as with the value engineering process, bidding contractors incur a lot of time and expense. This fact alone, however, will defer all but the most serious contractors. This process also involves considerable effort by the owner, and so is really viable only for particularly large and/or complex projects.

9.C. CONTRACT PLANS

For all of the specification methods mentioned, the contract plans must include the necessary bid information in order to protect the owner's best interests. They also need to describe the owner's objectives in enough detail that the micropile contractor can provide an adequate design and bid. It is recommended that the contract plans be prepared in a three-line format for both owner-controlled design and contractor design/build specification methods. Concept-only plans are provided for the contractor design/build method.

The quality of the subsurface information, existing utility plan, and micropile design criteria are very important for a mutually successful project. Inadequate subsurface information and conservative pile design criteria may create expensive contractor contingencies, higher pile prices and increase claim potential.

Sample contract plans and guide construction specifications are provided in Appendix A-1 (Contractor Design/Build of Micropiles) and Appendix A-2 (Contractor Design/Build of Foundation – Micropiles and Footing).

CHAPTER 10

FEASIBILITY AND COST DATA

10.A FEASIBILITY

Micropiles are used for structural support of foundations and in-situ earth reinforcement. Chapters 1 (Introduction), 3 (Applications), and 4 (Construction Techniques) demonstrated the wide range of conditions under which micropiles can be used. Micropiles are practical in any soil, fill, or rock condition, and can be installed at any angle. They can accommodate restrictive access and environmental problems, and have wide application both for new construction and rehabilitation of existing structures and/or marginally stable or failing slopes.

Technology-selection criteria for each foundation support and slope stabilization project must be site specific. Besides price, some standard criteria issues include environmental concerns, settlement sensitivity, soil disturbance, scheduling, physical access, noise sensitivity, pile tension, compression, and positioning capabilities. Sometimes micropiles are the only alternative. Other times, extenuating circumstances make them more economical than the more traditional systems.

Owners and engineers in the United States are gaining confidence with the development and implementation process of micropile technology, as attested by this manual. The American transportation construction industry must continue to build from the success of past research and project installations. It must continue to improve micropile design and installation methodologies and contracting methods to achieve the most optimal foundation and ground support solutions.

10.B MICROPILE COST DATA

Micropile costs are the product of many factors such as:

1. Physical access and environmental conditions.
2. Subsurface conditions.
3. Mobilization/demobilization.
4. Project location.
5. Pile quantity.
6. Pile capacities.
7. Pile length.
8. Pile inclination.
9. Pile testing requirements.
10. Pile installation schedule.
11. Local labor regulations.
12. Contractor overhead and margin percentages.
13. Risk assessment.
14. Contractual arrangement.

Because of these many cost factors, micropile pricing varies widely on every project. As a guideline, however, and assuming the below listed constraints, the contract bid price range for micropiles in the United States is typically \$150.00 to \$300.00 per lineal meter of pile (1996 costs). To take it a step further, the various cost factors must be analyzed in a “best case vs. worst case scenario” to determine a realistic micropile contract price range. Table 10-1 illustrates this cost analysis.

For example, price constraints may include the following factors:

1. No physical, environmental, or access restrictions.
2. No unusual subsurface conditions.
3. Average pile load capacities and lengths (1,000 kN, 15m).
4. Average pile quantity (50 piles).
5. One verification pile load test and proof testing 5 percent of the production piles.
6. One mobilization/demobilization.
7. Continuous pile drilling operations.
8. Prevailing labor rates.
9. Typical contractor overhead and margin percentages.

The sample problem from Chapter 5, along with a seismic retrofit sample project for comparison, are presented below in a discussion of budget ranges by micropile project type, using Table 10-1.

Table 10-1. Micropile Cost Influence Analysis

Cost Factor	Influence Range	Cost Influence (%)
Physical and access conditions	Very easy to very difficult	0% to +100%
Geology/soil conditions	Very easy to very difficult	0% to + 50%
Pile capacity	Very low to very high	-30% to + 30%
Pile lengths	Very short to very long	-25% to + 25%
Pile quantities	Very high to very low	-50% to +100%
Testing requirements	Very low to very high	-10% to + 10%
Mobilization/demobilization	One to multiple	0% to + 10%
Continuous drilling operations	Continuous to not continuous	0% to + 25%
Union agreements	Nonunion to very strong	-15% to + 30%
O/H and profit margins (risk evaluation)	Very low to very high	-10% to + 10%

10.C SAMPLE PROBLEMS-COST ESTIMATES

10.C.1 Sample Problem No. 1 (Chapter 5-Bridge Abutment Support)

Given: Single-span bridge located in mountainous terrain approximately 125 km northwest of Denver, Colorado. Each abutment is to be supported on 12 CASE 1, Type B micropiles with ultimate capacities of 1,190 kN in compression, and 490 kN in tension, and approximately 12 meters in length. Testing requirements are one verification pile load test (compression only) on a sacrificial pile and two proof tests. Access to each work bench must be established. Soil conditions are as shown in the Chapter 5 example problem. Drilling of bond zone is expected to be very difficult, due to cobbles and boulders. Drilling will be continuous at each location with reset up required at location of Abutment 2.

Solution: Evaluate each influence factor and multiply by the average of the two base prices as shown in Table 10-2, below:

Table 10-2. Sample Problem No. 1 – Cost Analysis (Chapter 5-Bridge Abutment Support)

Cost Factor	Influence	Cost Influence (%)
Physical and access conditions	Easy access	10
Geology/soil conditions	Difficult drilling	30
Pile capacity	Moderate pile capacity	0
Pile lengths	Moderate pile lengths	0
Pile quantities	Low pile quantities	25
Testing requirements	Moderate testing	10
Mobilization/demobilization	One mobilization	0
Continuous drilling operations	Two setups	5
Union agreements	Standard union agreements	0
O/H and profit margins (risk evaluation)	High-risk project	5
	Total:	**85%**

Therefore,

$$\left[1 + \frac{85}{100}\right] \times \left[\frac{\$150 + \$300}{2}\right] = \left[\begin{array}{c} \$416.00/\text{m} \quad \text{or} \quad \$4{,}995.00/\text{each (average unit)} \\ \text{or} \\ \$3{,}330.00/\text{each} \quad \text{to} \quad \$6{,}660.00/\text{each (range)} \end{array}\right]$$

10.C.2 Sample Problem No. 2 (Seismic Retrofit)

Given: Seismic retrofit of a 200-meter-long concrete viaduct located in San Francisco, California. A total of 30 footings need retrofitting with 12 CASE 1, Type B micropiles (360 micropiles) per footing. Ultimate capacities are 2,500 kN in compression and 1,500 kN in tension. Subsurface profile includes approximately 10 to 12 meters of medium-dense silty sands over weathered to moderately weathered serpentine bedrock. Micropiles average approximately 20 meters in length. Testing requirements include one each verification pile load test (compression only) on a sacrificial pile and 30 each proof tests. Access to each pile location is good, and overhead clearance varies between 5 to 12 meters for each footing. Drilling is assumed to be continuous allowing setups between each footing.

Table 10-3. Sample Problem No. 2 (Seismic Retrofit)

Cost Factor	Influence	Cost Influence
Physical and access conditions	Easy access	10
Geology/soil conditions	Easy drilling	10
Pile capacity	High pile capacities	10
Pile lengths	Moderate pile lengths	0
Pile quantities	Large pile quantity	-30
Testing requirements	Extreme testing	5
Mobilization/demobilization	One mobilization	0
Continuous drilling operations	Continuous drilling	0
Union agreements	Strong union	10
O/H and profit margins (risk evaluation)	Lower risk project	-5
	Total	**10%**

Therefore,

$$\left[1 + \frac{10}{100}\right] \times \left[\frac{\$150 + \$300}{2}\right] = \left[\begin{array}{c} \$250.00/\text{m} \quad \text{or} \quad \$5{,}000.00/\text{each} \ \ (\text{average unit}) \\ \text{or} \\ \$3{,}330.00/\text{each} \quad \text{to} \quad \$6{,}660.00/\text{each} \ \ (\text{range}) \end{array}\right]$$

Applying an unrealistic combination of the factors in Table 10-1 might raise the unit price by almost 400 percent beyond the "typical price," to approximately $700.00 per lineal meter. In such a case, micropiles may be technically feasible but may not appear cost effective, so an alternate technology may be investigated. Usually, however, the same factors that raised the micropile pricing will have the same cost implication on other piling options, which may not be technically feasible in any case. In addition, when comparing costs of alternative solutions, care should always be taken to clearly define the total end-product costs.

There are several methods of measuring and paying for micropiles. Table 10-4 lists recommendations for both owner-controlled design and contractor design/build specification methods.

Table 10-4. Micropile Measurement and Payment Units

Item	Measurement/Payment Unit	
	Owner-Controlled Design	Contractor Design/Build
Mobilization/demobilization	Lump sum	Lump sum
Pile load testing	Per each	Lump sum
Furnish and install piles (foundation support)	Per each	Lump sum
Furnish and install micropile slope stabilization	Per lineal meter of structure	Lump Sum

Proportioning the micropile unit costs (furnish and install only) typically results in the following breakdown:

Labor . . . . 30 – 50 percent
Equipment . . . . . 20 – 30 percent
Materials . . . . . . 25 – 40 percent

In closing, the unit price of micropiles usually exceeds that of conventional piles, especially driven piles. However, under certain combinations of circumstances, such as difficult ground conditions, site access constraints, low headroom/limited work area, etc., micropiles are cost effective and occasionally, represent the only technically feasible option.

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GLOSSARY OF TERMS

Admixture: Substance added to the grout to control bleed and/or shrinkage, improve flowability, reduce water content, or retard setting time.

Alignment Load (AL): A minimum load (5 percent DL maximum) applied to micropile during testing to keep the testing equipment correctly positioned.

Allowable Geotechnical Bond Load: For SLD, computed as the nominal grout-to-ground bond strength (α_{bond} nominal strength), divided by the geotechnical safety factor and then multiplied by the grouted bond surface area (bond length times drill hole circumference).

Apparent Free Micropile Length: The length of pile that is apparently not bonded to the surrounding ground, as calculated from the elastic load extension data during testing.

Bonded Length: The length of the micropile that is bonded to the ground and conceptually used to transfer the applied axial loads to the surrounding soil or rock. Also known as the load transfer length.

Bond-breaker: A sleeve placed over the steel reinforcement to prevent load transfer.

CASE 1 Micropile: A pile designed to accept vertical or lateral load directly, and transfer it to an appropriate bearing stratum. Usually includes significant steel reinforcement.

CASE 2 Micropile: One of a network of low-capacity piles used to delineate and internally reinforce a volume of soil.

Casing: Steel tube introduced during the drilling process in overburden soil to temporarily stabilize the drill hole. This is usually withdrawn as the pile is grouted, although in certain types of micropiles, some casing is permanently left in place to reinforce the unbonded length or provide additional capacity.

Centralizer: A device to support and position the reinforcing steel in the drill hole and/or casing so that a minimum grout cover is provided.

Coarse-grained Soils: Soils with more than 50 percent of the material by weight, larger than the No. 200 (0.075 mm) sieve size.

Cohesive Soils: Fine-grain soils that exhibit plasticity. Atterberg limits are commonly used to determine plasticity and better define a soil as cohesive or noncohesive.

Contractor: The person/firm responsible for performing the micropile work.

Corrosion-inhibiting Compound: Material used to protect against corrosion and/or lubricate the reinforcing steel.

Coupler: A device by which the pile load capacity can be transmitted from one partial length of reinforcement to another.

Creep Movement: The movement that occurs during the creep test of a micropile under a constant load.

Design Load (DL): The maximum unfactored load expected to be applied to the micropile during its service life.

Duplex Drilling: An overburden drilling system involving the simultaneous advancement of (inner) drill rod and (outer) drill casing. Flush from the inner drill rod exits the hole via the annulus between rod and casing.

Elastic Movement: The recoverable movement measured during a micropile test.

Elastic Ratio: A measure of pile stiffness calculated as elastic movement divided by pile load.

Encapsulation: A grout filled corrugated or deformed tube protecting the reinforcing steel against corrosion.

Fine-grained Soils: Soils with at least 50 percent of the material, by weight, smaller than the No. 200 (0.075 mm) sieve size.

Free (unbonded) Length: The designed length of the micropile that is not bonded to the surrounding ground or grout during stressing.

Geotechnical Bond Design Strength: For LFD, computed as the nominal grout-to-ground bond strength (α_{bond}), multiplied by a geotechnical resistance factor φ_G. Use $\varphi_G = 0.6$ for typical designs and non-seismic load groups; use $\varphi_G = 1.0$ for seismic loads groups

LFD: Load Factor Design

Maximum Test Load: The maximum load to which the micropile is subjected during testing.

Micropile: A small-diameter (typically less than 300 mm) drilled and grouted replacement pile which is typically reinforced.

Non-cohesive Soils: Granular soils that are generally nonplastic.

Overburden: Material, natural or placed, that requires cased drilling methods to provide an open borehole to underlying strata.

Permanent Micropile: Any micropile for permanent use, generally with more than a 24-month service life. May require special design, corrosion protection, and supervision during installation.

Plunge Length: The length that the pile casing is inserted into the grout-to-ground bond zone.

Post-grouting: The injection of additional grout into the load transfer length of a micropile after the primary grout has set. Also known as regrouting or secondary grouting.

Preloading: Loading the micropile, prior to the micropile's connection to the structure, to minimize or eliminate any structural movement in service.

Primary Grout: Portland-cement-based grout injected into the micropile hole prior to or after the installation of the reinforcement to direct the load transfer to the surrounding ground.

Proof Test: Incremental loading of a production micropile, recording the total movement at each increment.

Reinforcement: The steel component of the micropile that accepts and/or resists applied loadings.

Residual Movement: The nonelastic (nonrecoverable) movement of a micropile measured during load testing.

Safety Factor: The ratio of the ultimate capacity and design working load; used for the design of any component or interface.

Sheath: A smooth or corrugated pipe or tube that protects the reinforcing steel against corrosion.

Single-tube Drilling: The advancement of a steel casing through overburden, usually aided by water flushing through the casing. The water may or may not return to the surface around the casing, depending largely on the permeability of the overburden.

Spacer: A device to separate elements of a multiple-element reinforcement to ensure full bond development of each steel element.

SLD: Service Load Design

Temporary Micropile: Any micropile for temporary use, generally with less than a 24-month service life. Temporary micropiles installed in corrosive environments may require corrosion protection.

Type A-D: Classification of micropiles based on method and pressure of grouting.

Verification Load Test: Non-production (sacrificial) pile load test performed to verify the design of the pile system and the construction methods proposed, prior to installation of production piles.

Working Load: Equivalent term of Design Load.

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APPENDIX A-1

Micropile Guide Construction Specification and Sample Plans

[Foundation Support Projects]

Contractor Design/Build of Micropiles

Metric (SI) Units

(With Commentary)

*(**Commentary:** Owner-Controlled design specifications can vary in the amount of the design to be performed by the Owner's design engineer and the amount performed by the micropile specialty Contractor.* ***This guide specification is set up for the Owner-controlled design (Standard Design) method wherein the Owner provides preliminary plans showing the pile design loadings, footing design, and pile layout for each footing location. The Owner also provides related design criteria and requirements, subsurface data, rights-of-way limits, utility locations, site limitations, construction material and testing specifications, and required Contractor working drawing/design and construction submittals and review requirements. The micropile Contractor designs the individual micropile elements and pile top footing connections and selects the micropile construction process and equipment.*** *This approach is very similar to that commonly used by many highway agencies for Owner design of permanent tieback and permanent soil nail walls. During the bidding process, the* ***prequalified*** *micropile contractors prepare a preliminary micropile design and a firm cost proposal based on the Owner's preliminary plans and specifications. If the micropile portion of the project is to be subcontracted, general contractors will receive bids from the prequalified micropile contractors and* ***include*** *the best offer and name of the selected micropile contractor in their bid submittal. Once the contract is awarded, the selected micropile Contractor prepares detailed micropile design calculations and working drawings and submits them to the Engineer for review. After acceptance of the design, construction begins. For more detailed discussion on various contracting methods, refer to Chapter 9.)*

1.0 DESCRIPTION

This work shall consist of constructing micropiles as shown on the contract plans and approved working drawings and as specified herein. The micropile specialty Contractor is responsible for furnishing of all design, materials, products, accessories, tools, equipment, services, transportation, labor and supervision, and manufacturing techniques required for design, installation and testing of micropiles and pile top attachments for this project.

The selected micropile Contractor shall select the micropile type, size, pile top attachment, installation means and methods, estimate the ground-grout bond value and determine the required grout bond length and final micropile diameter. The micropile Contractor shall design and install micropiles that will develop the load capacities indicated on the contract plans. The micropile load capacities shall be verified by verification and proof load testing as required and must meet the test acceptance criteria specified herein.

Where the imperative mood is used within this specification, "The Contractor shall" is implied.

*(**Commentary:** Successful design and installation of high-quality micropiles require experienced Contractors having the necessary specialty drilling and grouting equipment and expertise and experienced work crews. **The most important section of the specifications to be enforced by the Owner deals with the experience qualifications of the micropile Contractor.** Failure to enforce the specified experience qualifications opens the door for inexperienced Contractors trying to cut costs. The results often are inferior workmanship, project delays, and project claims that, more often than not, substantially increase project costs. Results like these often discourage project Owners from implementing new technology, and draws them back to more traditional methods at any cost. This can be avoided with the proper specification implementation and, as importantly, enforcement to ensure a mutually successful project.)*

1.1 Micropile Contractor's Experience Requirements And Submittal.

The micropile Contractor shall be experienced in the construction and load testing of micropiles and have successfully constructed at least 5 projects in the last 5 years involving construction totalling at least 100 micropiles of similar capacity to those required in these plans and specifications.

The Contractor shall have previous micropile drilling and grouting experience in soil/rock similar to project conditions. The Contractor shall submit construction details, structural details and load test results for at least three previous successful micropile load tests from different projects of similar scope to this project.

The Contractor shall assign an Engineer to supervise the work with experience on at least 3 projects of similar scope to this project completed over the past 5 years. The Contractor shall not use consultants or manufacturers' representatives to satisfy the supervising Engineer requirements of this section. The on-site foremen and drill rig operators shall also have experience on at least 3 projects over the past 5 years installing micropiles of equal or greater capacity than required in these plans and specifications.

The micropiles shall be designed by a Registered Professional Engineer with experience in the design of at least 3 successfully completed micropile projects over the past 5 years, with micropiles of similar capacity to those required in these plans and specifications. The micropile designer may be either a employee of the Contractor or a separate Consultant designer meeting the stated experience requirements. *(**Commentary:** If the Owner prepares a fully detailed design, this paragraph can be deleted).*

At least 45 calendar days before the planned start of micropile construction, the Contractor shall submit 5 copies of the completed project reference list and a personnel list. The project reference list shall include a brief project description with the owner's name and current phone number and load test reports. The personnel list shall identify the micropile system designer (if applicable), supervising project Engineer, drill rig operators, and on-site foremen to be assigned to the project. The personnel list shall contain a summary of each individual's experience and be complete enough for the Engineer to determine whether each individual satisfies the required qualifications. The Engineer will approve or reject the Contractor's qualifications within 15 calendar days after receipt of a complete submission. Additional time required due to incomplete or unacceptable submittals will not be cause for time extension or impact or delay claims. All costs associated with incomplete or unacceptable submittals shall be borne by the Contractor.

Work shall not be started, nor materials ordered, until the Engineer's written approval of the Contractor's experience qualifications is given. The Engineer may suspend the Work if the Contractor uses non-approved personnel. If work is suspended, the Contractor shall be fully liable for all resulting costs and no adjustment in contract time will result from the suspension.

1.2 Pre-approved List

(***Commentary***: *The intent of this section, if used, is to reduce the administrative burden on specialty contractors who have previously satisfactorily demonstrated to the Owner Agency that they meet the Section A1.1 experience qualifications.*)

Listed below are the micropile specialty Contractors pre-qualified to design, furnish and install micropiles on this project, based on previous Contractor experience submittals verified and accepted by the Owner:

1. Contractor Name
   Mailing Address
   Contact Name
   Phone Number

2. Contractor Name
   Mailing Address
   Contact Name
   Phone Number

3. Contractor Name
   Mailing Address
   Contact Name
   Phone Number

The above named micropile specialty Contractors do not have to submit the experience qualification requirements called for in Section 1.1. The Section 1.8 design and Section 1.9 construction submittals are still required. The prime Contractor shall name the intended micropile specialty Contractor in the bid submittal documents under _________.
*(**Commentary:** Be careful not to automatically pre-approve Contractors for all projects based on pre-approval for one project. The projects may be very different.)*

The Section A1.1 experience qualifications and submittal requirements shall apply to other micropile specialty contractors not named on the pre-approved list.

1.3 Related Specifications

*(**Commentary:** Engineer to specify all related specifications.)*

1.4 Definitions

*(**Commentary:** Engineer to specify any additional definitions.)*

Admixture: Substance added to the grout to control bleed and/or shrinkage, improve flowability, reduce water content, or retard setting time.

Alignment Load (AL): A minimum initial load (5 percent DL maximum) applied to micropile during testing to keep the testing equipment correctly positioned.

Allowable Geotechnical Bond Load: For Service Load Design (SLD), computed as the nominal grout-to-ground bond strength ($\alpha_{\text{bond nominal strength}}$), divided by the geotechnical safety factor and then multiplied by the grouted bond surface area (bond length times drillhole circumference).

Bonded Length: The length of the micropile that is bonded to the ground and conceptually used to transfer the applied axial loads to the surrounding soil or rock. Also known as the load transfer length.

Bond-breaker: A sleeve placed over the steel reinforcement to prevent load transfer.

Casing: Steel tube introduced during the drilling process in overburden soil to temporarily stabilize the drill hole. This is usually withdrawn as the pile is grouted, although in certain types of micropiles, some casing is permanently left in place to provide added pile reinforcement.

Centralizer: A device to support and position the reinforcing steel in the drill hole and/or casing so that a minimum grout cover is provided.

Contractor: The person/firm responsible for performing the micropile work.

Coupler: The means by which load capacity can be transmitted from one partial length of reinforcement to another.

Creep Movement: The movement that occurs during the creep test of a micropile under a constant load.

Design Load (DL): The maximum unfactored load expected to be applied to the micropile during its service life.

Encapsulation: A corrugated or deformed tube protecting the reinforcing steel against corrosion.

Engineer: The Owner or Owner's authorized agent.

Free (unbonded) length: The designed length of the micropile that is not bonded to the surrounding ground or grout.

Geotechnical Bond Design Strength: For Load Factor Design (LFD), computed as the nominal grout-to-ground bond strength ($\alpha_{\text{bond nominal strength}}$), multiplied by a geotechnical resistance factor φ_G. Use $\varphi_G = 0.6$ for typical designs and non-seismic load groups; use $\varphi_G = 1.0$ for seismic loads groups

Micropile: A small-diameter, bored, cast-in-place composite pile, in which the applied load is resisted by steel reinforcement, cement grout and frictional grout/ground bond.

Maximum Test Load: The maximum load to which the micropile is subjected during testing. Recommended as 2.5 × DL for verification load tests and as 1.67 × DL for proof load tests.

Nominal Grout-to-Ground Bond Strength: The estimated ultimate geotechnical unit grout-to-ground bond strength selected for use in design. Same as $\alpha_{\text{bond nominal strength}}$ (SLD and LFD)

Overburden: Material, natural or placed, that may require cased drilling methods to provide an open borehole to underlying strata.

Post-grouting: The injection of additional grout into the load transfer length of a micropile after the primary grout has set. Also known as regrouting or secondary grouting.

Primary Grout: Portland-cement-based grout injected into the micropile hole prior to or after the installation of the reinforcement to direct the load transfer to the surrounding ground along the micropile.

Proof Load Test: Incremental loading of a production micropile, recording the total movement at each increment.

Reinforcement: The steel component of the micropile that accepts and/or resists applied loadings.

Sheathing: Smooth or corrugated piping or tubing that protects the reinforcing steel against corrosion.

Spacer: A device to separate elements of a multiple-element reinforcement.

Verification Load Test: Pile load test performed to verify the design of the pile system and the construction methods proposed, prior to installation of production piles.

1.5 Referenced Codes and Standards.

The following publications form a part of this specification to the extent indicated by the references. The latest publication as of the issue date of this specification shall govern, unless indicated otherwise.

1.5.1 American Society for Testing and Materials (ASTM)

American Association of State Highway and Transportation Officials (AASHTO)

ASTM	AASHTO	SPECIFICATION / TEST
A36, A572	M183, M223	Structural Steel
A82	M55	Cold-Drawn Steel Wire for Concrete Reinforcement
A252	–	Welded and Seamless Steel Pipe Piles
A615	M31	Deformed and Plain Billet Steel Bars for Concrete Reinforcement
A722	M275	Uncoated High-Strength Steel Bar for Prestressing Concrete
A775	–	Epoxy -Coated Reinforcing Steel Bars
A934	–	Epoxy-Coated Prefabricated Steel Reinforcing Bars
C 33	M80	Concrete Aggregates
C 109	T106	Compressive Strength of Hydraulic Cement Mortar
C 188	T133	Density of Hydraulic Cement
C 144	M45	Aggregate for Masonry Mortar
C 150	M 85	Portland Cement
C 494	M194	Chemical Admixtures for Concrete
D 1143	–	Method of Testing Piles Under Static Axial Compressive Load
D 1784	–	Polyvinyl Chloride (PVC) Pipe (Class 13464-B)
D 3350	M 252	Polyethylene Corrugated Tubing
D 3689	–	Method of Testing Individual Piles Under Static Axial Tensile Load
D 3966		Standard Test Method for Piles Under Lateral Load
–	T 26	Quality of Water to be Used in Concrete

1.5.2 American Welding Society (AWS)

D1.1 Structural Welding Code-Steel

D1.2 Structural Welding Code-Reinforcing Steel

1.5.3 American Petroleum Institute (API)

5CT (N-80)	Specification for casing and tubing
RP 13B-1	Recommended Practice – Standard Procedure for Field Testing Water Based Drilling Fluids

1.6 Available Information.

Available information developed by the Owner, or by the Owner's duly authorized representative include the following items:

1. Plans prepared by ___________ , dated __________. The plans include the plan view, profile and typical cross sections for the proposed micropile locations. ***(Commentary:*** *Refer to chapter 9 of the FHWA "Micropile Design and Construction Guidelines Manual", Report No. FHWA- SA-97-070 for detailed guidance on plan information to provide on the Owner-Controlled Design preliminary plans. An example preliminary plan for the bridge foundation support design example no. 1 is included at the end of this guide specification.)*

2. Geotechnical Report No.(s) ________ titled __________, dated__________ , included or referenced in the bid documents, contains the results of test pits, exploratory borings and other site investigation data obtained in the vicinity of the proposed micropile locations.

(Commentary: *The subsurface conditions expected can significantly impact the contractor's choice of procedures, methods, or equipment, the bidding process, and contract administration. Experience has proven that use of a geotechnical summary is advantageous* *toward achieving a successful contract. It is recommended that advisory wording be inserted into the contract special provisions to "red flag" the conditions to bidders. Preparation and use of a "Summary of Geotechnical Conditions", such as used by the Washington State DOT, is recommended. WSDOT's in-house guide for preparation of said summary, along with an example writeup for the Sample Problem No.1 contained in this manual, are included in Appendix B. WSDOT inserts the Summary into the contract special provisions, making it a legal part of the contract documents. The purpose is to alert and be fair to bidders, and thus prevent/minimize differing site condition construction claims and dispute.)*

1.7 Construction Site Survey

Before bidding the Work, the Contractor shall review the available subsurface information and visit the site to assess the site geometry, equipment access conditions, and location of existing structures and above ground facilities.

The Contractor is responsible for field locating and verifying the location of all utilities shown on the plans prior to starting the Work. Maintain uninterrupted service for those utilities designated to remain in service throughout the Work. Notify the Engineer of any utility locations different from shown on the plans that may require micropile relocations or structure design modification. Subject to the Engineer's approval, additional cost to the Contractor due to micropile relocations and/or structure design modification resulting from utility locations different from shown on the plans, will be paid as Extra Work.

***(Commentary:** The location of both active and abandoned buried utilities within the ground mass to receive micropiles can have a profound impact on the design and construction of the micropiles. Careful consideration of the presence and location of all utilities is required for successful design and installation of micropiles.)*

Prior to start of any micropile construction activity, the Contractor and Engineer shall jointly inspect the site to observe and document the pre-construction condition of the site, existing structures and facilities.

1.8 Micropile Design Requirements.

The micropiles shall be designed to meet the specified loading conditions, as shown on the contract plans and approved working drawings. Design the micropiles and pile top to footing connections using the Service Load Design (SLD) procedures contained in the FHWA "Micropile Design and Construction Guidelines Manual", Report No. FHWA- SA-97-070. *(**Commentary:** The FHWA micropile manual- Chapter 5- also presents Load Factor Design (LFD) procedures for micropile foundations. Revise specification if LFD design is required.*)

The required geotechnical safety factors/strength factors (for SLD Design) or load and resistance factors (for LFD Design) shall be in accord with the FHWA manual, unless specified otherwise. Estimated soil/rock design shear strength parameters, unit weights, applied foundation loadings, slope and external surcharge loads, corrosion protection requirements, known utility locations, easements, right-of-ways and other applicable design criteria will be as shown on the plans or specified herein. Structural design of any individual micropile structure elements not covered in the FHWA manual shall be by the service load design method in conformance with appropriate articles of the most current Edition of the AASHTO Standard Specifications for Highway Bridges, including current interim specifications.

Steel pipe used for micropile permanent casing shall incorporate an additional _____ mm thickness of sacrificial steel for corrosion protection. *(**Commentary**: This paragraph is optional and to be selected by the designer or specified by the owner. AASHTO Section 4.5.7.4 Cross-Section Adjustment for Corrosion, states - "For concrete-filled pipe piles where corrosion may be expected, 1/16 inch (1.6mm) shall be deducted from the shell thickness to allow for reduction in section due to corrosion.")*

Where required as shown on the contract plans, corrosion protection of the internal steel reinforcing bars, consisting of either encapsulation, epoxy coating, or grout, shall be provided in accordance with Materials Section 2.0. Where permanent casing is used for a portion of the micropile, encapsulation shall extend at least 1.5 m into the casing.

*(**Commentary:** When installation of micropiles provides additional support for existing structures, such as for seismic retrofit or underpinning applications, the structural designer should add appropriate specification verbage and design criteria into this specification to cover the penetration of the existing structural elements and the top of pile anchorage to the existing structure.)*

1.8.1 Micropile Design Submittals.

At least 21 calendar days before the planned start of micropile structure construction, submit complete design calculations and working drawings to the Engineer for review and approval. Include all details, dimensions, quantities, ground profiles, and cross-sections necessary to construct the micropile structure. Verify the limits of the micropile structure and ground survey data before preparing the detailed working drawings.

The drawings and calculations shall be signed and sealed by the contractor's Professional Engineer or by the Consultant designer's Professional Engineer (if applicable), previously approved by the owner's Engineer. If the micropile contractor uses a consultant designer to prepare the design, the micropile contractor shall still have overall contract responsibility for both the design and the construction.

1.8.2 Design Calculations.

Design calculations shall include, but not be limited to, the following items:

1. A written summary report which describes the overall micropile design.
2. Applicable code requirements and design references.
3. Micropile structure critical design cross-section(s) geometry including soil/rock strata and piezometric levels and location, magnitude and direction of design applied loadings, including slope or external surcharge loads .

4. Design criteria including, soil/rock shear strengths (friction angle and cohesion), unit weights, and ground-grout bond values and micropile drillhole diameter assumptions for each soil/rock strata.

5. Safety factors/strength factors (for Service Load Design) or load and resistance factors (for Load Factor Design) used in the design on the ground-grout bond values, surcharges, soil/rock and material unit weights, steel, grout, and concrete materials.

6. Seismic design earthquake acceleration coefficient.

7. Design calculation sheets (both static and seismic) with the project number, micropile structure location, designation, date of preparation, initials of designer and checker, and page number at the top of each page. Provide an index page with the design calculations.

8. Design notes including an explanation of any symbols and computer programs used in the design.

9. Pile to footing connection calculations.

1.8.3 Working Drawings.

The working drawings shall include all information required for the construction and quality control of the piling. Working drawings shall include, but not be limited to, the following items unless provided in the contract plans:

1. A plan view of the micropile structure(s) identifying:

   (a) A reference baseline and elevation datum.

   (b) The offset from the construction centerline or baseline to the face of the micropile structure at all changes in horizontal alignment.

   (c) Beginning and end of micropile structure stations.

(d) Right-of-way and permanent or temporary construction easement limits, location of all known active and abandoned existing utilities, adjacent structures or other potential interferences. The centerline of any drainage structure or drainage pipe behind, passing through, or passing under the micropile structure.

(e) Subsurface exploration locations shown on a plan view of the proposed micropile structure alignment with appropriate reference base lines to fix the locations of the explorations relative to the micropile structure.

2. An elevation view of the micropile structure(s) identifying:

    (a) Elevation view showing micropile locations and elevations; vertical and horizontal spacing; batter and alignment and the location of drainage elements (if applicable).

    (b) Existing and finish grade profiles both behind and in front of the micropile structure.

3. Design parameters and applicable codes.

4. General notes for constructing the micropile structure including construction sequencing or other special construction requirements.

5. Horizontal and vertical curve data affecting the micropile structure and micropile structure control points. Match lines or other details to relate micropile structure stationing to centerline stationing.

6. A listing of the summary of quantities on the elevation drawing of each micropile structure showing pay item estimated quantities.

7. Micropile typical sections including micropile spacing and inclination; minimum drillhole diameter; pipe casing and reinforcing bar sizes and details; splice types and locations; centralizers and spacers; grout bond zone and casing plunge lengths (if used);

corrosion protection details; and connection details to the substructure footing, anchorage, plates, etc.

8. A typical detail of verification and production proof test micropiles defining the micropile length, minimum drillhole diameter, inclination, and load test bonded and unbonded test lengths.

9. Details, dimensions, and schedules for all micropiles, casing and reinforcing steel, including reinforcing bar bending details.

10. Details for constructing micropile structures around drainage facilities (if applicable).

The working drawings and design calculations shall be signed and sealed by the Contractor's Professional Engineer or by the Consultant designer's Professional Engineer (if applicable), previously pre-qualified by the Owner. If the micropile Contractor uses a Consultant designer to prepare the design, the micropile Contractor shall still have overall contract responsibility for both the design and the construction.

Submit 5 sets of the working drawings with the initial submission. Drawing sheet size shall be 550 by 850 mm. One set will be returned with any indicated corrections. The Engineer will approve or reject the Contractor's submittal within 15 calendar days after receipt of a complete submission. If revisions are necessary, make the necessary corrections and resubmit 5 revised sets. When the drawings are approved, furnish 5 sets and a Mylar sepia set of the approved drawings. The Contractor will not be allowed to begin micropile structure construction or incorporate materials into the work until the submittal requirements are satisfied and found acceptable to the Engineer. Changes or deviations from the approved submittals must be re-submitted for approval. No adjustments in contract time or delay or impact claims will be allowed due to incomplete submittals. *(**Commentary:** Submittals procedures shall be coordinated with Owner/Agency procedures).*

Revise the drawings when plan dimensions are changed due to field conditions or for other reasons. Within 30 days after completion of the work, submit as-built drawings to the

Engineer. Provide revised design calculations signed by the approved Registered Professional Engineer for all design changes made during the construction of the micropile structure.

1.9 Construction Submittals.

The Contractor shall prepare and submit to the Engineer, for review of completeness, 5 copies of the following for the micropile system or systems to be constructed:

1. Detailed step-by-step description of the proposed micropile construction procedure, including personnel, testing and equipment to assure quality control. This step-by-step procedure shall be shown on the working drawings in sufficient detail to allow the Engineer to monitor the construction and quality of the micropiles.

2. Proposed start date and time schedule and micropile installation schedule providing the following:

    Micropile number
    Micropile design load
    Type and size of reinforcing steel
    Minimum total bond length
    Total micropile length
    Micropile top footing attachment

3. If welding of casing is proposed, submit the proposed welding procedure, certified by a qualified welding specialist.

4. Information on headroom and space requirements for installation equipment that verify the proposed equipment can perform at the site.

5. Plan describing how surface water, drill flush, and excess waste grout will be controlled and disposed.

6. Certified mill test reports for the reinforcing steel or coupon test results for permanent casing without mill certification. The ultimate strength, yield strength, elongation, and material properties composition shall be included. For API N-80 pipe casing, coupon test results may be submitted in lieu of mill certification.

7. Proposed Grouting Plan. The grouting plan shall include complete descriptions, details, and supporting calculations for the following:

   (a) Grout mix design and type of materials to be used in the grout including certified test data and trial batch reports.

   (b) Methods and equipment for accurately monitoring and recording the grout depth, grout volume and grout pressure as the grout is being placed.

   (c) Grouting rate calculations, when requested by the Engineer. The calculations shall be based on the initial pump pressures or static head on the grout and losses throughout the placing system, including anticipated head of drilling fluid (if applicable) to be displaced.

   (d) Estimated curing time for grout to achieve specified strength. Previous test results for the proposed grout mix completed within one year of the start of grouting may be submitted for initial verification and acceptance and start of production work. During production, grout shall be tested in accord with Section 3.4.5.

   (e) Procedure and equipment for Contractor monitoring of grout quality.

8. Detailed plans for the proposed micropile load testing method. This shall include all drawings, details, and structural design calculations necessary to clearly describe the proposed test method, reaction load system capacity and equipment setup, types and accuracy of apparatus to be used for applying and measuring the test loads and pile top movements in accordance with Section 3.6, Pile Load Tests.

9. Calibration reports and data for each test jack, pressure gauge and master pressure gauge and electronic load cell to be used. The calibration tests shall have been performed by an independent testing laboratory, and tests shall have been performed within 90 calendar days of the date submitted. Testing shall not commence until the Engineer has reviewed and accepted the jack, pressure gauge, master pressure gauge and electronic load cell calibration data.

Work other than test pile installation shall not begin until the construction submittals have been received, reviewed, and accepted in writing by the Engineer. Provide submittal items 1 through 5 at least 21 calendar days prior to initiating micropile construction, item 7 as the work progresses for each delivery and submittal items 6, 8 and 9 at least 7 days prior to start of micropile load testing or incorporation of the respective materials into the work. The Contractor shall allow the Engineer 7 calendar days to review the construction submittals after a complete set has been received. Additional time required due to incomplete or unacceptable submittals shall not be cause for delay or impact claims. All costs associated with incomplete or unacceptable Contractor submittals shall be the responsibility of the Contractor.

1.10 Pre-construction Meeting.

A pre-construction meeting will be scheduled by the Engineer and held prior to the start of micropile construction. The Engineer, prime Contractor, micropile specialty Contractor, micropile designer, excavation Contractor and geotechnical instrumentation specialist (if applicable) shall attend the meeting. Attendance is mandatory. The pre-construction meeting will be conducted to clarify the construction requirements for the work, to coordinate the construction schedule and activities, and to identify contractual relationships and delineation of responsibilities amongst the prime Contractor and the various Subcontractors - specifically those pertaining to excavation for micropile structures, anticipated subsurface conditions, micropile installation and testing, micropile structure survey control and site drainage control.

2.0 MATERIALS.

Furnish materials new and without defects. Remove defective materials from the jobsite at no additional cost. Materials for micropiles shall consist of the following:

Admixtures for Grout: Admixtures shall conform to the requirements of ASTM C 494/AASHTO M194. Admixtures that control bleed, improve flowability, reduce water content, and retard set may be used in the grout, subject to the review and acceptance of the Engineer. Admixtures shall be compatible with the grout and mixed in accordance with the manufacturer's recommendations. Expansive admixtures shall only be added to the grout used for filling sealed encapsulations and anchorage covers. Accelerators are not permitted. Admixtures containing chlorides are not permitted.

Cement: All cement shall be Portland cement conforming to ASTM C 150/AASHTO M85, Types II, III or V.

Centralizers and Spacers: Centralizers and spacers shall be fabricated from schedule 40 PVC pipe or tube, steel, or material non-detrimental to the reinforcing steel. Wood shall not be used. Centralizers and spacers shall be securely attached to the reinforcement; sized to position the reinforcement within 10 mm of plan location from center of pile; sized to allow grout tremie pipe insertion to the bottom of the drillhole; and sized to allow grout to freely flow up the drillhole and casing and between adjacent reinforcing bars.

Encapsulation: Encapsulation (double corrosion protection) shall be shop fabricated using high-density, corrugated polyethylene tubing conforming to the requirements of ASTM D3350/AASHTO M252 with a nominal wall thickness of 0.8 mm. The inside annulus between the reinforcing bars and the encapsulating tube shall be a minimum of 5mm and be fully grouted with non-shrink grout conforming to Materials Section 2.0.

Epoxy Coating: The minimum thickness of coating applied electrostatically to the reinforcing steel shall be 0.3 mm. Epoxy coating shall be in accordance with ASTM A775 or ASTM

A934. Bend test requirements are waived. Bearing plates and nuts encased in the pile concrete footing need not be epoxy coated.

Fine Aggregate: If sand - cement grout is used, sand shall conform to ASTM C 144/AASHTO M45.

Grout: Neat cement or sand/cement mixture with a minimum 3-day compressive strength of 14 MPa and a 28-day compressive strength of 28 MPa per AASHTO T106/ASTM C109. *(**Commentary:** Note to designers/specifiers. A 28-day grout strength of 28 MPa is common for micropiles. If the micropile design calls for a higher grout strength, revise the specification accordingly.)*

Grout Protection: Provide a minimum 25 mm grout cover over bare or epoxy coated bars (excluding bar couplers) or minimum 12 mm grout cover over the encapsulation of encapsulated bars.

Permanent Casing Pipe: Permanent steel casing/pipe shall have the diameter and at least minimum wall thickness shown on the approved Working Drawings. The permanent steel casing/pipe:

1. shall meet the Tensile Requirements of ASTM A252, Grade 3, except the yield strength shall be a minimum of 345 MPa to 552 MPa as used in the design submittal.
2. may be new "Structural Grade" (a.k.a. "Mill Secondary") steel pipe meeting above but without Mill Certification, free from defects (dents, cracks, tears) and with two coupon tests per truckload delivered to the fabricator.

For permanent casing/pipe that will be welded, the following material conditions apply:

1. the carbon equivalency (CE) as defined in AWS D1.l, Section X15.1, shall not exceed 0.45, as demonstrated by mill certifications
2. the sulfur content shall not exceed 0.05%, as demonstrated by mill certifications

For permanent casing/pipe that will be shop or field welded, the following fabrication or construction conditions apply:

1. the steel pipe shall not be joined by welded lap splicing
2. welded seams and splices shall be complete penetration welds
3. partial penetration welds may be restored in conformance with AWS D1.1
4. the proposed welding procedure certified by a welding specialist shall be submitted for approval

Threaded casing joints shall develop at least the required nominal resistance used in the design of the micropile.

*(**Commentary:** From a practical standpoint, the adequacy of pipe and reinforcing bar splices and threaded joint connections will be verified by the verification and proof load testing).*

Plates and Shapes: Structural steel plates and shapes for pile top attachments shall conform to ASTM A 36/AASHTO M183, or ASTM A 572/AASHTO M223, Grade 350.

Reinforcing Bars: Reinforcing steel shall be deformed bars in accordance with ASTM A 615/AASHTO M31, Grade 420 or Grade 520 or ASTM A 722/AASHTO M275, Grade 1035. When a bearing plate and nut are required to be threaded onto the top end of reinforcing bars for the pile top to footing anchorage, the threading may be continuous spiral deformed ribbing provided by the bar deformations (e.g., Dywidag or Williams continuous threadbars) or may be cut into a reinforcing bar. If threads are cut into a reinforcing bar, the next larger bar number designation from that shown on the Plans shall be provided, at no additional cost.

Bar tendon couplers, if required, shall develop the ultimate tensile strength of the bars without evidence of any failure.

Reinforcing Bar Corrosion Protection:

*(**Commentary:** Corrosion protection requirements vary between Transportation Agencies. The most common and simplest tests utilized to measure the aggressiveness of the soil environment include electrical resistivity, pH, chloride, and sulfate. Per FHWA-RD-89-198, the ground is considered aggressive if any one of these indicators show critical values as detailed below:*

PROPERTY	TEST DESIGNATION*	CRITICAL VALUES*
Resistivity	*AASHTO T-288, ASTM G 57*	*below 2,000 ohm-cm*
pH	*AASHTO T-289, ASTM G 51*	*below 5*
Sulfate	*AASHTO T-290, ASTM D516M, ASTM D4327*	*above 200 ppm*
Chloride	*AASHTO T-291, ASTM D512, ASTM D4327*	*above 100 ppm*

** Specifier should check test standards for latest updates and individual transportation agencies may have limits on critical values different than tabulated above. Standard specifications or test methods for any of the above items which are common to your agency can be referenced in lieu of the above listed AASHTO/ASTM references.*

Sheathing: Smooth plastic sheathing, including joints, shall be watertight. Polyvinyl chloride (PVC) sheathing shall conform to ASTM D 1784, Class 13464-B.

Water: Water used in the grout mix shall conform to AASHTO T 26 and shall be potable, clean, and free from substances that may be injurious to cement and steel.

3.0 CONSTRUCTION REQUIREMENTS

3.1 Site Drainage Control.

The Contractor shall control and properly dispose of drill flush and construction related waste, including excess grout, in accord with the standard specifications and all applicable local codes and regulations. Provide positive control and discharge of all surface water that will affect construction of the micropile installation. Maintain all pipes or conduits used to control surface water during construction. Repair damage caused by surface water at no additional cost. Upon substantial completion of the Work, remove surface water control pipes or conduits from the site. Alternatively, with the approval of the Engineer, pipes or conduits that are left in place, may be fully grouted and abandoned or left in a way that protects the structure and all adjacent facilities from migration of fines through the pipe or conduit and potential ground loss.

Immediately contact the Engineer if unanticipated existing subsurface drainage structures are discovered during excavation or drilling. Suspend work in these areas until remedial measures meeting the Engineer's approval are implemented. Cost of remedial measures or repair work resulting from encountering unanticipated subsurface drainage structures, will be paid for as Extra Work.

3.2 Excavation

Coordinate the work and the excavation so the micropile structures are safely constructed. Perform the micropile construction and related excavation in accordance with the Plans and approved submittals. No excavations steeper than those specified herein or shown on the Plans will be made above or below the micropile structure locations without written approval of the Engineer.

3.3 Micropile Allowable Construction Tolerances

1. Centerline of piling shall not be more than 75 mm from indicated plan location.
2. Pile shall be plumb within 2 percent of total-length plan alignment.
3. Top elevation of pile shall be plus 25 mm or minus 50 mm maximum from vertical elevation indicated.
4. Centerline of reinforcing steel shall not be more than 15 mm from indicated location.

3.4 Micropile Installation

The micropile Contractor shall select the drilling method, the grouting procedure, and the grouting pressure used for the installation of the micropiles. The micropile Contractor shall also determine the micropile casing size, final drillhole diameter and bond length, and central tendon reinforcement steel sizing necessary to develop the specified load capacities and load testing requirements. The micropile Contractor is also responsible for estimating the grout take. There will be no extra payment for grout overruns. *(**Commentary:** Note, extra payment for grout takes is appropriate for micropiles in Karst. Otherwise, the bid price of these piles will be artificially high to cover risk of high grout loss.)*

3.4.1 Drilling

The drilling equipment and methods shall be suitable for drilling through the conditions to be encountered, without causing damage to any overlying or adjacent structures or services. The drillhole must be open along it's full length to at least the design minimum drillhole diameter prior to placing grout and reinforcement. *(**Commentary**: When micropile construction will occur in close proximity to settlement sensitive structures, recommend including the following sentence in the specification - Vibratory pile driving hammers shall not be used to advance casing.)*

Temporary casing or other approved method of pile drillhole support will be required in caving or unstable ground to permit the pile shaft to be formed to the minimum design drillhole diameter. The Contractor's proposed method(s) to provide drillhole support and to prevent detrimental ground movements shall be reviewed by the Engineer. Detrimental ground movement is defined as movement which requires remedial repair measures. Use of drilling fluid containing bentonite is not allowed. *(**Commentary**: The specification verbage related to drillhole support methods and difficulty of drilling may vary project to project depending on the subsurface conditions revealed by the subsurface investigation data. It is the micropile specialty contractor's responsibility to select the proper drilling equipment and methods for the site conditions. It is the owner's responsibility to provide the available subsurface information. For projects with difficult ground conditions, use of an "advisory specification" included in the contract documents is recommended. Refer to Appendix B for an example.)*

Costs of removal or remedial measures due to encountering unanticipated subsurface obstructions will be paid for as Extra Work.

3.4.2 Ground Heave or Subsidence.

During construction, the Contractor shall observe the conditions vicinity of the micropile construction site on a daily basis for signs of ground heave or subsidence. Immediately notify the Engineer if signs of movements are observed. Contractor shall immediately suspend or modify drilling or grouting operations if ground heave or subsidence is observed, if the micropile structure is adversely affected, or if adjacent structures are damaged from the drilling or grouting. If the Engineer determines that the movements require corrective action, the Contractor shall take corrective actions necessary to stop the movement or perform repairs. When due to the Contractor's methods or operations or failure to follow the specified/approved construction sequence, as determined by the Engineer, the costs of providing corrective actions will be borne by the Contractor. When due to differing site conditions, as determined by the Engineer, the costs of providing corrective actions will be paid as Extra Work.

3.4.3 Pipe Casing and Reinforcing Bars Placement and Splicing.

Reinforcement may be placed either prior to grouting or placed into the grout - filled drillhole before temporary casing (if used) is withdrawn. Reinforcement surface shall be free of deleterious substances such as soil, mud, grease or oil that might contaminate the grout or coat the reinforcement and impair bond. Pile cages and reinforcement groups, if used, shall be sufficiently robust to withstand the installation and grouting process and the withdrawal of the drill casings without damage or disturbance.

The Contractor shall check pile top elevations and adjust all installed micropiles to the planned elevations.

Centralizers and spacers (if used) shall be provided at 3-m centers maximum spacing. The upper and lower most centralizer shall be located a maximum of 1.5 m from the top and bottom of the micropile. Centralizers and spacers shall permit the free flow of grout without misalignment of the reinforcing bar(s) and permanent casing. The central reinforcement bars with centralizers shall be lowered into the stabilized drill hole and set. The reinforcing steel shall be inserted into the drill hole to the desired depth without difficulty. Partially inserted reinforcing bars shall not be driven or forced into the hole. Contractor shall redrill and reinsert reinforcing steel when necessary to facilitate insertion.

Lengths of casing and reinforcing bars to be spliced shall be secured in proper alignment and in a manner to avoid eccentricity or angle between the axes of the two lengths to be spliced. Splices and threaded joints shall meet the requirements of Materials Section 2.0. Threaded pipe casing joints shall be located at least two casing diameters (OD) from a splice in any reinforcing bar. When multiple bars are used, bar splices shall be staggered at least 0.3 meters.

3.4.4 Grouting.

Micropiles shall be primary grouted the same day the load transfer bond length is drilled. The Contractor shall use a stable neat cement grout or a sand cement grout with a minimum 28-day unconfined compressive strength of 28 MPa. Admixtures, if used, shall be mixed in accordance with manufacturer's recommendations. The grouting equipment used shall produce a grout free of lumps and undispersed cement. The Contractor shall have means and methods of measuring the grout quantity and pumping pressure during the grouting operations. The grout pump shall be equipped with a pressure gauge to monitor grout pressures. A second pressure gauge shall be placed at the point of injection into the pile top. The pressure gauges shall be capable of measuring pressures of at least 1 MPa or twice the actual grout pressures used, whichever is greater. The grout shall be kept in agitation prior to mixing. Grout shall be placed within one hour of mixing. The grouting equipment shall be sized to enable each pile to be grouted in one continuous operation. The grout shall be injected from the lowest point of the drill hole and injection shall continue until uncontaminated grout flows from the top of the pile. The grout may be pumped through grout tubes, casing, hollow-stem augers, or drill rods. Temporary casing, if used, shall be extracted in stages ensuring that, after each length of casing is removed the grout level is brought back up to the ground level before the next length is removed. The tremie pipe or casing shall always extend below the level of the existing grout in the drillhole. The grout pressures and grout takes shall be controlled to prevent excessive heave or fracturing of rock or soil formations. Upon completion of grouting, the grout tube may remain in the hole, but must be filled with grout.

If the Contractor elects to use a postgrouting system, Working Drawings and details shall be submitted to the Engineer for review in accordance with Section 1.8, Pre-installation Submittals.

3.4.5 Grout Testing

Grout within the micropile verification and proof test piles shall attain the minimum required 3-day compressive strength of 14 MPa prior to load testing. Previous test results for the proposed grout mix completed within one year of the start of work may be submitted for initial verification of the required compressive strengths for installation of pre-production verification test piles and initial production piles. During production, micropile grout shall be tested by the Contractor for compressive strength in accordance with AASHTO T106/ASTM C109 at a frequency of no less than one set of three 50-mm grout cubes from each grout plant each day of operation or per every 10 piles, whichever occurs more frequently. The compressive strength shall be the average of the 3 cubes tested.

Grout consistency as measured by grout density shall be determined by the Contractor per ASTM C 188/AASHTO T 133 or API RP-13B-1 at a frequency of at least one test per pile, conducted just prior to start of pile grouting. The Baroid Mud Balance used in accordance with API RP-13B-1 is an approved device for determining the grout density of neat cement grout. The measured grout density shall be between ________ kg/m^3 and ________ kg/m^3.

Grout samples shall be taken directly from the grout plant. Provide grout cube compressive strength and grout density test results to the Engineer within 24 hours of testing. *(**Commentary**: If the Engineer will perform the grout testing, revise this section accordingly).*

3.5 Micropile Installation Records.

Contractor shall prepare and submit to the Engineer full-length installation records for each micropile installed. The records shall be submitted within one work shift after that pile installation is completed. The data shall be recorded on the micropile installation log included at the end of this specification. A separate log shall be provided for each micropile. *(**Commentary**: In addition to the expertise of the micropile specialty Contractor, the quality of the individual construction elements is directly related to the final product overall quality. As with other drilled pile systems, the actual load carrying capacity of a micropile can only be definitively proven by pile load tests. It is not practical or economical to test every pile* *installed. Therefore, aggressive inspection by the Contractor and Owner's Engineer is needed to assure that each individual micropile is well constructed and to justify load testing only a small number, e.g, 5%, of the total number of production piles installed.)*

3.6 Pile Load Tests

Perform verification and proof testing of piles at the locations specified herein or designated by the Engineer. Perform compression load testing in accord with ASTM D1143 and tension load testing in accord with ASTM D3689, except as modified herein.

*(**Commentary on number of load tests:** Specifier/designer need to determine and write into this portion of the specification the number and location of required verification and proof tests. The total number of load tests and maximum test loads to be specified can vary on a project-by-project basis. They are dependent on ground type and variability , required pile capacity, pile loading type (i.e., static or seismic), total number of piles, criticality of the structure and available site access and work space. Guideline criteria for estimating the total number of verification and proof test piles are given in Chapter 7. For structure foundations, the following is recommended as a minimum. Perform verification testing of at least one sacrificial test pile per structure, prior to installation of any production piles. New users should perform proof tests on production piles at a frequency of 5 percent (1 in 20). For experienced users, number of tests are to be determined by Owner/Engineer on a project by project basis. If pile capacity demands are greatest in compression, the piles should be load tested in compression. If the pile capacity demands are equal for both compression and tension, or greater in tension, it is recommended that tension testing alone be conducted, to reduce costs).*

*(**Commentary:** Specifier - Indicate here whether compression or tension testing, or both are required for your project).*

3.6.1 Verification Load Tests

Perform pre-production verification pile load testing to verify the design of the pile system and the construction methods proposed prior to installing any production piles. __________ sacrificial verification test piles shall be constructed in conformance with the approved Working Drawings. Verification test pile(s) shall be installed at the following locations ___________.

Verification load tests shall be performed to verify that the Contractor installed micropiles will meet the required compression and tension load capacities and load test acceptance criteria and to verify that the length of the micropile load transfer bond zone is adequate. The micropile verification load test results must verify the Contractor's design and installation methods, and be reviewed and accepted by the Engineer prior to beginning installation of production micropiles.

The drilling-and-grouting method, casing length and outside diameter, reinforcing bar lengths, and depth of embedment for the verification test pile(s) shall be identical to those specified for the production piles at the given locations. The verification test micropile structural steel sections shall be sized to safely resist the maximum test load. *(**Commentary:** Note that if additional steel area is provided in the verification test, the measured deflection will be lower than production piles.)*

The maximum verification and proof test loads applied to the micropile shall not exceed 80 percent of the structural capacity of the micropile structural elements, to include steel yield in tension, steel yield or buckling in compression, or grout crushing in compression. Any required increase in strength of the verification test pile elements above the strength required for the production piles shall be provided for in the contractor's bid price.

The jack shall be positioned at the beginning of the test such that unloading and repositioning during the test will not be required. When both compression and tension load testing is to be performed on the same pile, the pile shall be tested under compression loads prior to testing under tension loads.

3.6.2 Testing Equipment and Data Recording.

Testing equipment shall include dial gauges, dial gauge support, jack and pressure gauge, electronic load cell, and a reaction frame. The load cell is required only for the creep test portion of the verification test. ***(Commentary:*** *The purpose and value of an electronic load cell is to measure small changes in load for load tests where the load is held for a long duration, such as during verification or creep testing. It is not intended to be used during proof testing, including the short term creep portion. Experience has proven that load cells have been problematic under field conditions, yet even with errors resulting from cell construction, off-center loading, and other effects, a load cell is very sensitive to small changes in load and is strongly recommended for creep testing.)* The contractor shall provide a description of test setup and jack , pressure gauge and load cell calibration curves in accordance with the Submittals Section.

Design the testing reaction frame to be sufficiently rigid and of adequate dimensions such that excessive deformation of the testing equipment does not occur. Align the jack, bearing plates, and stressing anchorage such that unloading and repositioning of the equipment will not be required during the test.

Apply and measure the test load with a hydraulic jack and pressure gauge. The pressure guage shall be graduated in 500 kPa increments or less. The jack and pressure gauge shall have a pressure range not exceeding twice the anticipated maximum test pressure. Jack ram travel shall be sufficient to allow the test to be done without resetting the equipment. Monitor the creep test load hold during verification tests with both the pressure gauge and the electronic load cell. Use the load cell to accurately maintain a constant load hold during the creep test load hold increment of the verification test.

Measure the pile top movement with a dial gauge capable of measuring to 0.025 mm. The dial gauge shall have a travel sufficient to allow the test to be done without having to reset the gauge. Visually align the gauge to be parallel with the axis of the micropile and support the gauge independently from the jack, pile or reaction frame. Use a minimum of two dial gauges when the test setup requires reaction against the ground or single reaction piles on each side of the test pile. ***(Commentary:*** *Experience with testing piles reacting against the ground, or against single reaction piles on each side of the test pile, has resulted in racking and misalignment of the system on some projects. Two dial guages are recommended for this test setup to determine if racking is occurring and to provide a more accurate average micropile head movement measurement).*

The required load test data shall be recorded by the Engineer.

3.6.3 Verification Test Loading Schedule.

Test verification piles designated for compression or tension load testing to a maximum test load of 2.5 times the micropile Design Load shown on the Plans or Working Drawings. ***(Commentary:*** *See Section 5.E.4 for more detailed verification load testing information.)* The verification pile load tests shall be made by incrementally loading the micropile in accordance with the following cyclic load schedule for both compression and tension loading:

AL = Alignment Load	DL = Design Load	
	LOAD	HOLD TIME
1	AL (0.05 DL)	1 minute
2	0.25 DL	1 minute
3	0.50 DL	1 minute
4	AL	1 minute
5	0.25 DL	1 minute
6	0.50 DL	1 minute
7	0.75 DL	1 minute
8	AL	1 minute
9	0.25 DL	1 minute
10	0.50 DL	1 minute
11	0.75 DL	1 minute
12	1.00 DL	1 minute

AL = Alignment Load	DL = Design Load	
	LOAD	HOLD TIME
13	AL	1 minute
14	0.25 DL	1 minute
15	0.50 DL	1 minute
16	0.75 DL	1 minute
17	1.00 DL	1 minute
18	1.33 DL	**60 minutes (Creep Test Load Hold)**
19	1.75 DL	1 minute
20	2.00 DL	1 minute
21	2.25 DL	1 minute
22	**2.50 DL (Maximum Test Load)**	10 minutes
23	AL	1 minute

The test load shall be applied in increments of 25 percent of the DL load. Each load increment shall be held for a minimum of 1 minute. Pile top movement shall be measured at each load increment. The load-hold period shall start as soon as each test load increment is applied. The verification test pile shall be monitored for creep at the 1.33 Design Load (DL). Pile movement during the creep test shall be measured and recorded at 1, 2, 3, 4, 5, 6, 10, 20, 30, 50, and 60 minutes. The alignment load shall not exceed 5 percent of the DL load. Dial gauges shall be reset to zero after the initial AL is applied.

The acceptance criteria for micropile verification load tests are:

1. The pile shall sustain the first compression or tension 1.0 DL test load with no more than _______mm total vertical movement at the top of the pile, relative to the position of the top of the pile prior to testing. *(**Commentary:** Structural designer to determine maximum allowable total pile top structural axial displacement at 1.0 DL test load based on structural design requirements. Also, if the verification test pile has to be upsized structurally to accommodate the maximum required verification test load, this*

provision will not apply. Only the proof tested production piles will then be subject to this criteria. Refer to Chapter 5 for more design guidance).

2. At the end of the 1.33 DL creep test load increment, test piles shall have a creep rate not exceeding 1 mm/log cycle time (1 to 10 minutes) or 2 mm/log cycle time (6 to 60 minutes or the last log cycle if held longer).The creep rate shall be linear or decreasing throughout the creep load hold period.

3. Failure does not occur at the 2.5 DL maximum test load. Failure is defined as load at which attempts to further increase the test load simply result in continued pile movement.

The Engineer will provide the Contractor written confirmation of the micropile design and construction within 3 working days of the completion of the verification load tests. This written confirmation will either confirm the capacities and bond lengths specified in the Working Drawings for micropiles or reject the piles based upon the verification test results.

3.6.4 Verification Test Pile Rejection

If a verification tested micropile fails to meet the acceptance criteria, the Contractor shall modify the design, the construction procedure, or both.These modifications may include modifying the installation methods, increasing the bond length, or changing the micropile type. Any modification that necessitates changes to the structure shall require the Engineer's prior review and acceptance. Any modifications of design or construction procedures or cost of additional verification test piles and load testing shall be at the Contractor's expense. At the completion of verification testing, test piles shall be removed down to the elevation specified by the Engineer.

3.6.5 Proof Load Tests

Perform proof load tests on the first set of production piles installed at each designated substructure unit prior to the installation of the remaining production piles in that unit. The first set of production piles is the number required to provide the required reaction capacity for the proof tested pile. The initial proof test piles shall be installed at the following substructure units _______________. Proof testing shall be conducted at a frequency of 5% (1 in 20) of the subsequent production piles installed, beyond the first 20, in each abutment and pier. Location of additional proof test piles shall be as designated by the Engineer. *(**Commentary:** The above is a guideline for new users. Experienced users may go with a lesser number of proof load tests as determined by the Owner/Engineer.)*

3.6.6 Proof Test Loading Schedule

Test piles designated for compression or tension proof load testing to a maximum test load of 1.67 times the micropile Design Load shown on the Plans or Working Drawings. *(**Commentary:** See Section 5.E.4 for more detailed proof load testing information.)* Proof tests shall be made by incrementally loading the micropile in accordance with the following schedule, to be used for both compression and tension loading:

AL = Alignment Load		DL = Design Load
	LOAD	**HOLD TIME**
1	AL	1 minute
2	0.25 DL	1 minute
3	0.50 DL	1 minute
4	0.75 DL	1 minute
5	1.00 DL	1 minute
6	1.33 DL	**10 or 60 minute Creep Test**
7	**1.67 DL (Maximum Test Load)**	1 minute
8	AL	1 minute

Depending on performance, either a 10 minute or 60 minute creep test shall be performed at the 1.33 DL Test Load. Where the pile top movement between 1 and 10 minutes exceeds 1 mm, the Maximum Test Load shall be maintained an additional 50 minutes. Movements shall be recorded at 1, 2, 3, 5, 6, 10, 20, 30, 50 and 60 minutes. The alignment load shall not exceed 5 percent of DL. Dial guages shall be reset to zero after the itital AL is applied.

The acceptance criteria for micropile proof load tests are:

1. The pile shall sustain the compression or tension 1.0 DL test load with no more than ______ mm total vertical movement at the top of the pile, relative to the position of the top of the pile prior to testing. *(**Commentary**: Structural designer to determine maximum allowable total pile top structural axial displacement at the 1.0 DL test load based on structure design requirements. Refer to Chapter 5 for more design guidance.)*
2. At the end of the 1.33 DL creep test load increment, test piles shall have a creep rate not exceeding 1 mm/log cycle time (1 to 10 minutes) or 2 mm/log cycle time (6 to 60 minutes).The creep rate shall be linear or decreasing throughout the creep load hold period.
3. Failure does not occur at the 1.67 DL maximum test load. Failure is defined as the load at which attempts to further increase the test load simply result in continued pile movement.

3.6.7 Proof Test Pile Rejection

If a proof-tested micropile fails to meet the acceptance criteria, the Contractor shall immediately proof test another micropile within that footing. For failed piles and further construction of other piles, the Contractor shall modify the design, the construction procedure, or both. These modifications may include installing replacement micropiles, incorporating piles at not more than 50% of the maximum load attained, postgrouting, modifying installation methods, increasing the bond length, or changing the micropile type. Any modification that necessitates changes to the structure design shall require the Engineer's prior review and acceptance. Any modifications of design or construction procedures, or cost of additional verification test piles and verification and/or proof load testing, or replacement production micropiles, shall be at the Contractor's expense.

4.0 METHOD OF MEASUREMENT.

Measurement will be made as follows for the quantity, as specified or directed by the Engineer:

- Mobilization will be measured on a lump-sum basis.
- Micropiles will be measured per each, installed, and accepted.
- Micropile verification load testing will be measured per each.
- Micropile proof load testing will be measured per each.

The final pay quantities will be the design quantity increased or decreased by any changes authorized by the Engineer.

5.0 BASIS OF PAYMENT

The quantities accepted for payment will be paid for at the contract unit prices for the following items:

Pay Item	Unit
Mobilization and Demobilization	Lump Sum
Micropile Verification Load Test	Each
Micropile Proof Load Test	Each
Micropiles	Each*
Micropiles Variations in Length to Top of Rock ..	LF**
Unexpected Obstruction Drilling	Hour***

*For the option where the contractor designs the footing and number of piles, the foundation system should be bid as lump sum and a schedule of values established for progress payments after award.

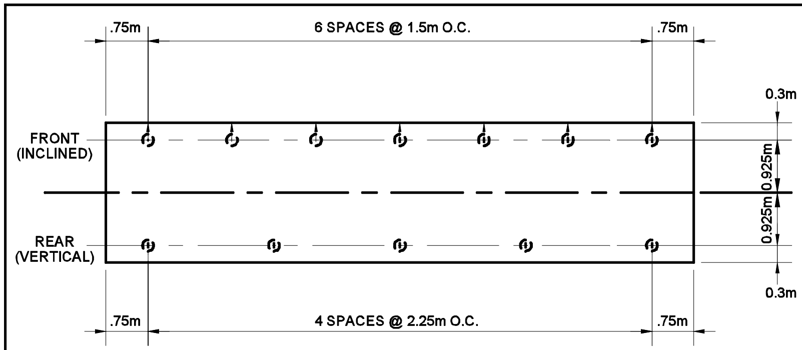
**Where piles are founded in rock, micropiles will be paid on a per each basis assuming Rock at Elevation _____. Additional length or shorter length due to variations in the top of rock will be paid on a add or deduct lineal foot basis where the linear footage = Elevation ___ minus Elevation of As-Built Rock.

***If "obstructions" are not defined in the Standard Specifications, a definition should be added.

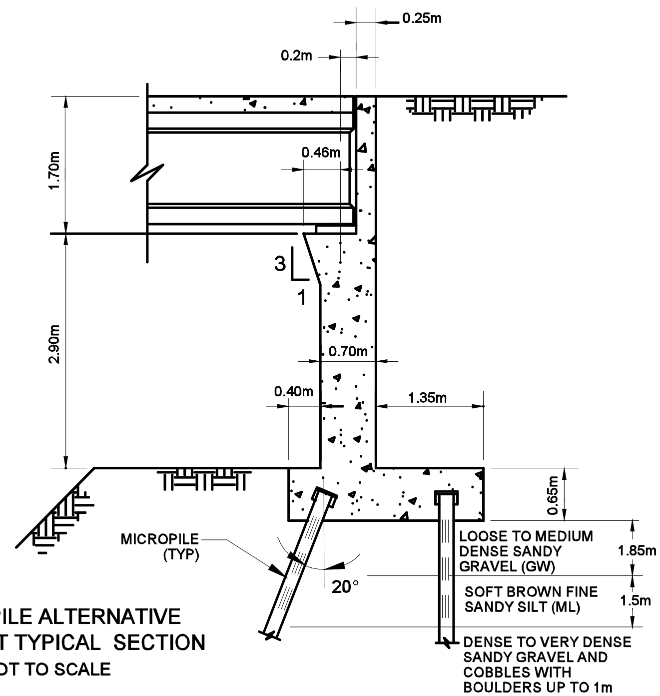
The contract unit prices for the above items will be full and complete payment for providing all design, materials, labor, equipment, and incidentals to complete the work.

Where verification test piles are designated as sacrificial, the micropile verification load test bid item shall include the cost of the sacrificial micropile.

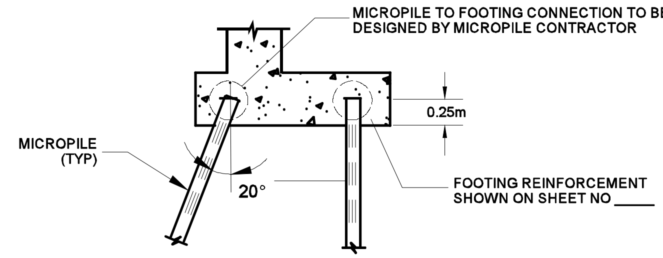
The unit contract amount for “Micropiles” shall include the drilling, furnishing, and placing the reinforcing steel and casing, grouting, and pile top attachments. The micropile Contractor is also responsible for estimating the grout take. There will be no extra payment for grout overruns.



MICROPILE ALTERNATIVE
ABUTMENT FOOTING – PLAN
NOT TO SCALE



MICROPILE ALTERNATIVE
ABUTMENT TYPICAL SECTION
NOT TO SCALE



MAXIMUM LOAD CAPACTY (SLD EXAMPLE)		
	COMPRESSION DESIGN CAP	TENSION DESIGN CAP
SERVICE LOAD DESIGN	595 kN	0 kN

GROUP 1 NON-SEISMIC UNFACTORED LOADS PER PILE

NOTES

1. MICROPILE DESIGN TO BE PERFORMED BY PREQUALIFIED MICROPILE CONTRACTOR IN ACCORDANCE WITH PROJECT SPECIFICATIONS & PILE CAPACITY TABLE ON THIS SHEET
2. MICROPILE BEARING PLATE SHALL EFFECTIVELY DISTRIBUTE THE DESIGN FORCE TO THE FOOTING CONCRETE SUCH THAT THE CONE SHEAR REQUIREMENTS OF ACI 349, APPENDIX B, ARE MET AND THE BENDING STRESS DOES NOT EXCEED 0.55f_y FOR STEEL.
3. MICROPILE BOND ZONE TO BE COMPLETELY FOUNDED IN DENSE TO VERY DENSE SANDY GRAVEL.

COMMENTARY: THIS ISTHE MINIMUM AMOUNT OF INFORMATION THAT THE OWNER NEEDS TO PROVIDE IN THE CONTRACT PLANS FOR THE OWNER CONTROLLED DESIGN SPECIFICATION METHOD. AS AN OPTION, OTHER FOOTING DESIGNS WITH ALTERNATE MICROPILE LAYOUTS CAN BE PROVIDED IN TABLE FORM.

CONTRACTOR DESIGN/BUILD
OF MICROPILES

SAMPLE CONTRACT PLANS

Micropile Installation Log

Project Name:	
Contract No.:	

Pile Designation #		Time @	
Installation Date		Start of Drilling	
Drill Rig/Drill Method		Start of Grouting	
Drill Rig/ #, Operator		Pile Completion	
Grout Plant #, Operator		Total Duration	

Drill Bit Type and Size		Cement Type	
Casing Dia./Wall Thickness		Admixtures	
Pile Inclination		w/c Ratio	
Reinforcement Size/Length			

Pile Length Above B.O.F.		Tremie Grout Quantity (bags)	
Upper Cased Length		Pressure Grout Quantity (bags)	
Cased and Bond Length (Plunge)		Grouting after plunge (bags)	
Bond Length Below Casing		Total Grout Quantity (bags)	
Total Pile Length		Grout Ratio (bags/m bond)	

Comments - Pile Drilling

Depth from B.O.F (m)	**Soil / Rock Description**	**Flush Description**	**Comments**

Comments - Pile Grouting

Depth from B.O.F (m)	**Pressure Range Max/Average (MPa)**	**Comments**

B.O.F. = Bottom of Footing (ref. FHWA-SA-97-070)

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APPENDIX A-2

Micropile Guide Construction Specification and Sample Plans

[Foundation Support Projects]

Contractor Design/Build of Foundation (Micropiles and Footings)

Metric (SI) Units

(With Commentary)

*(**Commentary:** This guide specification is set up for a post-bid design solicitation to solicit micropile structure designs where the Owner has selected a micropile system as the desired system for the given structure location(s). It can be modified as appropriate to also serve as a pre-bid design solicitation and /or for a solicitation where alternate foundation or structure types are allowed by the Owner, with the Contractor allowed to select and submit a design for the foundation or structure type which the Contractor feels is most cost-effective.* ***This guide specification is set up for the method wherein the Owner provides preliminary plans showing a pile footing design and total footing loads and moments for foundation support projects. Owner also provides related design criteria and requirements, subsurface data, rights-of-way limits, utility locations, site limitations, construction material and testing specifications and required Contractor working drawing/design and construction submittals and review requirements. <u>The micropile Contractor designs the individual micropile elements, including their spacing and layout, and pile top footing connections and selects the micropile construction process and equipment. As compared to the Appendix A-1 contracting method, with this approach the Design/Build Contractor has the flexibility to provide fewer higher capacity micropiles.</u>*** *During the bidding process, the **pre-qualified** micropile contractors prepare a micropile design and a firm cost proposal based on the Owner's preliminary plans and specifications. If the micropile portion of the project is to be subcontracted, general contractors will receive bids from the pre-qualified micropile contractors and include the best offer and name of the selected micropile Contractor in their bid submittal. Once the contract is awarded, the selected micropile Contractor prepares detailed micropile design calculations and working drawings and submits them to the Engineer for review. After acceptance of the design, construction begins. For more detailed discussion on various contracting methods, refer to Chapter 9.)*

1.0 DESCRIPTION

This work shall consist of constructing micropiles as shown on the contract plans and approved working drawings and as specified herein. The micropile specialty Contractor is responsible for furnishing of all design, materials, products, accessories, tools, equipment, services, transportation, labor and supervision, and manufacturing techniques required for design, installation and testing of micropiles and pile top attachments for this project.

The selected micropile Contractor shall select the micropile type, size, pile top attachment, installation means and methods, estimate the ground-grout bond value and determine the required grout bond length and final micropile diameter. The micropile Contractor shall design and install micropiles that will develop the load capacities indicated on the contract plans. The micropile load capacities shall be verified by verification and proof load testing as required and must meet the test acceptance criteria specified herein.

Where the imperative mood is used within this specification, "The Contractor shall" is implied.

*(**Commentary:** Successful design and installation of high-quality micropiles require experienced Contractors having the necessary specialty drilling and grouting equipment and expertise and experienced work crews. **The most important section of the specifications to be enforced by the Owner deals with the experience qualifications of the micropile Contractor.** Failure to enforce the specified experience qualifications opens the door for inexperienced Contractors trying to cut costs. The results often are inferior workmanship, project delays, and project claims that, more often than not, substantially increase project costs. Results like these often discourage project Owners from implementing new technology, and draws them back to more traditional methods at any cost. This can be avoided with the proper specification implementation and, as importantly, enforcement to ensure a mutually successful project.)*

1.1 Micropile Contractor’s Experience Requirements And Submittal.

The micropile Contractor shall be experienced in the construction and load testing of micropiles and have successfully constructed at least 5 projects in the last 5 years involving construction totalling at least 100 micropiles of similar capacity to those required in these plans and specifications.

The Contractor shall have previous micropile drilling and grouting experience in soil/rock similar to project conditions. The Contractor shall submit construction details, structural details and load test results for at least three previous successful micropile load tests from different projects of similar scope to this project.

The Contractor shall assign an Engineer to supervise the work with experience on at least 3 projects of similar scope to this project completed over the past 5 years. The Contractor shall not use consultants or manufacturers’ representatives to satisfy the supervising Engineer requirements of this section. The on-site foremen and drill rig operators shall also have experience on at least 3 projects over the past 5 years installing micropiles of equal or greater capacity than required in these plans and specifications.

The micropiles shall be designed by a Registered Professional Engineer with experience in the design of at least 3 successfully completed micropile projects over the past 5 years, with micropiles of similar capacity to those required in these plans and specifications. The micropile designer may be either a employee of the Contractor or a separate Consultant designer meeting the stated experience requirements. *(**Commentary:** If the Owner prepares a fully detailed design, this paragraph can be deleted).*

At least 45 calendar days before the planned start of micropile construction, the Contractor shall submit 5 copies of the completed project reference list and a personnel list. The project reference list shall include a brief project description with the owner's name and current phone number and load test reports. The personnel list shall identify the micropile system designer (if applicable), supervising project Engineer, drill rig operators, and on-site foremen to be assigned to the project. The personnel list shall contain a summary of each individual's experience and be complete enough for the Engineer to determine whether each individual satisfies the required qualifications. The Engineer will approve or reject the Contractor's qualifications within 15 calendar days after receipt of a complete submission. Additional time required due to incomplete or unacceptable submittals will not be cause for time extension or impact or delay claims. All costs associated with incomplete or unacceptable submittals shall be borne by the Contractor.

Work shall not be started, nor materials ordered, until the Engineer's written approval of the Contractor's experience qualifications is given. The Engineer may suspend the Work if the Contractor uses non-approved personnel. If work is suspended, the Contractor shall be fully liable for all resulting costs and no adjustment in contract time will result from the suspension.

1.2 Pre-approved List

(***Commentary****: The intent of this section, if used, is to reduce the administrative burden on specialty contractors who have previously satisfactorily demonstrated to the Owner Agency that they meet the Section A1.1 experience qualifications.*)

Listed below are the micropile specialty Contractors pre-qualified to design, furnish and install micropiles on this project, based on previous Contractor experience submittals verified and accepted by the Owner:

1. Contractor Name
   Mailing Address
   Contact Name
   Phone Number

2. Contractor Name
   Mailing Address
   Contact Name
   Phone Number

3. Contractor Name
   Mailing Address
   Contact Name
   Phone Number

The above named micropile specialty Contractors do not have to submit the experience qualification requirements called for in Section 1.1. The Section 1.8 design and Section 1.9 construction submittals are still required. The prime Contractor shall name the intended micropile specialty Contractor in the bid submittal documents under _________.
*(**Commentary:** Be careful not to automatically pre-approve Contractors for all projects based on pre-approval for one project. The projects may be very different.)*

The Section A1.1 experience qualifications and submittal requirements shall apply to other micropile specialty contractors not named on the pre-approved list.

1.3 Related Specifications

*(**Commentary:** Engineer to specify all related specifications.)*

1.4 Definitions

*(**Commentary:** Engineer to specify any additional definitions.)*

Admixture: Substance added to the grout to control bleed and/or shrinkage, improve flowability, reduce water content, or retard setting time.

Alignment Load (AL): A minimum initial load (5 percent DL maximum) applied to micropile during testing to keep the testing equipment correctly positioned.

Allowable Geotechnical Bond Load: For Service Load Design (SLD), computed as the nominal grout-to-ground bond strength ($\alpha_{\text{bond nominal strength}}$), divided by the geotechnical safety factor and then multiplied by the grouted bond surface area (bond length times drillhole circumference).

Bonded Length: The length of the micropile that is bonded to the ground and conceptually used to transfer the applied axial loads to the surrounding soil or rock. Also known as the load transfer length.

Bond-breaker: A sleeve placed over the steel reinforcement to prevent load transfer.

Casing: Steel tube introduced during the drilling process in overburden soil to temporarily stabilize the drill hole. This is usually withdrawn as the pile is grouted, although in certain types of micropiles, some casing is permanently left in place to provide added pile reinforcement.

Centralizer: A device to support and position the reinforcing steel in the drill hole and/or casing so that a minimum grout cover is provided.

Contractor: The person/firm responsible for performing the micropile work.

Coupler: The means by which load capacity can be transmitted from one partial length of reinforcement to another.

Creep Movement: The movement that occurs during the creep test of a micropile under a constant load.

Design Load (DL): The maximum unfactored load expected to be applied to the micropile during its service life.

Encapsulation: A corrugated or deformed tube protecting the reinforcing steel against corrosion.

Engineer: The Owner or Owner's authorized agent.

Free (unbonded) length: The designed length of the micropile that is not bonded to the surrounding ground or grout.

Geotechnical Bond Design Strength: For Load Factor Design (LFD), computed as the nominal grout-to-ground bond strength ($\alpha_{\text{bond nominal strength}}$), multiplied by a geotechnical resistance factor φ_G. Use $\varphi_G = 0.6$ for typical designs and non-seismic load groups; use $\varphi_G = 1.0$ for seismic loads groups

Micropile: A small-diameter, bored, cast-in-place composite pile, in which the applied load is resisted by steel reinforcement, cement grout and frictional grout/ground bond.

Maximum Test Load: The maximum load to which the micropile is subjected during testing. Recommended as 2.5 × DL for verification load tests and as 1.67 × DL for proof load tests.

Nominal Grout-to-Ground Bond Strength: The estimated ultimate geotechnical unit grout-to-ground bond strength selected for use in design. Same as $\alpha_{\text{bond nominal strength}}$ (SLD and LFD)

Overburden: Material, natural or placed, that may require cased drilling methods to provide an open borehole to underlying strata.

Post-grouting: The injection of additional grout into the load transfer length of a micropile after the primary grout has set. Also known as regrouting or secondary grouting.

Primary Grout: Portland-cement-based grout injected into the micropile hole prior to or after the installation of the reinforcement to direct the load transfer to the surrounding ground along the micropile.

Proof Load Test: Incremental loading of a production micropile, recording the total movement at each increment.

Reinforcement: The steel component of the micropile that accepts and/or resists applied loadings.

Sheathing: Smooth or corrugated piping or tubing that protects the reinforcing steel against corrosion.

Spacer: A device to separate elements of a multiple-element reinforcement.

Verification Load Test: Pile load test performed to verify the design of the pile system and the construction methods proposed, prior to installation of production piles.

1.5 Referenced Codes and Standards.

The following publications form a part of this specification to the extent indicated by the references. The latest publication as of the issue date of this specification shall govern, unless indicated otherwise.

1.5.1 American Society for Testing and Materials (ASTM)

American Association of State Highway and Transportation Officials (AASHTO)

ASTM	AASHTO	SPECIFICATION / TEST
A36, A572	M183, M223	Structural Steel
A82	M55	Cold-Drawn Steel Wire for Concrete Reinforcement
A252	–	Welded and Seamless Steel Pipe Piles
A615	M31	Deformed and Plain Billet Steel Bars for Concrete Reinforcement
A722	M275	Uncoated High-Strength Steel Bar for Prestressing Concrete
A775	–	Epoxy -Coated Reinforcing Steel Bars
A934	–	Epoxy-Coated Prefabricated Steel Reinforcing Bars
C 33	M80	Concrete Aggregates
C 109	T106	Compressive Strength of Hydraulic Cement Mortar
C 188	T133	Density of Hydraulic Cement
C 144	M45	Aggregate for Masonry Mortar
C 150	M 85	Portland Cement
C 494	M194	Chemical Admixtures for Concrete
D 1143	–	Method of Testing Piles Under Static Axial Compressive Load
D 1784	–	Polyvinyl Chloride (PVC) Pipe (Class 13464-B)
D 3350	M 252	Polyethylene Corrugated Tubing
D 3689	–	Method of Testing Individual Piles Under Static Axial Tensile Load
D 3966		Standard Test Method for Piles Under Lateral Load
–	T 26	Quality of Water to be Used in Concrete

1.5.2 American Welding Society (AWS)

D1.1 Structural Welding Code-Steel

D1.2 Structural Welding Code-Reinforcing Steel

1.5.3 American Petroleum Institute (API)

5CT (N-80)	Specification for casing and tubing
RP 13B-1	Recommended Practice – Standard Procedure for Field Testing Water Based Drilling Fluids

1.6 Available Information.

Available information developed by the Owner, or by the Owner's duly authorized representative include the following items:

1. Plans prepared by ___________ , dated __________. The plans include the plan view, profile and typical cross sections for the proposed micropile locations. ***(Commentary:*** *Refer to chapter 9 of the FHWA "Micropile Design and Construction Guidelines Manual", Report No. FHWA- SA-97-070 for detailed guidance on plan information to provide on the Owner-Controlled Design preliminary plans. An example preliminary plan for the bridge foundation support design example no. 1 is included at the end of this guide specification.)*

2. Geotechnical Report No.(s) ________ titled __________, dated__________ , included or referenced in the bid documents, contains the results of test pits, exploratory borings and other site investigation data obtained in the vicinity of the proposed micropile locations.

(Commentary: *The subsurface conditions expected can significantly impact the contractor's choice of procedures, methods, or equipment, the bidding process, and contract administration. Experience has proven that use of a geotechnical summary is advantageous* *toward achieving a successful contract. It is recommended that advisory wording be inserted into the contract special provisions to "red flag" the conditions to bidders. Preparation and use of a "Summary of Geotechnical Conditions", such as used by the Washington State DOT, is recommended. WSDOT's in-house guide for preparation of said summary, along with an example writeup for the Sample Problem No.1 contained in this manual, are included in Appendix B. WSDOT inserts the Summary into the contract special provisions, making it a legal part of the contract documents. The purpose is to alert and be fair to bidders, and thus prevent/minimize differing site condition construction claims and dispute.)*

1.7 Construction Site Survey

Before bidding the Work, the Contractor shall review the available subsurface information and visit the site to assess the site geometry, equipment access conditions, and location of existing structures and above ground facilities.

The Contractor is responsible for field locating and verifying the location of all utilities shown on the plans prior to starting the Work. Maintain uninterrupted service for those utilities designated to remain in service throughout the Work. Notify the Engineer of any utility locations different from shown on the plans that may require micropile relocations or structure design modification. Subject to the Engineer's approval, additional cost to the Contractor due to micropile relocations and/or structure design modification resulting from utility locations different from shown on the plans, will be paid as Extra Work.

***(Commentary:** The location of both active and abandoned buried utilities within the ground mass to receive micropiles can have a profound impact on the design and construction of the micropiles. Careful consideration of the presence and location of all utilities is required for successful design and installation of micropiles.)*

Prior to start of any micropile construction activity, the Contractor and Engineer shall jointly inspect the site to observe and document the pre-construction condition of the site, existing structures and facilities.

1.8 Micropile Design Requirements.

The micropiles shall be designed to meet the specified loading conditions, as shown on the contract plans and approved working drawings. Design the micropiles and pile top to footing connections using the Service Load Design (SLD) procedures contained in the FHWA "Micropile Design and Construction Guidelines Manual", Report No. FHWA- SA-97-070. *(**Commentary:** The FHWA micropile manual- Chapter 5- also presents Load Factor Design (LFD) procedures for micropile foundations. Revise specification if LFD design is required.)*

The required geotechnical safety factors/strength factors (for SLD Design) or load and resistance factors (for LFD Design) shall be in accord with the FHWA manual, unless specified otherwise. Estimated soil/rock design shear strength parameters, unit weights, applied foundation loadings, slope and external surcharge loads, corrosion protection requirements, known utility locations, easements, right-of-ways and other applicable design criteria will be as shown on the plans or specified herein. Structural design of any individual micropile structure elements not covered in the FHWA manual shall be by the service load design method in conformance with appropriate articles of the most current Edition of the AASHTO Standard Specifications for Highway Bridges, including current interim specifications.

Steel pipe used for micropile permanent casing shall incorporate an additional _____ mm thickness of sacrificial steel for corrosion protection. *(**Commentary**: This paragraph is optional and to be selected by the designer or specified by the owner. AASHTO Section 4.5.7.4 Cross-Section Adjustment for Corrosion, states - "For concrete-filled pipe piles where corrosion may be expected, 1/16 inch (1.6mm) shall be deducted from the shell thickness to allow for reduction in section due to corrosion.")*

Where required as shown on the contract plans, corrosion protection of the internal steel reinforcing bars, consisting of either encapsulation, epoxy coating, or grout, shall be provided in accordance with Materials Section 2.0. Where permanent casing is used for a portion of the micropile, encapsulation shall extend at least 1.5 m into the casing.

*(**Commentary:** When installation of micropiles provides additional support for existing structures, such as for seismic retrofit or underpinning applications, the structural designer should add appropriate specification verbage and design criteria into this specification to cover the penetration of the existing structural elements and the top of pile anchorage to the existing structure.)*

1.8.1 Micropile Design Submittals.

At least 21 calendar days before the planned start of micropile structure construction, submit complete design calculations and working drawings to the Engineer for review and approval. Include all details, dimensions, quantities, ground profiles, and cross-sections necessary to construct the micropile structure. Verify the limits of the micropile structure and ground survey data before preparing the detailed working drawings.

The drawings and calculations shall be signed and sealed by the contractor's Professional Engineer or by the Consultant designer's Professional Engineer (if applicable), previously approved by the owner's Engineer. If the micropile contractor uses a consultant designer to prepare the design, the micropile contractor shall still have overall contract responsibility for both the design and the construction.

1.8.2 Design Calculations.

Design calculations shall include, but not be limited to, the following items:

1. A written summary report which describes the overall micropile design.
2. Applicable code requirements and design references.
3. Micropile structure critical design cross-section(s) geometry including soil/rock strata and piezometric levels and location, magnitude and direction of design applied loadings, including slope or external surcharge loads.

4. Design criteria including, soil/rock shear strengths (friction angle and cohesion), unit weights, and ground-grout bond values and micropile drillhole diameter assumptions for each soil/rock strata.

5. Safety factors/strength factors (for Service Load Design) or load and resistance factors (for Load Factor Design) used in the design on the ground-grout bond values, surcharges, soil/rock and material unit weights, steel, grout, and concrete materials.

6. Seismic design earthquake acceleration coefficient.

7. Design calculation sheets (both static and seismic) with the project number, micropile structure location, designation, date of preparation, initials of designer and checker, and page number at the top of each page. Provide an index page with the design calculations.

8. Design notes including an explanation of any symbols and computer programs used in the design.

9. Pile to footing connection calculations.

10. Calculations for footing reinforcement different from that on the Owner provided preliminary plans, if required to accommodate the Contractor designed pile layout.

1.8.3 Working Drawings.

The working drawings shall include all information required for the construction and quality control of the piling. Working drawings shall include, but not be limited to, the following items unless provided in the contract plans:

1. A plan view of the micropile structure(s) identifying:

   (a) A reference baseline and elevation datum.

   (b) The offset from the construction centerline or baseline to the face of the micropile structure at all changes in horizontal alignment.

(c) Beginning and end of micropile structure stations.

(d) Right-of-way and permanent or temporary construction easement limits, location of all known active and abandoned existing utilities, adjacent structures or other potential interferences. The centerline of any drainage structure or drainage pipe behind, passing through, or passing under the micropile structure.

(e) Subsurface exploration locations shown on a plan view of the proposed micropile structure alignment with appropriate reference base lines to fix the locations of the explorations relative to the micropile structure.

2. An elevation view of the micropile structure(s) identifying:

   (a) Elevation view showing micropile locations and elevations; vertical and horizontal spacing; batter and alignment and the location of drainage elements (if applicable).

   (b) Existing and finish grade profiles both behind and in front of the micropile structure.

3. Design parameters and applicable codes.

4. General notes for constructing the micropile structure including construction sequencing or other special construction requirements.

5. Horizontal and vertical curve data affecting the micropile structure and micropile structure control points. Match lines or other details to relate micropile structure stationing to centerline stationing.

6. A listing of the summary of quantities on the elevation drawing of each micropile structure showing pay item estimated quantities.

7. Micropile typical sections including micropile spacing and inclination; minimum drillhole diameter; pipe casing and reinforcing bar sizes and details; splice types and locations; centralizers and spacers; grout bond zone and casing plunge lengths (if used);

corrosion protection details; and connection details to the substructure footing, anchorage, plates, etc.

8. A typical detail of verification and production proof test micropiles defining the micropile length, minimum drillhole diameter, inclination, and load test bonded and unbonded test lengths.

9. Details, dimensions, and schedules for all micropiles, casing and reinforcing steel, including reinforcing bar bending details.

10. Details for constructing micropile structures around drainage facilities (if applicable).

The working drawings and design calculations shall be signed and sealed by the Contractor's Professional Engineer or by the Consultant designer's Professional Engineer (if applicable), previously pre-qualified by the Owner. If the micropile Contractor uses a Consultant designer to prepare the design, the micropile Contractor shall still have overall contract responsibility for both the design and the construction.

Submit 5 sets of the working drawings with the initial submission. Drawing sheet size shall be 550 by 850 mm. One set will be returned with any indicated corrections. The Engineer will approve or reject the Contractor's submittal within 15 calendar days after receipt of a complete submission. If revisions are necessary, make the necessary corrections and resubmit 5 revised sets. When the drawings are approved, furnish 5 sets and a Mylar sepia set of the approved drawings. The Contractor will not be allowed to begin micropile structure construction or incorporate materials into the work until the submittal requirements are satisfied and found acceptable to the Engineer. Changes or deviations from the approved submittals must be re-submitted for approval. No adjustments in contract time or delay or impact claims will be allowed due to incomplete submittals. *(**Commentary:** Submittals procedures shall be coordinated with Owner/Agency procedures).*

Revise the drawings when plan dimensions are changed due to field conditions or for other reasons. Within 30 days after completion of the work, submit as-built drawings to the Engineer. Provide revised design calculations signed by the approved Registered Professional Engineer for all design changes made during the construction of the micropile structure.

1.9 Construction Submittals.

The Contractor shall prepare and submit to the Engineer, for review of completeness, 5 copies of the following for the micropile system or systems to be constructed:

1. Detailed step-by-step description of the proposed micropile construction procedure, including personnel, testing and equipment to assure quality control. This step-by-step procedure shall be shown on the working drawings in sufficient detail to allow the Engineer to monitor the construction and quality of the micropiles.

2. Proposed start date and time schedule and micropile installation schedule providing the following:

   Micropile number
   Micropile design load
   Type and size of reinforcing steel
   Minimum total bond length
   Total micropile length
   Micropile top footing attachment

3. If welding of casing is proposed, submit the proposed welding procedure, certified by a qualified welding specialist.

4. Information on headroom and space requirements for installation equipment that verify the proposed equipment can perform at the site.

5. Plan describing how surface water, drill flush, and excess waste grout will be controlled and disposed.

6. Certified mill test reports for the reinforcing steel or coupon test results for permanent casing without mill certification. The ultimate strength, yield strength, elongation, and material properties composition shall be included. For API N-80 pipe casing, coupon test results may be submitted in lieu of mill certification.

7. Proposed Grouting Plan. The grouting plan shall include complete descriptions, details, and supporting calculations for the following:

   (a) Grout mix design and type of materials to be used in the grout including certified test data and trial batch reports.

   (b) Methods and equipment for accurately monitoring and recording the grout depth, grout volume and grout pressure as the grout is being placed.

   (c) Grouting rate calculations, when requested by the Engineer. The calculations shall be based on the initial pump pressures or static head on the grout and losses throughout the placing system, including anticipated head of drilling fluid (if applicable) to be displaced.

   (d) Estimated curing time for grout to achieve specified strength. Previous test results for the proposed grout mix completed within one year of the start of grouting may be submitted for initial verification and acceptance and start of production work. During production, grout shall be tested in accord with Section 3.4.5.

   (e) Procedure and equipment for Contractor monitoring of grout quality.

8. Detailed plans for the proposed micropile load testing method. This shall include all drawings, details, and structural design calculations necessary to clearly describe the proposed test method, reaction load system capacity and equipment setup, types and accuracy of apparatus to be used for applying and measuring the test loads and pile top movements in accordance with Section 3.6, Pile Load Tests.

9. Calibration reports and data for each test jack, pressure gauge and master pressure gauge and electronic load cell to be used. The calibration tests shall have been performed by an independent testing laboratory, and tests shall have been performed within 90 calendar days of the date submitted. Testing shall not commence until the Engineer has reviewed and accepted the jack, pressure gauge, master pressure gauge and electronic load cell calibration data.

Work other than test pile installation shall not begin until the construction submittals have been received, reviewed, and accepted in writing by the Engineer. Provide submittal items 1 through 5 at least 21 calendar days prior to initiating micropile construction, item 7 as the work progresses for each delivery and submittal items 6, 8 and 9 at least 7 days prior to start of micropile load testing or incorporation of the respective materials into the work. The Contractor shall allow the Engineer 7 calendar days to review the construction submittals after a complete set has been received. Additional time required due to incomplete or unacceptable submittals shall not be cause for delay or impact claims. All costs associated with incomplete or unacceptable Contractor submittals shall be the responsibility of the Contractor.

1.10 Pre-construction Meeting.

A pre-construction meeting will be scheduled by the Engineer and held prior to the start of micropile construction. The Engineer, prime Contractor, micropile specialty Contractor, micropile designer, excavation Contractor and geotechnical instrumentation specialist (if applicable) shall attend the meeting. Attendance is mandatory. The pre-construction meeting will be conducted to clarify the construction requirements for the work, to coordinate the construction schedule and activities, and to identify contractual relationships and delineation of responsibilities amongst the prime Contractor and the various Subcontractors - specifically those pertaining to excavation for micropile structures, anticipated subsurface conditions, micropile installation and testing, micropile structure survey control and site drainage control.

2.0 MATERIALS.

Furnish materials new and without defects. Remove defective materials from the jobsite at no additional cost. Materials for micropiles shall consist of the following:

Admixtures for Grout: Admixtures shall conform to the requirements of ASTM C 494/AASHTO M194. Admixtures that control bleed, improve flowability, reduce water content, and retard set may be used in the grout, subject to the review and acceptance of the Engineer. Admixtures shall be compatible with the grout and mixed in accordance with the manufacturer's recommendations. Expansive admixtures shall only be added to the grout used for filling sealed encapsulations and anchorage covers. Accelerators are not permitted. Admixtures containing chlorides are not permitted.

Cement: All cement shall be Portland cement conforming to ASTM C 150/AASHTO M85, Types II, III or V.

Centralizers and Spacers: Centralizers and spacers shall be fabricated from schedule 40 PVC pipe or tube, steel, or material non-detrimental to the reinforcing steel. Wood shall not be used. Centralizers and spacers shall be securely attached to the reinforcement; sized to position the reinforcement within 10 mm of plan location from center of pile; sized to allow grout tremie pipe insertion to the bottom of the drillhole; and sized to allow grout to freely flow up the drillhole and casing and between adjacent reinforcing bars.

Encapsulation: Encapsulation (double corrosion protection) shall be shop fabricated using high-density, corrugated polyethylene tubing conforming to the requirements of ASTM D3350/AASHTO M252 with a nominal wall thickness of 0.8 mm. The inside annulus between the reinforcing bars and the encapsulating tube shall be a minimum of 5mm and be fully grouted with non-shrink grout conforming to Materials Section 2.0.

Epoxy Coating: The minimum thickness of coating applied electrostatically to the reinforcing steel shall be 0.3 mm. Epoxy coating shall be in accordance with ASTM A775 or ASTM

A934. Bend test requirements are waived. Bearing plates and nuts encased in the pile concrete footing need not be epoxy coated.

Fine Aggregate: If sand - cement grout is used, sand shall conform to ASTM C 144/AASHTO M45.

Grout: Neat cement or sand/cement mixture with a minimum 3-day compressive strength of 14 MPa and a 28-day compressive strength of 28 MPa per AASHTO T106/ASTM C109. *(**Commentary:** Note to designers/specifiers. A 28-day grout strength of 28 MPa is common for micropiles. If the micropile design calls for a higher grout strength, revise the specification accordingly.)*

Grout Protection: Provide a minimum 25 mm grout cover over bare or epoxy coated bars (excluding bar couplers) or minimum 12 mm grout cover over the encapsulation of encapsulated bars.

Permanent Casing Pipe: Permanent steel casing/pipe shall have the diameter and at least minimum wall thickness shown on the approved Working Drawings. The permanent steel casing/pipe:

1. shall meet the Tensile Requirements of ASTM A252, Grade 3, except the yield strength shall be a minimum of 345 MPa to 552 MPa as used in the design submittal.
2. may be new "Structural Grade" (a.k.a. "Mill Secondary") steel pipe meeting above but without Mill Certification, free from defects (dents, cracks, tears) and with two coupon tests per truckload delivered to the fabricator.

For permanent casing/pipe that will be welded, the following material conditions apply:

1. the carbon equivalency (CE) as defined in AWS D1.l, Section X15.1, shall not exceed 0.45, as demonstrated by mill certifications
2. the sulfur content shall not exceed 0.05%, as demonstrated by mill certifications

For permanent casing/pipe that will be shop or field welded, the following fabrication or construction conditions apply:

1. the steel pipe shall not be joined by welded lap splicing
2. welded seams and splices shall be complete penetration welds
3. partial penetration welds may be restored in conformance with AWS D1.1
4. the proposed welding procedure certified by a welding specialist shall be submitted for approval

Threaded casing joints shall develop at least the required nominal resistance used in the design of the micropile.

*(**Commentary:** From a practical standpoint, the adequacy of pipe and reinforcing bar splices and threaded joint connections will be verified by the verification and proof load testing).*

Plates and Shapes: Structural steel plates and shapes for pile top attachments shall conform to ASTM A 36/AASHTO M183, or ASTM A 572/AASHTO M223, Grade 350.

Reinforcing Bars: Reinforcing steel shall be deformed bars in accordance with ASTM A 615/AASHTO M31, Grade 420 or Grade 520 or ASTM A 722/AASHTO M275, Grade 1035. When a bearing plate and nut are required to be threaded onto the top end of reinforcing bars for the pile top to footing anchorage, the threading may be continuous spiral deformed ribbing provided by the bar deformations (e.g., Dywidag or Williams continuous threadbars) or may be cut into a reinforcing bar. If threads are cut into a reinforcing bar, the next larger bar number designation from that shown on the Plans shall be provided, at no additional cost.

Bar tendon couplers, if required, shall develop the ultimate tensile strength of the bars without evidence of any failure.

Reinforcing Bar Corrosion Protection:

*(**Commentary:** Corrosion protection requirements vary between Transportation Agencies. The most common and simplest tests utilized to measure the aggressiveness of the soil environment include electrical resistivity, pH, chloride, and sulfate. Per FHWA-RD-89-198, the ground is considered aggressive if any one of these indicators show critical values as detailed below:*

PROPERTY	TEST DESIGNATION*	CRITICAL VALUES*
Resistivity	*AASHTO T-288, ASTM G 57*	*below 2,000 ohm-cm*
pH	*AASHTO T-289, ASTM G 51*	*below 5*
Sulfate	*AASHTO T-290, ASTM D516M, ASTM D4327*	*above 200 ppm*
Chloride	*AASHTO T-291, ASTM D512, ASTM D4327*	*above 100 ppm*

** Specifier should check test standards for latest updates and individual transportation agencies may have limits on critical values different than tabulated above. Standard specifications or test methods for any of the above items which are common to your agency can be referenced in lieu of the above listed AASHTO/ASTM references.*

Sheathing: Smooth plastic sheathing, including joints, shall be watertight. Polyvinyl chloride (PVC) sheathing shall conform to ASTM D 1784, Class 13464-B.

Water: Water used in the grout mix shall conform to AASHTO T 26 and shall be potable, clean, and free from substances that may be injurious to cement and steel.

3.0 CONSTRUCTION REQUIREMENTS

3.1 Site Drainage Control.

The Contractor shall control and properly dispose of drill flush and construction related waste, including excess grout, in accord with the standard specifications and all applicable local codes and regulations. Provide positive control and discharge of all surface water that will affect construction of the micropile installation. Maintain all pipes or conduits used to control surface water during construction. Repair damage caused by surface water at no additional cost. Upon substantial completion of the Work, remove surface water control pipes or conduits from the site. Alternatively, with the approval of the Engineer, pipes or conduits that are left in place, may be fully grouted and abandoned or left in a way that protects the structure and all adjacent facilities from migration of fines through the pipe or conduit and potential ground loss.

Immediately contact the Engineer if unanticipated existing subsurface drainage structures are discovered during excavation or drilling. Suspend work in these areas until remedial measures meeting the Engineer's approval are implemented. Cost of remedial measures or repair work resulting from encountering unanticipated subsurface drainage structures, will be paid for as Extra Work.

3.2 Excavation

Coordinate the work and the excavation so the micropile structures are safely constructed. Perform the micropile construction and related excavation in accordance with the Plans and approved submittals. No excavations steeper than those specified herein or shown on the Plans will be made above or below the micropile structure locations without written approval of the Engineer.

3.3 Micropile Allowable Construction Tolerances

1. Centerline of piling shall not be more than 75 mm from indicated plan location.
2. Pile shall be plumb within 2 percent of total-length plan alignment.
3. Top elevation of pile shall be plus 25 mm or minus 50 mm maximum from vertical elevation indicated.
4. Centerline of reinforcing steel shall not be more than 15 mm from indicated location.

3.4 Micropile Installation

The micropile Contractor shall select the drilling method, the grouting procedure, and the grouting pressure used for the installation of the micropiles. The micropile Contractor shall also determine the micropile casing size, final drillhole diameter and bond length, and central tendon reinforcement steel sizing necessary to develop the specified load capacities and load testing requirements. The micropile Contractor is also responsible for estimating the grout take. There will be no extra payment for grout overruns. *(**Commentary:** Note, extra payment for grout takes is appropriate for micropiles in Karst. Otherwise, the bid price of these piles will be artificially high to cover risk of high grout loss.)*

3.4.1 Drilling

The drilling equipment and methods shall be suitable for drilling through the conditions to be encountered, without causing damage to any overlying or adjacent structures or services. The drillhole must be open along it's full length to at least the design minimum drillhole diameter prior to placing grout and reinforcement. *(**Commentary**: When micropile construction will occur in close proximity to settlement sensitive structures, recommend including the following sentence in the specification - Vibratory pile driving hammers shall not be used to advance casing.)*

Temporary casing or other approved method of pile drillhole support will be required in caving or unstable ground to permit the pile shaft to be formed to the minimum design drillhole diameter. The Contractor's proposed method(s) to provide drillhole support and to prevent detrimental ground movements shall be reviewed by the Engineer. Detrimental ground movement is defined as movement which requires remedial repair measures. Use of drilling fluid containing bentonite is not allowed. *(**Commentary**: The specification verbage related to drillhole support methods and difficulty of drilling may vary project to project depending on the subsurface conditions revealed by the subsurface investigation data. It is the micropile specialty contractor's responsibility to select the proper drilling equipment and methods for the site conditions. It is the owner's responsibility to provide the available subsurface information. For projects with difficult ground conditions, use of an "advisory specification" included in the contract documents is recommended. Refer to Appendix B for an example.)*

Costs of removal or remedial measures due to encountering unanticipated subsurface obstructions will be paid for as Extra Work.

3.4.2 Ground Heave or Subsidence.

During construction, the Contractor shall observe the conditions vicinity of the micropile construction site on a daily basis for signs of ground heave or subsidence. Immediately notify the Engineer if signs of movements are observed. Contractor shall immediately suspend or modify drilling or grouting operations if ground heave or subsidence is observed, if the micropile structure is adversely affected, or if adjacent structures are damaged from the drilling or grouting. If the Engineer determines that the movements require corrective action, the Contractor shall take corrective actions necessary to stop the movement or perform repairs. When due to the Contractor's methods or operations or failure to follow the specified/approved construction sequence, as determined by the Engineer, the costs of providing corrective actions will be borne by the Contractor. When due to differing site conditions, as determined by the Engineer, the costs of providing corrective actions will be paid as Extra Work.

3.4.3 Pipe Casing and Reinforcing Bars Placement and Splicing.

Reinforcement may be placed either prior to grouting or placed into the grout - filled drillhole before temporary casing (if used) is withdrawn. Reinforcement surface shall be free of deleterious substances such as soil, mud, grease or oil that might contaminate the grout or coat the reinforcement and impair bond. Pile cages and reinforcement groups, if used, shall be sufficiently robust to withstand the installation and grouting process and the withdrawal of the drill casings without damage or disturbance.

The Contractor shall check pile top elevations and adjust all installed micropiles to the planned elevations.

Centralizers and spacers (if used) shall be provided at 3-m centers maximum spacing. The upper and lower most centralizer shall be located a maximum of 1.5 m from the top and bottom of the micropile. Centralizers and spacers shall permit the free flow of grout without misalignment of the reinforcing bar(s) and permanent casing. The central reinforcement bars with centralizers shall be lowered into the stabilized drill hole and set. The reinforcing steel shall be inserted into the drill hole to the desired depth without difficulty. Partially inserted reinforcing bars shall not be driven or forced into the hole. Contractor shall redrill and reinsert reinforcing steel when necessary to facilitate insertion.

Lengths of casing and reinforcing bars to be spliced shall be secured in proper alignment and in a manner to avoid eccentricity or angle between the axes of the two lengths to be spliced. Splices and threaded joints shall meet the requirements of Materials Section 2.0. Threaded pipe casing joints shall be located at least two casing diameters (OD) from a splice in any reinforcing bar. When multiple bars are used, bar splices shall be staggered at least 0.3 meters.

3.4.4 Grouting.

Micropiles shall be primary grouted the same day the load transfer bond length is drilled. The Contractor shall use a stable neat cement grout or a sand cement grout with a minimum 28-day unconfined compressive strength of 28 MPa. Admixtures, if used, shall be mixed in accordance with manufacturer's recommendations. The grouting equipment used shall produce a grout free of lumps and undispersed cement. The Contractor shall have means and methods of measuring the grout quantity and pumping pressure during the grouting operations. The grout pump shall be equipped with a pressure gauge to monitor grout pressures. A second pressure gauge shall be placed at the point of injection into the pile top. The pressure gauges shall be capable of measuring pressures of at least 1 MPa or twice the actual grout pressures used, whichever is greater. The grout shall be kept in agitation prior to mixing. Grout shall be placed within one hour of mixing. The grouting equipment shall be sized to enable each pile to be grouted in one continuous operation. The grout shall be injected from the lowest point of the drill hole and injection shall continue until uncontaminated grout flows from the top of the pile. The grout may be pumped through grout tubes, casing, hollow-stem augers, or drill rods. Temporary casing, if used, shall be extracted in stages ensuring that, after each length of casing is removed the grout level is brought back up to the ground level before the next length is removed. The tremie pipe or casing shall always extend below the level of the existing grout in the drillhole. The grout pressures and grout takes shall be controlled to prevent excessive heave or fracturing of rock or soil formations. Upon completion of grouting, the grout tube may remain in the hole, but must be filled with grout.

If the Contractor elects to use a postgrouting system, Working Drawings and details shall be submitted to the Engineer for review in accordance with Section 1.8, Pre-installation Submittals.

3.4.5 Grout Testing

Grout within the micropile verification and proof test piles shall attain the minimum required 3-day compressive strength of 14 MPa prior to load testing. Previous test results for the proposed grout mix completed within one year of the start of work may be submitted for initial verification of the required compressive strengths for installation of pre-production verification test piles and initial production piles. During production, micropile grout shall be tested by the Contractor for compressive strength in accordance with AASHTO T106/ASTM C109 at a frequency of no less than one set of three 50-mm grout cubes from each grout plant each day of operation or per every 10 piles, whichever occurs more frequently. The compressive strength shall be the average of the 3 cubes tested.

Grout consistency as measured by grout density shall be determined by the Contractor per ASTM C 188/AASHTO T 133 or API RP-13B-1 at a frequency of at least one test per pile, conducted just prior to start of pile grouting. The Baroid Mud Balance used in accordance with API RP-13B-1 is an approved device for determining the grout density of neat cement grout. The measured grout density shall be between ________ kg/m^3 and ________ kg/m^3.

Grout samples shall be taken directly from the grout plant. Provide grout cube compressive strength and grout density test results to the Engineer within 24 hours of testing. *(**Commentary**: If the Engineer will perform the grout testing, revise this section accordingly).*

3.5 Micropile Installation Records.

Contractor shall prepare and submit to the Engineer full-length installation records for each micropile installed. The records shall be submitted within one work shift after that pile installation is completed. The data shall be recorded on the micropile installation log included at the end of this specification. A separate log shall be provided for each micropile. *(**Commentary**: In addition to the expertise of the micropile specialty Contractor, the quality of the individual construction elements is directly related to the final product overall quality. As with other drilled pile systems, the actual load carrying capacity of a micropile can only be definitively proven by pile load tests. It is not practical or economical to test every pile* *installed. Therefore, aggressive inspection by the Contractor and Owner's Engineer is needed to assure that each individual micropile is well constructed and to justify load testing only a small number, e.g, 5%, of the total number of production piles installed.)*

3.6 Pile Load Tests

Perform verification and proof testing of piles at the locations specified herein or designated by the Engineer. Perform compression load testing in accord with ASTM D1143 and tension load testing in accord with ASTM D3689, except as modified herein.

*(**Commentary on number of load tests:** Specifier/designer need to determine and write into this portion of the specification the number and location of required verification and proof tests. The total number of load tests and maximum test loads to be specified can vary on a project-by-project basis. They are dependent on ground type and variability , required pile capacity, pile loading type (i.e., static or seismic), total number of piles, criticality of the structure and available site access and work space. Guideline criteria for estimating the total number of verification and proof test piles are given in Chapter 7. For structure foundations, the following is recommended as a minimum. Perform verification testing of at least one sacrificial test pile per structure, prior to installation of any production piles. New users should perform proof tests on production piles at a frequency of 5 percent (1 in 20). For experienced users, number of tests are to be determined by Owner/Engineer on a project by project basis. If pile capacity demands are greatest in compression, the piles should be load tested in compression. If the pile capacity demands are equal for both compression and tension, or greater in tension, it is recommended that tension testing alone be conducted, to reduce costs).*

*(**Commentary:** Specifier - Indicate here whether compression or tension testing, or both are required for your project).*

3.6.1 Verification Load Tests

Perform pre-production verification pile load testing to verify the design of the pile system and the construction methods proposed prior to installing any production piles. __________ sacrificial verification test piles shall be constructed in conformance with the approved Working Drawings. Verification test pile(s) shall be installed at the following locations ___________.

Verification load tests shall be performed to verify that the Contractor installed micropiles will meet the required compression and tension load capacities and load test acceptance criteria and to verify that the length of the micropile load transfer bond zone is adequate. The micropile verification load test results must verify the Contractor's design and installation methods, and be reviewed and accepted by the Engineer prior to beginning installation of production micropiles.

The drilling-and-grouting method, casing length and outside diameter, reinforcing bar lengths, and depth of embedment for the verification test pile(s) shall be identical to those specified for the production piles at the given locations. The verification test micropile structural steel sections shall be sized to safely resist the maximum test load. ***(Commentary:** Note that if additional steel area is provided in the verification test, the measured deflection will be lower than production piles.)*

The maximum verification and proof test loads applied to the micropile shall not exceed 80 percent of the structural capacity of the micropile structural elements, to include steel yield in tension, steel yield or buckling in compression, or grout crushing in compression. Any required increase in strength of the verification test pile elements above the strength required for the production piles shall be provided for in the contractor’s bid price.

The jack shall be positioned at the beginning of the test such that unloading and repositioning during the test will not be required. When both compression and tension load testing is to be performed on the same pile, the pile shall be tested under compression loads prior to testing under tension loads.

3.6.2 Testing Equipment and Data Recording.

Testing equipment shall include dial gauges, dial gauge support, jack and pressure gauge, electronic load cell, and a reaction frame. The load cell is required only for the creep test portion of the verification test. ***(Commentary:*** *The purpose and value of an electronic load cell is to measure small changes in load for load tests where the load is held for a long duration, such as during verification or creep testing. It is not intended to be used during proof testing, including the short term creep portion. Experience has proven that load cells have been problematic under field conditions, yet even with errors resulting from cell construction, off-center loading, and other effects, a load cell is very sensitive to small changes in load and is strongly recommended for creep testing.)* The contractor shall provide a description of test setup and jack , pressure gauge and load cell calibration curves in accordance with the Submittals Section.

Design the testing reaction frame to be sufficiently rigid and of adequate dimensions such that excessive deformation of the testing equipment does not occur. Align the jack, bearing plates, and stressing anchorage such that unloading and repositioning of the equipment will not be required during the test.

Apply and measure the test load with a hydraulic jack and pressure gauge. The pressure guage shall be graduated in 500 kPa increments or less. The jack and pressure gauge shall have a pressure range not exceeding twice the anticipated maximum test pressure. Jack ram travel shall be sufficient to allow the test to be done without resetting the equipment. Monitor the creep test load hold during verification tests with both the pressure gauge and the electronic load cell. Use the load cell to accurately maintain a constant load hold during the creep test load hold increment of the verification test.

Measure the pile top movement with a dial gauge capable of measuring to 0.025 mm. The dial gauge shall have a travel sufficient to allow the test to be done without having to reset the gauge. Visually align the gauge to be parallel with the axis of the micropile and support the gauge independently from the jack, pile or reaction frame. Use a minimum of two dial gauges when the test setup requires reaction against the ground or single reaction piles on each side of the test pile. ***(Commentary:*** *Experience with testing piles reacting against the ground, or against single reaction piles on each side of the test pile, has resulted in racking and misalignment of the system on some projects. Two dial guages are recommended for this test setup to determine if racking is occurring and to provide a more accurate average micropile head movement measurement).*

The required load test data shall be recorded by the Engineer.

3.6.3 Verification Test Loading Schedule.

Test verification piles designated for compression or tension load testing to a maximum test load of 2.5 times the micropile Design Load shown on the Plans or Working Drawings. ***(Commentary:*** *See Section 5.E.4 for more detailed verification load testing information.)* The verification pile load tests shall be made by incrementally loading the micropile in accordance with the following cyclic load schedule for both compression and tension loading:

AL = Alignment Load	DL = Design Load	
	LOAD	HOLD TIME
1	AL (0.05 DL)	1 minute
2	0.25 DL	1 minute
3	0.50 DL	1 minute
4	AL	1 minute
5	0.25 DL	1 minute
6	0.50 DL	1 minute
7	0.75 DL	1 minute
8	AL	1 minute
9	0.25 DL	1 minute
10	0.50 DL	1 minute
11	0.75 DL	1 minute
12	1.00 DL	1 minute

AL = Alignment Load	DL = Design Load	
	LOAD	HOLD TIME
13	AL	1 minute
14	0.25 DL	1 minute
15	0.50 DL	1 minute
16	0.75 DL	1 minute
17	1.00 DL	1 minute
18	1.33 DL	**60 minutes (Creep Test Load Hold)**
19	1.75 DL	1 minute
20	2.00 DL	1 minute
21	2.25 DL	1 minute
22	**2.50 DL (Maximum Test Load)**	10 minutes
23	AL	1 minute

The test load shall be applied in increments of 25 percent of the DL load. Each load increment shall be held for a minimum of 1 minute. Pile top movement shall be measured at each load increment. The load-hold period shall start as soon as each test load increment is applied. The verification test pile shall be monitored for creep at the 1.33 Design Load (DL). Pile movement during the creep test shall be measured and recorded at 1, 2, 3, 4, 5, 6, 10, 20, 30, 50, and 60 minutes. The alignment load shall not exceed 5 percent of the DL load. Dial gauges shall be reset to zero after the initial AL is applied.

The acceptance criteria for micropile verification load tests are:

1. The pile shall sustain the first compression or tension 1.0 DL test load with no more than _______mm total vertical movement at the top of the pile, relative to the position of the top of the pile prior to testing. *(**Commentary:** Structural designer to determine maximum allowable total pile top structural axial displacement at 1.0 DL test load based on structural design requirements. Also, if the verification test pile has to be upsized structurally to accommodate the maximum required verification test load, this*

provision will not apply. Only the proof tested production piles will then be subject to this criteria. Refer to Chapter 5 for more design guidance).

2. At the end of the 1.33 DL creep test load increment, test piles shall have a creep rate not exceeding 1 mm/log cycle time (1 to 10 minutes) or 2 mm/log cycle time (6 to 60 minutes or the last log cycle if held longer).The creep rate shall be linear or decreasing throughout the creep load hold period.

3. Failure does not occur at the 2.5 DL maximum test load. Failure is defined as load at which attempts to further increase the test load simply result in continued pile movement.

The Engineer will provide the Contractor written confirmation of the micropile design and construction within 3 working days of the completion of the verification load tests. This written confirmation will either confirm the capacities and bond lengths specified in the Working Drawings for micropiles or reject the piles based upon the verification test results.

3.6.4 Verification Test Pile Rejection

If a verification tested micropile fails to meet the acceptance criteria, the Contractor shall modify the design, the construction procedure, or both.These modifications may include modifying the installation methods, increasing the bond length, or changing the micropile type. Any modification that necessitates changes to the structure shall require the Engineer's prior review and acceptance. Any modifications of design or construction procedures or cost of additional verification test piles and load testing shall be at the Contractor's expense. At the completion of verification testing, test piles shall be removed down to the elevation specified by the Engineer.

3.6.5 Proof Load Tests

Perform proof load tests on the first set of production piles installed at each designated substructure unit prior to the installation of the remaining production piles in that unit. The first set of production piles is the number required to provide the required reaction capacity for the proof tested pile. The initial proof test piles shall be installed at the following substructure units _______________. Proof testing shall be conducted at a frequency of 5% (1 in 20) of the subsequent production piles installed, beyond the first 20, in each abutment and pier. Location of additional proof test piles shall be as designated by the Engineer. *(**Commentary:** The above is a guideline for new users. Experienced users may go with a lesser number of proof load tests as determined by the Owner/Engineer.)*

3.6.6 Proof Test Loading Schedule

Test piles designated for compression or tension proof load testing to a maximum test load of 1.67 times the micropile Design Load shown on the Plans or Working Drawings. *(**Commentary:** See Section 5.E.4 for more detailed proof load testing information.)* Proof tests shall be made by incrementally loading the micropile in accordance with the following schedule, to be used for both compression and tension loading:

AL = Alignment Load	DL = Design Load	
	LOAD	**HOLD TIME**
1	AL	1 minute
2	0.25 DL	1 minute
3	0.50 DL	1 minute
4	0.75 DL	1 minute
5	1.00 DL	1 minute
6	1.33 DL	**10 or 60 minute Creep Test**
7	**1.67 DL (Maximum Test Load)**	1 minute
8	AL	1 minute

Depending on performance, either a 10 minute or 60 minute creep test shall be performed at the 1.33 DL Test Load. Where the pile top movement between 1 and 10 minutes exceeds 1 mm, the Maximum Test Load shall be maintained an additional 50 minutes. Movements shall be recorded at 1, 2, 3, 5, 6, 10, 20, 30, 50 and 60 minutes. The alignment load shall not exceed 5 percent of DL. Dial guages shall be reset to zero after the ititial AL is applied.

The acceptance criteria for micropile proof load tests are:

1. The pile shall sustain the compression or tension 1.0 DL test load with no more than ______ mm total vertical movement at the top of the pile, relative to the position of the top of the pile prior to testing. *(**Commentary**: Structural designer to determine maximum allowable total pile top structural axial displacement at the 1.0 DL test load based on structure design requirements. Refer to Chapter 5 for more design guidance.)*
2. At the end of the 1.33 DL creep test load increment, test piles shall have a creep rate not exceeding 1 mm/log cycle time (1 to 10 minutes) or 2 mm/log cycle time (6 to 60 minutes).The creep rate shall be linear or decreasing throughout the creep load hold period.
3. Failure does not occur at the 1.67 DL maximum test load. Failure is defined as the load at which attempts to further increase the test load simply result in continued pile movement.

3.6.7 Proof Test Pile Rejection

If a proof-tested micropile fails to meet the acceptance criteria, the Contractor shall immediately proof test another micropile within that footing. For failed piles and further construction of other piles, the Contractor shall modify the design, the construction procedure, or both. These modifications may include installing replacement micropiles, incorporating piles at not more than 50% of the maximum load attained, postgrouting, modifying installation methods, increasing the bond length, or changing the micropile type. Any modification that necessitates changes to the structure design shall require the Engineer's prior review and acceptance. Any modifications of design or construction procedures, or cost of additional verification test piles and verification and/or proof load testing, or replacement production micropiles, shall be at the Contractor's expense.

4.0 METHOD OF MEASUREMENT.

Measurement will be made as follows for the quantity, as specified or directed by the Engineer:

- Mobilization will be measured on a lump-sum basis.
- Micropiles will be measured on a lump-sum basis.
- Micropile verification load testing will be measured on a lump-sum basis.
- Micropile proof load testing will be measured on a lump-sum basis.

The final pay quantities will be the design quantity increased or decreased by any changes authorized by the Engineer.

5.0 BASIS OF PAYMENT

The quantities accepted for payment will be paid for at the contract unit prices for the following items:

Pay Item	Unit
Mobilization and Demobilization	Lump Sum
Micropile Load Tests	Lump Sum
Micropiles and Footings	Lump Sum*
Micropiles Variations in Length to Top of Rock ..	LF**
Unexpected Obstruction Drilling	Hour***

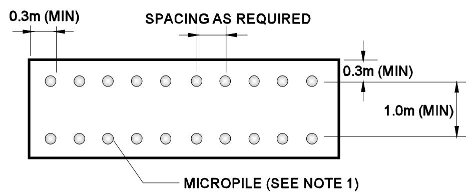
*For the option where the contractor designs the footing and number of piles, the foundation system should be bid as lump sum and a schedule of values established for progress payments after award.

**Where piles are founded in rock, micropiles will be paid on a per each basis assuming Rock at Elevation _____. Additional length or shorter length due to variations in the top of rock will be paid on a add or deduct lineal foot basis where the linear footage = Elevation ___ minus Elevation of As-Built Rock.

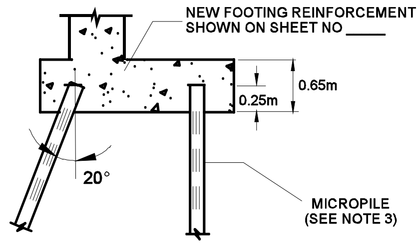
***If "obstructions" are not defined in the Standard Specifications, a definition should be added.

The contract unit prices for the above items will be full and complete payment for providing all design, materials, labor, equipment, and incidentals to complete the work.

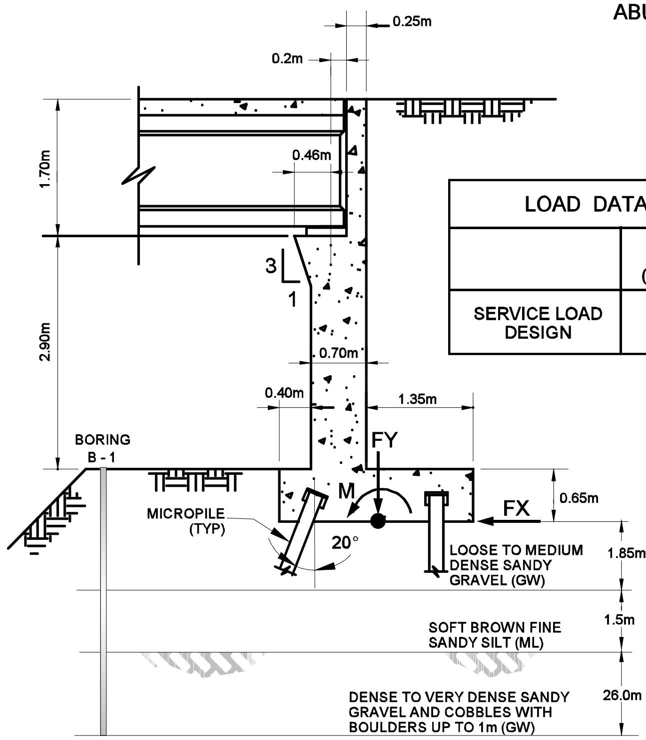
The unit contract amount for “Micropiles” shall include the drilling, furnishing, and placing the reinforcing steel and casing, grouting, and pile top attachments. The micropile Contractor is also responsible for estimating the grout take. There will be no extra payment for grout overruns.



ABUTMENT PLAN VIEW
NOT TO SCALE



ABUTMENT SECTION
NOT TO SCALE



TYPICAL SECTION
NOT TO SCALE

LOAD DATA (SLD EXAMPLE)			
	FX (kN/m)	FY (kN/m)	M (kNm/m)
SERVICE LOAD DESIGN	80.3	457.4	266.0

COMMENTARY: THIS IS THE MINIMUM AMOUNT OF INFORMATION THAT THE OWNER NEEDS TO PROVIDE IN THE CONTRACT PLANS FOR THE CONTRACTOR DESIGN-BUILD SPECIFICATION METHOD

DESIGN CRITERIA

DESIGN: 1996 AASHTO STANDARD SPECIFICATIONS, 16TH EDITION

REINFORCED CONCRETE: F_y = 520 MPa
f_c' = 27.6 MPa

STRUCTURAL STEEL MICROPILES: f_y = 34.5 TO 55.2 MPa

PILE TOP ATTACHMENTS: f_y = 24.8 TO 34.5 MPa

NOTES

1. PILE DESIGN AND LAYOUT TO BE DESIGNED BY MICROPILE CONTRACTOR IN ACCORDANCE WITH PROJECT SPECIFICATIONS AND DESIGN CRITERIA & LOAD DATA ON THIS SHEET.
2. FOOTING REINFORCEMENT HAS BEEN DESIGNED FOR A FACTORED DESIGN LOAD OF _____ kN/m. ANY ADDITIONAL FOOTING REINFORCEMENT REQUIRED BY CONTRACTOR DESIGNED PILE LAYOUT SHALL BE SUBMITTED FOR APPROVAL.
3. MICROPILE BEARING PLATE SHALL EFFECTIVELY DISTRIBUTE THE DESIGN FORCE TO THE FOOTING CONCRETE SUCH THAT THE CONE SHEAR REQUIREMENTS OF ACI 349, APPENDIX B, ARE MET AND THE BENDING STRESS DOES NOT EXCEED $0.55f_y$ FOR STEEL.
4. PILE TO FOOTING CONNECTION TO BE DESIGNED BY MICROPILE CONTRACTOR.
5. SEE PROJECT SPECIFICATIONS FOR SOILS INFORMATION

CONTRACTOR DESIGN/BUILD OF FOUNDATIONS (MICROPILES AND FOOTINGS)

SAMPLE CONTRACT PLANS

Micropile Installation Log

Project Name:	
Contract No.:	

Pile Designation #		Time @	
Installation Date		Start of Drilling	
Drill Rig/Drill Method		Start of Grouting	
Drill Rig/ #, Operator		Pile Completion	
Grout Plant #, Operator		Total Duration	

Drill Bit Type and Size		Cement Type	
Casing Dia./Wall Thickness		Admixtures	
Pile Inclination		w/c Ratio	
Reinforcement Size/Length			

Pile Length Above B.O.F.		Tremie Grout Quantity (bags)	
Upper Cased Length		Pressure Grout Quantity (bags)	
Cased and Bond Length (Plunge)		Grouting after plunge (bags)	
Bond Length Below Casing		Total Grout Quantity (bags)	
Total Pile Length		Grout Ratio (bags/m bond)	

Comments - Pile Drilling

Depth from B.O.F (m)	Soil / Rock Description	Flush Description	Comments

Comments - Pile Grouting

Depth from B.O.F (m)	Pressure Range Max/Average (MPa)	Comments

B.O.F. = Bottom of Footing (ref. FHWA-SA-97-070)

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APPENDIX B

Washington State DOT

Guide for Preparation of a

"Summary of Geotechnical Conditions"

1. Describe subsurface conditions in plain English. Avoid use of geologic jargon and/or nomenclature which the contractors (and their lawyers) will not understand. Identify depths/thicknesses of the soil or rock layers and their moisture state and density condition. Identify the depth/elevation of groundwater and state its nature (e.g. perched, regional, artesian, etc.). If referring to an anomalous soil, rock or groundwater condition, refer to boring log designation where the anomaly was encountered.

2. For each structure, if necessary, state the impact the soil, rock or groundwater condition may (will) have on construction. Where feasible, refer to the boring log(s) or data which provides the indication of the risk. Be sure to mention the potential of risk for:

   - Caving ground.
   - Slope instability due to temporary excavation, or as a result of a project element (e.g. buttress, tieback wall, soil nail cuts).
   - Groundwater flow and control, if anticipated, in construction excavations or drill holes.
   - Dense layers (may inhibit pile driving, shaft or tunnel excavation, drilling for nails, micropiles, dowels or anchors).
   - Obstructions, including cobbles or boulders, if applicable.

3. Where design assumptions and parameters can be affected by the manner in which the structure is built, or if the assumptions or parameters can impact the contractor's construction methods, draw attention to these issues. This may include:

   - Soil or rock strengths (e.g. point load tests, RQD, UCS, UU, CU tests, etc.).
   - Whether shafts or piles are predominantly friction or end bearing by design.
   - The reasons for minimum tip elevations or bond lengths specified in the contract.
   - Downdrag loads and the effect on design/construction.
   - If certain construction methods are required or prohibited, state the (geotechnical). reason for the requirement.
   - Liquefaction potential and impact on design/construction.
   - List of geotechnical reports or information. This should include the project specific report and memoranda (copies available at the Project office) as well as pertinent reports which may be historical or regional in nature.

4. List of geotechnical reports or information. This should include the project specific report and memoranda (copies available at the Project office) as well as pertinent reports which may be historical or regional in nature.

5. The intent of the Summary is to inform the contractor of what we, as the geotechnical designers, know or strongly suspect about the subsurface conditions. The Summary should be brief (1 or 2 pages maximum). It should not include tabulations of all available data (e.g. boring logs, lab tests, etc.). Only that data which is pertinent to the adverse construction condition you are anticipating should be mentioned. It should not include sections or commentary about structures or project elements about which we have no real concerns.

EXAMPLE

SUMMARY OF GEOTECHNICAL CONDITIONS

Micropile Manual-Sample Problem No. 1

Site and Subsurface Conditions

The soils at the site consist of loose to very dense sandy gravel with cobbles and boulders. The soil is highly permeable in nature. Boulders up to 1 meter in diameter or larger are likely to be encountered frequently. Augers were used to drill the test borings in the top 4.5 meters. Below 4.5 meters coring was required to drill through the very dense bouldery soil. Ground water throughout the site is near the river level and will follow river level fluctuations.

Potential Impact of Site Conditions on Foundation Construction

It is anticipated that the bottom of the footing excavation will be located at or above the groundwater level, provided that the footing is constructed during the summer or fall when the river level is relatively low. Due to the coarse, clean nature of the soils at the footing foundation level, if groundwater is encountered (as would be the case during periods of high water in the river), it will be difficult to keep the footing excavation dewatered, as water flow rates through the soil will be high.

The bouldery conditions at the site will have a impact on the construction of the micropiles at Abutments 1 and 2. The contractor should be aware that these conditions will reduce the rate at which the contractor can construct the micropiles. Cobbles and boulders should be expected throughout the soil mass starting about 4 meter depth below the abutment bottom of footing elevations, therefore rock drilling equipment will be required. Considering the presence of ground water and the lack of soil cohesion, the soil will quickly cave without support. Drilling slurry is not likely to be effective due to the gravelly nature of the soil, and therefore, full depth casing is required during drilling. Permanent casing is to be left in place into the top of the grout bond zone, as shown on the plans.

Due to the highly permeable nature of the gravel, grout overrun beyond the theoretical drill hole quantity should be expected. The contractor is responsible for estimating grout quantity. There will be no extra payment for grout overruns.

Available Geotechnical Reports

The following geotechnical report contains design and construction information relevant to the project and is available at the Project Engineer's office:

Cook, Kerry B., Chief Geotech Engineer (and Ace Fisherman), "*Sample Problem No. 1 Bridge Replacement Project*", FHWA-WFLHD Geotechnical Report No. 00, January 1996.