2.6 MICROPILES

INTRODUCTION AND HISTORY

Background

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

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

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

Scope

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

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

Micropile construction techniques are amongst those used to install soil-nails - sub-horizontal in situ reinforcements used in excavation support and slope stabilization (Fig. 2.6-4). However, soil nailing was regarded in concept, design, and function to be beyond the scope of the report and had already been the subject of major Federal (NCHRP 1987, FEWA 1996) and private studies (Juran and Elias 1996, and Bruce 1993).

![Diagram of soil conditions and micropiles](image)

**Fig. 2.6-1.** Overlap of in situ reinforcement applications: (a) nails, and (b) micropiles, in excavations; (c) micropiles, and (d) nails, for general slope stabilization; and (e) dowels to stabilize residual slips in clay (Bruce and Jewell 1996, 1997).

**SOIL IMPROVEMENT AND GEOSYNTHETICS**

**Historical Note**

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

**FUNDAMENTAL CONCEPTS**

**Characteristics and Definitions**

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

With conventional cast-in-place replacement piles, in which most, and occasionally all, the load is resisted by concrete as opposed to steel, small cross-sectional area is synonymous with low structural capacity. Micropiles, however, are distinguished by not having followed this pattern; innovative and vigorous drilling and grouting methods like those developed in related geotechnical practices such as ground anchorages permit high grout-grout bond values to be generated along the micropile's periphery. To exploit this potential benefit, therefore, high capacity steel elements, occupying up to 50 percent of the hole volume, can be used as the principal (or sole) load-bearing elements, with the surrounding grout serving only to transfer, by friction, the applied load between the soil and the steel. End-bearing is not relied upon, and in any event, is relatively insignificant given the pile geometries involved (Fig. 2.6-2). Early micropile diameters were around 100 mm (4 in), but with the development of more-powerful drilling equipment, diameters of up to 300 mm (12 in) are now considered practical. Thus, micropiles are capable of sustaining surprisingly high loads (compressive loads of over 5000 kN (120 kip) have been recorded), or conversely, can resist lower loads with minimal movement.

The development of highly specialized drilling equipment and methods also allows micropiles to be drilled through virtually every ground condition, natural and artificial, with minimal vibration, disturbance and noise, and at any angle below horizon. Micropiles are therefore used widely for underpinning existing structures,
Here, the piles are not heavily reinforced since they are not individually and directly loaded; rather they encircle a zone of reinforced, composite, confined material that offers resistance with minimal movement. The piles are fully bonded over their entire length so that as this case to work, the soil over its entire profile is likely to some reasonable degree of competence. Luzzi’s research (1982) has shown that a positive “network effect” is achieved in terms of load/movement performance, such as the effectiveness and efficiency of the reticulated pile/slab interaction in the composite mass.

![Diagram of pile/slab interaction](image)

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

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

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

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

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

There is no question that the drilling method and technique will affect the magnitude of the ground ground and which can be mobilized. On the other hand, the act of placing the reinforcement cannot be expected to influence this bond development. Generally, however, transitional practices both in micropiles (e.g., French NRM DTU 12.2, 1992) and in ground anchors (e.g., British Code BS 8081, 1989) confirm that the method of grouting is generally the most sensitive connection control over ground bond development. The following classification of micropile type, based primarily on the type and pressure of the grouting is therefore adopted. It is shown schematically in Fig. 2.6-4.

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

- **Type B**: Next cement grout is injected into the drilled hole as the temporary steel drill casing or auger is withdrawn. Pressures are typically in the range of 0.3 to 1 MPa (5 to 20 ksf), and are limited by the ability of the soil to maintain a grout tight "seal" around the casing during its withdrawal, and the need to avoid hydrofracture pressures and/or excessive grout consumptions.

- **Type C**: Next cement grout is placed in the hole as for Type A. Between 15 and 25 minutes later, and so before hardening of this primary grout, similar grout is injected, once, via a preplaced sleeved grout pipe at a pressure of at least 1 MPa (20 ksf). This type of pile, referred to in France as IGU (Injexion Globale et Unitaire), seems to be common practice only in that country.

- **Type D**: Next cement grout is placed in the hole as for Type A. Some hours later, when this primary grout has hardened, similar grout is injected via a preplaced sleeved grout pipe. In this case, however, a packer is used inside the sleeved pipe so that specific horizons can be treated, several times if necessary, at pressures of 1 to 8 MPa (150 to 800 ksf). This is referred to in France as IRS (Injection Répétitive et Sélective), and is common practice worldwide.

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

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

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

**Applications**

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

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

**Fig. 2.6-5.** Classification of micropile applications.
Table 2.6-1. Details of microtipe classification based on type of grouting (continued).

<table>
<thead>
<tr>
<th>Microtipe Type and Grouting Method</th>
<th>Subtype</th>
<th>Drill Casing</th>
<th>Reinforcement</th>
<th>Drains</th>
<th>Comparison with Other Types or Classifications</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>TYPE A</strong> Gravity grout only</td>
<td>A1</td>
<td>Temporary or unlined (open hole or auger)</td>
<td>None, monofilament, cables or tube</td>
<td>Sand/cement mortar or cement grout, intended to be base of hole (see example), no grouts or pressure applied</td>
<td>Original &quot;Root Pile&quot; \nGEWI Pile</td>
<td>Mobility of Type A microtipes used only when bend zone is in rock or stiff cohesive soil. \nIncludes unlined or uncoated piles, but very rare. \nUncoated microtipes may not be used (or allowed by code).</td>
</tr>
<tr>
<td></td>
<td>A2</td>
<td>Permanent, full length</td>
<td>Drill casing itself</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>A3</td>
<td>Permanent, upper shaft only</td>
<td>Drill casing in upper shaft, backfill or tube in lower shaft (may extend full length)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>TYPE B</strong> Pressure-grouted through the casing during withdrawal</td>
<td>B1</td>
<td>Temporary or fully extended</td>
<td>Monosand(s) or tube (cages rare due to lower structural capacity)</td>
<td>Cement grout is first injected into holes (see example), then pressure (up to 1 MPa) typically is applied to additional cement injected during withdrawal of casing</td>
<td>Later &quot;Root Pile&quot; \nFrench Type I \nItalian &quot;Root Pile&quot; \nGEWI Pile</td>
<td></td>
</tr>
<tr>
<td></td>
<td>B2</td>
<td>Permanent, full length</td>
<td>Drill casing itself</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B3</td>
<td>Permanent, upper shaft only</td>
<td>Drill casing in upper shaft, backfill or tube in lower shaft (may extend full length)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

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

<table>
<thead>
<tr>
<th>Microtipe Type and Grouting Method</th>
<th>Subtype</th>
<th>Drill Casing</th>
<th>Reinforcement</th>
<th>Drains</th>
<th>Comparison with Other Types or Classifications</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>TYPE C</strong> Primary grout placed under gravity head, then pull of secondary &quot;global&quot; pressure grouting</td>
<td>C1</td>
<td>Temporary or unlined (open hole or auger)</td>
<td>Monosand(s) or tube (cages rare due to lower structural capacity)</td>
<td>Cement grout is first injected into holes (see example), then pressure (up to 1 MPa) typically is applied to additional cement injected during withdrawal of casing</td>
<td>French Type III (friction Global et Stabilité)</td>
<td>Typically, the cement is used in France. \nSecondary grouting via a separate sliced pipe or through the reinforcement tube equipped with sleeve.</td>
</tr>
<tr>
<td></td>
<td>C2</td>
<td>Not possible</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C3</td>
<td>Not conducted</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td><strong>TYPE D</strong> Primary grout placed under gravity head, then pull of secondary &quot;global&quot; pressure grouting</td>
<td>D1</td>
<td>Temporary or unlined (open hole or auger)</td>
<td>Monosand(s) or tube (cages rare due to lower structural capacity)</td>
<td>Cement grout is first injected into holes (see example), then pressure (up to 1 MPa) typically is applied to additional cement injected during withdrawal of casing</td>
<td>French Type IV (Friction Global et Stabilité) \nTable</td>
<td>Typically, the cement is used in France. \nSecondary grouting via a separate sliced pipe or through the reinforcement tube equipped with sleeve.</td>
</tr>
<tr>
<td></td>
<td>D2</td>
<td>Not possible</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td></td>
<td>D3</td>
<td>Permanent, upper shaft only</td>
<td>Via packer, as many times as necessary to achieve bond</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Notes:
- "Notes" column contains additional details about the grouting methods and their applications, including notes on the use of cement grout and the effectiveness of the grouting methods under various conditions.
For micropiles used as in situ reinforcement, the original CASE 2 network featured low capacity Type A piles. The research by Pearlman et al. (1992) on groups of piles suggests that in certain conditions and arrangements, the piles themselves are principally, directly, and locally subjected to bending and shearing forces. This would, by definition, be a CASE 1 design approach. Such piles are usually highly reinforced and of Type A or B only.

Whereas CASE 1 and CASE 2 concepts alone or together can apply to slope stabilization and excavations on support, generally only CASE 2 concepts apply to the other major applications of in situ reinforcement. Little commercial work has been done in these applications (with the exception of improving the structural stability of tailhongs (Fig. 2.6-35). However, the potential is real and the subject is being actively pursued in the “FOREVER” program in France. Table 2.6-2 summarizes the link between application, classification, design concept, and constructional method. It also provides an indication of how common each application appears to be worldwide.

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

<table>
<thead>
<tr>
<th>APPLICATION</th>
<th>STRUCTURAL SUPPORT</th>
<th>IN-SITU REINFORCEMENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sub-application</td>
<td>Underpinning of Existing Foundations</td>
<td>Type A (end bearing)</td>
</tr>
<tr>
<td></td>
<td>New Foundation</td>
<td>Type A (end bearing)</td>
</tr>
<tr>
<td></td>
<td>Seismic Retrofitting</td>
<td>Type A (end bearing)</td>
</tr>
<tr>
<td>Design concept</td>
<td>CASE 1</td>
<td>CASE 1 and CASE 2 with minor CASE 1</td>
</tr>
<tr>
<td></td>
<td>CASE 2</td>
<td>CASE 2 with minor CASE 1</td>
</tr>
</tbody>
</table>

| Construction type | Type A (end bearing) |
| Type B in soil | Type A in soil |
| Type C (only in France) | Type A in soil |

<table>
<thead>
<tr>
<th>Estimate of relative application</th>
<th>Probably 1% of total work applications</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 5%</td>
<td>Less than 1%</td>
</tr>
<tr>
<td>Most known to date</td>
<td>Less than 1%</td>
</tr>
</tbody>
</table>

SOIL IMPROVEMENT AND GEOSYNTHETICS

DESIGN CONCEPTS

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

- Design of single micropiles [i.e., CASE 1, axial and lateral loads (101 pages)]
  - Geotechnical (external) design
  - Structural (internal) design

- Design of Groups of Micropiles (CASE 1) (73 pages)
  - Experimental Observations
  - Axial Loading (Load and Movement Calculation)
  - Lateral Loading (Load and Deflection Calculation)
  - Combined Loading
  - Cyclic Loading
  - Specific Methods for Foundation Underpinning, In situ Soil Reinforcement, Slope Stabilization and Crossing Slopes

- Design of Networks of Micropiles (CASE 2) (17 pages)
  - Foundation Underpinning
  - Slope Stabilization

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

The static design methods for single CASE 1 piles draw from conventional bored pile theory, prestressed ground anchor practice, and of course from the more limited pool of micropile knowledge, per se. In competent soils, it is accepted that the governing capacity calculation is the internal structural capacity of the pile itself, such as the great magnitude of ground/bond capacity which can be developed with contemporary drilling and grouting methods. This therefore focuses attention on the size, nature and yield strength of the reinforcement, assuming that the ground bond (ultimate 1.5 MPa (208 kpsi) for plain bar, 3 MPa (450 psi) for deformed bar) is not critical, and that the contribution of the ground, in compression, is clearly defined (typical allowable design stress of 0 to 40 percent U.C.S.). Allowable stress values for steel range to 594%. Movement calculations are driven by the same factors, plus the "effective" free length, i.e., that length below the head over which the pile reinforcement is actually being compressed. In this regard, the research of Bruce et al. (1992) has shown how this effective free length can be accurately calculated based on cyclic load-movement test data. Table 2.6-3 summarizes some geotechnical design guidelines.
Regarding groups, Lizzii (1982) showed, via laboratory tests, that for interpile spacings of 2 to 7 pile diameters, the axial load bearing capacity of the pile group was up to 50 percent greater than the sum of the individual piles in that group. Although this observation has been supported by numerous other researchers, no advantage appears to be taken of this “positive group effect” in contemporary micropile practice, although, doubtless it does contribute to the surprisingly “stiff” response of micropile supported structures in practice.

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

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

CONSTRUCTION

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

Drilling

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

Thereafter, the technical and economic success of the job is largely dependent on the contractor’s ability to drill through the overburden, any obstructions (natural and artificial), and into the bedrock if that is where the pile is to be founded. There are fundamentally six generic methods of drilling overburden, as summarized in Table 2.6-4, and the most appropriate method is selected with respect to the site, the subsurface conditions, and the type and size of the pile.
Table 2.6.1: Geotechnical Design Guidelines for Single Piles (continued).

<table>
<thead>
<tr>
<th>Load Level</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate Load</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td>F</td>
<td>G</td>
<td>H</td>
<td>I</td>
</tr>
<tr>
<td>Measurement Control</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td>F</td>
<td>G</td>
<td>H</td>
<td>I</td>
</tr>
<tr>
<td>Deflection Control</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td>F</td>
<td>G</td>
<td>H</td>
<td>I</td>
</tr>
<tr>
<td>Load</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td>F</td>
<td>G</td>
<td>H</td>
<td>I</td>
</tr>
<tr>
<td>Max. Load</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
<td>F</td>
<td>G</td>
<td>H</td>
<td>I</td>
</tr>
</tbody>
</table>

Fig. 2.6.6. "Group Effect" and "Network Effect" model test data for different microdrilled arrangements in coarse-sieved sand (from Lize: 1975).

Arrangement of Piles in Model Test:

- Test No. 1
- Test No. 2
- Test No. 3

Load-Carrying Capacities:

1. Single Pile $P = 14 \times 0.5kN = 7kN$
2. 10 Vertical Piles $P = 24kN$
3. 10 "Dweiped" Pile $P = 22kN$

Axial Test Loads:

$6\times14 = 140 \times 0.5 = 70kN$
$6\times22 = 132 \times 1 = 132kN$
$6\times22 = 132 \times 1 = 132kN$
Drilling rigs are typically diesel- or electric-hydraulically powered, and may be crawler or frame mounted. Special rigs have been developed for very restricted site conditions, and these rigs, although they may be relatively small in width and/or height, can provide considerable rotary power—essential for overburden drilling.

Drilling is most commonly conducted with water flush, although foam flush is frequently used in very difficult drilling conditions (Bruce et al. 1993). Air flush should only be permitted with extreme caution when drilling over hard rock in urban environments for fear of causing pneumatic bursting of the ground and structural failures.

Reinforcement

Reinforcement commonly consists of one or more steel bars, Grade 60 or 150. Typical bar diameters range from 25 to 63 mm (1 to 2.5 in.). Individual bar pieces are coupled together in lengths, which, depending on the site circumstances, may vary from 1 m to over 6 m (3 to 20 ft). Centralizers, usually plastic, should be located at 2 m intervals along each bar.

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

### Table 2.6-5: Axial tension and compression loads for ASTM 80 steel casing.

<table>
<thead>
<tr>
<th>Casing DC</th>
<th>5-1/2</th>
<th>7</th>
<th>6-6/8</th>
</tr>
</thead>
<tbody>
<tr>
<td>in/mm</td>
<td>132.7</td>
<td>177.8</td>
<td>244.5</td>
</tr>
<tr>
<td>Wall Thickness</td>
<td>0.231</td>
<td>0.448</td>
<td>0.672</td>
</tr>
<tr>
<td>in/mm</td>
<td>9.17</td>
<td>12.65</td>
<td>11.29</td>
</tr>
<tr>
<td>Steel Area</td>
<td>5.83</td>
<td>10.17</td>
<td>13.37</td>
</tr>
<tr>
<td>in²/ft²</td>
<td>3.760</td>
<td>6.563</td>
<td>8.736</td>
</tr>
<tr>
<td>Y'ld Load</td>
<td>496</td>
<td>814</td>
<td>1,066</td>
</tr>
<tr>
<td>kips/ton</td>
<td>2.075</td>
<td>3.016</td>
<td>4.826</td>
</tr>
</tbody>
</table>

Growing

Grouts used in the Primary injection phase are stable and have high 28-day unconfined compressive strengths—typically in excess of 25 MPa (500 kPa). In the United States, bentonite-based mixes of W/C = 0.40 to 0.53 are common, whereas in other countries, sand/cement mixes are more widely used, especially where grout takes into the surrounding formation (e.g., karstic limestone conditions) may be excessive. Special grouts and/or grout water chemistries may be required for the use of special cements, but usually a Type I or II is sufficient—Type III if higher early strength is required. Additives are rarely necessary, although plasticizers are useful in very hot conditions or when pumping distances are substantial. Mixing is best conducted in high speed, high shear mixers.

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

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

### QA/QC and Testing

**Drying Installation**

Full details are to be maintained through all the construction processes to ensure the final quality of the product. Of particular importance is the recording of all relevant grout pressure-volume-depth-time data, since to a large extent, the grouting process is a major construction determinant of the ground bond capacity. Certain contractors also favor testing the 120-day grout (e.g., specific gravity, unit weight) prior to injection, to ensure that the injected grout meets the specifications, since samples for strength testing give only retrospective proof of the ability of the grout to reach the specified quality.

**After Installation**

For axially-loaded CASE I piles, load tests are conducted on a representative number of elements. It is common to use ASTM D 1143-81: (Compression) and ASTM D 3555-87 (Tension) [ASTM 1992], although the information yielded from both can be greatly expanded by incrementally cycling the load, in the fashion of Performance Anchor testing (PTI 1996). As shown in Fig. 2.6-8, such testing permits the total pile movements to be partitioned into permanent and elastic components, so allowing fundamental investigations into load transfer mechanisms. CASE I piles subjected to lateral loading can be tested according to ASTM D 3966-81.
There is no common, absolute set of acceptance criteria for CASE 1 total-loaded piles, although many "solutions" based on geometric analyses of load-settlement curves have been proposed (Kulhawy et al. 1993). Optimally, the acceptance criteria are selected project by project, with respect to short-term movement, and creep performance. Analyzing pile load test data to meet these criteria is best conducted with the full insight afforded by cyclic loading programs.

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

THE FUTURE

In the United States, as is the case worldwide, new geotechnical and structural challenges for both static and seismic retrofit are fostering the continuing growth of micropile technology. In particular, the demands of seismic engineering are providing new impetus to the study and understanding of pile performance in general, and pile networks especially.

Aided by the classification breakthrough made by FHWA (1990), researchers in the United States, France, and Japan are poised to close the gap that still exists between the level of analytical understanding and the excellence of the construction, testing, and performance knowledge. One consequence will be a rapid growth in the application of CASE 2 structures, optimally and rigorously designed to ensure efficient and economic solutions especially for seismic applications.

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

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