Ground Anchors and Micropiles: A Decade of Progress in the United States

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ABSTRACT: Ground anchors have been used since 1934, and the history of micropiles began less than 20 years later. Both technologies have been widely employed in a range of urban engineering projects related to earth retention, slope stabilization, and structural support applications. Within the last decade, however, their use has grown rapidly in the United States, and this increasing popularity has been reflected in a number of significant Federal and industry funded studies and documents. This paper attempts to illustrate these milestones, with reference to these documents and significant case histories.

1. INTRODUCTION

The last decade of the twentieth century has seen the focus of the American specialty geotechnical construction industry continue to shift towards urban development and redevelopment (Bruce, 1995). This direction follows, by many years, similar trends in densely populated older areas in Western Europe and Asia in particular where the need to construct and upgrade transportation, sewerage, and commercial facilities fostered the evolution of many, varied construction techniques and specialists. Direct consequences of contemporary urban engineering projects are the requirements to provide security of deep excavations, and the underpinning of existing structures, both requiring minimal movements. Consequently, there has been an increasing demand for the benefits of ground anchors and micropiles. These two popular, well-developed and well-understood technologies share many similarities – especially in terms of design and construction – and it is timely to compare and contrast their growth in the 1990s in the United States.

In the following discussion, the difference in their respective range of applications should be borne in mind: anchors also continue to be a standard solution for stabilizing concrete dams threatened by sliding, overturning, and seismicity, whereas the potential of micropiles continues to be exploited for a range of solutions required for industrial and highway applications.

2. ANCHORS

2.1 *Historical Perspective*

Prestressed ground anchors have been used for earth retention and dam stabilization in the U.S. since 1960 (Gould, 1990), typically employing European technology, acquired under license, or applied under joint venture. From the 1970s, some form of standardization was applied nationally via the Prestressed Concrete Institute (1974) and the Post Tensioning Institute (PTI) (1986). These guidelines, together with active promotion by the specialty contractors, federal studies, professional society engineering conferences, and "local" specifications by State Departments of Transportation, spread awareness of the technique and promoted ever-improving standards of application.

Arguably the most influential factor in the 1990s was the developing and publishing of the Recommendations of the Post Tensioning Institute (1996): in the continued absence of a national standard, this document has provided a pragmatic guideline for the design, construction, testing, and acceptance of ground anchors. Although local specifications still exist, and indeed proliferate, the PTI Recommendations are becoming increasingly adopted as the basis of anchor specifications nationwide.

These Recommendations pay due deference to, but do not imitate, the voluminous British Standard BS 8081 (1989), which provides both guidance and background in impressive detail. At this time in the United States, the PTI document represents a fine, necessary, and appropriate balance between the theoretical and the practical, the comprehensive and the necessary. They address the design of the anchor only, and not the design of the entire retained structure.

2.2 Highlights of the PTI Recommendations

The 1996 PTI Recommendations classified and simplified many aspects incompletely, or incorrectly addressed in previous versions. In addition, it referred to several innovations that had occurred in the interim. The committee formed by PTI to work on the revised edition comprised representatives of all parties involved in anchor work. For the owners, there were representatives from the Federal Highway Administration, the U.S. Bureau of Reclamation, the Federal Energy Regulatory Commission, and the U.S. Army Corps of Engineers, while design engineers, anchor contractors, and material suppliers were equally strongly represented. The Recommendations were reviewed in draft and endorsed by the Anchored Earth Retention Committee of the International Association of Foundation Drilling. The result of this cooperation between these often conflicting interests was a set of guidelines for the design, installation, and testing of anchors that were intended to be realistic and practical, while still satisfying the concerns for reliability and safety that are recognized worldwide.

In summarizing the most important changes and additions to each chapter of the 1986 PTI Recommendations, it should be noted that the former separation into rock, soil, and resin anchors was abandoned, since most aspects apply equally to all three types. Only Chapter 6, dealing with design, still distinguishes between them, out of necessity.

<u>Chapter 1</u> confirms that the <u>Scope</u> of the work deals with permanent and prestressed rock and soil anchors. Significantly, the units are primarily SI, with imperial (soft) equivalents in parenthesis.

<u>Chapter 2</u> on <u>Definitions</u> was expanded to include most of the terms used for anchor work in an attempt to standardize these for all parties dealing with them. Particular attention was devoted to this apparently routine section since the Committee felt it was essential to provide clear and comprehensive guidance at a time when ground anchors are being specified, designed, and constructed in ever increasing numbers by a wider-based, generalist population, as opposed to a relatively small number of innovative specialists.

<u>Chapter 3</u> on <u>Specifications</u> was broadened to list the tasks and responsibilities that need to be allocated for anchor works. Anchor contractors in the United States often report that responsibilities were being insufficiently or only vaguely addressed in project specifications. Chapter 3 also makes an appeal to all parties involved for clear communication, close cooperation, and speedy reviews of documents and submittals, especially in the start-up phase of a project. This is in line with the spirit of "Partnering" which is prevalent in United States construction practice (Nicholson and Bruce, 1992). The chapter identifies the main responsibilities which have to be allocated, and also confirms the fundamental classification of specifications: prescriptive, performance, and open. In this way, regardless of the type of specification decided upon, no critical responsibilities may go neglected, through oversight.

<u>Chapter 4</u> on <u>Materials</u> was expanded from one to eight pages. Indented strand and epoxy coated strand and bars were added for tendon materials, while reference to wire and compacted

strand was dropped due to lack of use in the United States. For evaluating adequate bond behavior of strand, bond capacity tests are now required to be performed by the manufacturer prior to supply to site. In this test, a 15.2-mm diameter strand (the most widely used strand diameter in the United States for permanent anchor tendons) embedded in a 400 mm long neat cement grout column inside a steel pipe with a grout strength of 25 to 30 MPa must not move more than 0.25 mm at the unloaded end when a 35 kN tensile force is applied to the other end of the strand. For epoxy coated strand, filling of the interstices between wires with epoxy is required, as well as the use of wedges capable of biting through the outer layer of epoxy. Stripping of the epoxy to allow the use of regular wedges is not permitted to prevent damage to the strand and its corrosion protection.

For each component of an anchor tendon, including its corrosion protection system, the appropriate American Society for Testing and Materials (ASTM) specifications are either defined or recommended. Minimum performance requirements are given for most of the anchor components, including minimum wall thicknesses for the tendon encapsulation, namely 2 to 3 mm.

Chapter 5 on Corrosion Protection underwent the most fundamental and controversial changes of all. These basically constitute a further step closer to European anchor specifications, but differences in philosophy still remain. While European standards appear to be gravitating towards technically perfect and absolutely reliable solutions for protecting the tendon against corrosion, such as triple protection, or electric isolation testing of the installed and occasionally even the stressed anchor, Americans are more prepared to look at the cost-benefit ratio of the corrosion protection system. Based on published data (FIP, 1986), the number of known anchor failures due to corrosion is a very small percentage of the total number of anchors installed, and provided there are no catastrophic consequences, such a failure rate can be an acceptable construction and performance risk. Considering further that there are almost no failures known in the bond length and few in the free length, electric isolation testing, as a means of confirming the integrity of the installed corrosion protection system, where the tiniest imperfection will result in rejection of the anchor, is considered too costly and impractical on a routine basis. It is required, however, in the presence of stray electric currents. More emphasis is put on the corrosion protection near the stressing end where statistics show by far the highest frequency of corrosion failures. Strong reliance is placed on the expertise of the tendon fabricator to meet the new criteria, and attention is directed towards satisfying the details as thoroughly as possible.

The corrosion protection decision tree shown in Figure 1 guides the designer in selecting the type of protection to be specified. It fundamentally distinguishes between Class I (double corrosion protection) and Class II (single corrosion protection). Selection is based on service life, soil aggressivity, consequences of failure, and costs. One notable result is that for permanent anchors, a Class II protection may be used only in non-aggressive soils for anchors where failure does not have catastrophic consequences and where the increase in cost over Class I anchors would result in an unjustifiable and considerable extra expense. Further details are provided in Table 1. This approach is already being adopted for the design of large permanent anchors for dams especially in the western states.

Chapter 6 on Design was expanded to include such general considerations as feasibility of anchors, design objectives, fully bonded versus unbonded anchors, restressable, destressable, and removable anchors, and anchor capacity/safety factors. The safety factor on the tendon at the design load is not permitted to be less than 1.67. The guide values for the typical average ultimate bond stresses for rock, cohesive, and noncohesive soil were revised upwards in response to the greater experience available. It is emphasized, however, that actual bond capacity will largely depend on the installation technique and local variations in the actual soil conditions. The value of site specific testing is underlined.

Chapter 7 on Construction consolidates much of the information given in the 1986 Recommendations, but extra emphasis is placed on proper handling, storage, and insertion of the anchor tendon in order to preserve the corrosion protection system provided, and to avoid contamination of exposed prestressing steel. Guidelines are given for achievable tolerances for drill hole inclination and deviation from its plan location. More practical guidance is provided on rock and soil drilling methods and pressure grouting techniques, including post grouting for anchors in cohesive soils or very weak, argillaceous rocks.

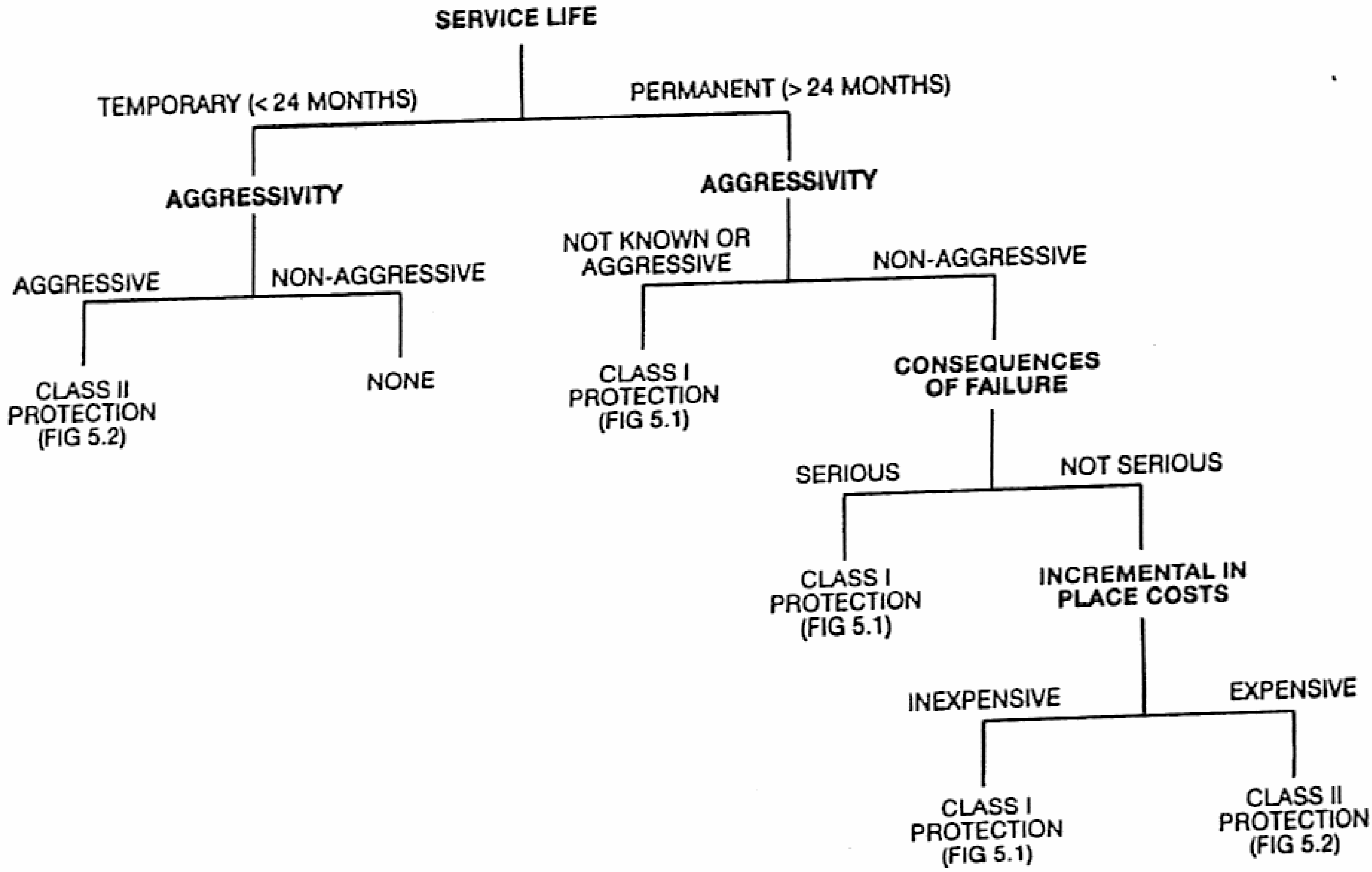

Figure 1. Corrosion protection decision tree (PTI, 1996).

Table 1. Corrosion protection requirements (after PTI, 1996).

CLASS	PROTECTION REQUIREMENTS		
	ANCHORAGE	UNBONDED LENGTH	TENDON BOND LENGTH
I Encapsulated Tendon	1. Trumpet 2. Cover if exposed	1. Grease-filled sheath, or 2. Grout-filled sheath, or 3. Epoxy for fully bonded anchors	1. Grout filled encapsulation, or 2. Epoxy
II Grout Protected Tendon	1. Trumpet 2. Cover if exposed	1. Grease-filled sheath, or 2. Heat shrink sleeve	Grout

Chapter 8 on Stressing, Load Testing, and Acceptance expands on the reasons for anchor testing, the requirements for the equipment and its setup. While the requirements and procedures for the Performance Test and the Extended Creep Test, required for soils having a Plasticity Index greater than 20, have not changed, for the Proof Test, the additional step of returning to the Alignment Load after the test load period and before off locking the anchor is recommended, especially for cases where the Proof Test results cannot be compared directly with the Performance Test results for equivalent anchors. This extra step will allow the partition of the total movement measured into permanent and elastic components for a more meaningful evaluation of the anchor performance. This proposal, the logic of which has been quickly recognized and accepted by practitioners left confused by "gray areas" in the previous Recommendations, has been long overdue in American practice.

Acceptance criteria are given for creep, movement, and lock-off load. While they do not differ widely from the 1986 Recommendations, greater emphasis was put on explaining the reasons behind the acceptance criteria and guidelines are given on what can be done in case an anchor fails to meet these acceptance criteria (Figure 2). The new Recommendations point out that the calculated minimum apparent free length of the anchor may need to be set higher than the traditional 80 percent of the designed free length, especially when later a redistribution of the free length friction could cause unacceptable structural movement or where no prestress load is allowed to be transferred in the free length by friction.

A new section on "Acceptability Based on Total Movements" was added, defining the criteria for minimum and maximum apparent free length for Proof Tested anchors where no partition of the total movement into residual and elastic movement is possible.

The provisions of another new section "Procedures in the Event of Failure during Testing" allow anchors that failed to reach the test load, to be locked off and accepted at half the failure load. Anchors that have failed the Creep Test may also be locked off at 50 percent of the failure load, or when subsequently post-grouted, need then to be subjected to an enhanced Creep Test in which the creep movement between 1 and 60 minutes is not allowed to exceed 1 mm.

It is also explained that the intrinsic creep behavior of epoxy filled strand itself is significant. Since the purpose of the test is to measure plastic movements in the bond zone, the measured creep movements of epoxy coated strand anchors must be adjusted by deducting the creep movement in the epoxy-coated strand itself. These movements are conservatively estimated with 0.015 percent of the apparent free length during the 6 to 60 minute log cycle at a test load of 80 percent of the tendon ultimate strength, and 0.012 percent at a 75 percent F_{pu} test load. However, this additional creep movement does not adversely affect the service behavior of epoxy coated strand anchors : only their higher relaxation properties, as defined in ASTM Specification A 882, need to be considered for the long term losses. Again this emphasis has been driven by field observations and professional debate: the use of epoxy coated, epoxy filled strand is increasing, principally for high quality dam anchorage projects, in which understanding of time dependent behavior - both for acceptance criteria and for assuaging owner concerns - is critical.

The new Recommendations also require wedges for strand tendons to be seated at a minimum load of 50 percent of their ultimate load capacity. Specified lock-off loads of less than that will require shimming and unshimming of the wedge plate. Overlapping wedge bites must be avoided, and are positively discouraged.

The section on "Monitoring Service Behavior" was expanded to include minimum criteria for a monitoring program. It is pointed out that such a program needs to be considered at the design stage. The monitoring program is to include the number of anchors to be monitored (typically 3 to 10 percent) their location, frequency, reporting procedures, and maximum load losses or gains allowed. An anchor monitoring program will also require monitoring of the movement of the anchored structure for a proper evaluation of anchor behavior.

A summary of the material and testing specifications referenced in the text, as well as a revised selected bibliography, completed the 1996 Recommendations.

2.3 *Further Comments*

The 1996 PTI Recommendations are intended to be a practical guide to anchor practitioners, from owners and designers to contractors and their field supervisory personnel. Their tone and content have been specifically designed to satisfy the needs peculiar to the contemporary United States anchor market, which does not otherwise enjoy the benefit of an "official" national standard at a time of rapid product expansion. They are in no way intended as a competitor to FIP or national standards - especially those of the Western European countries : the value of these documents as

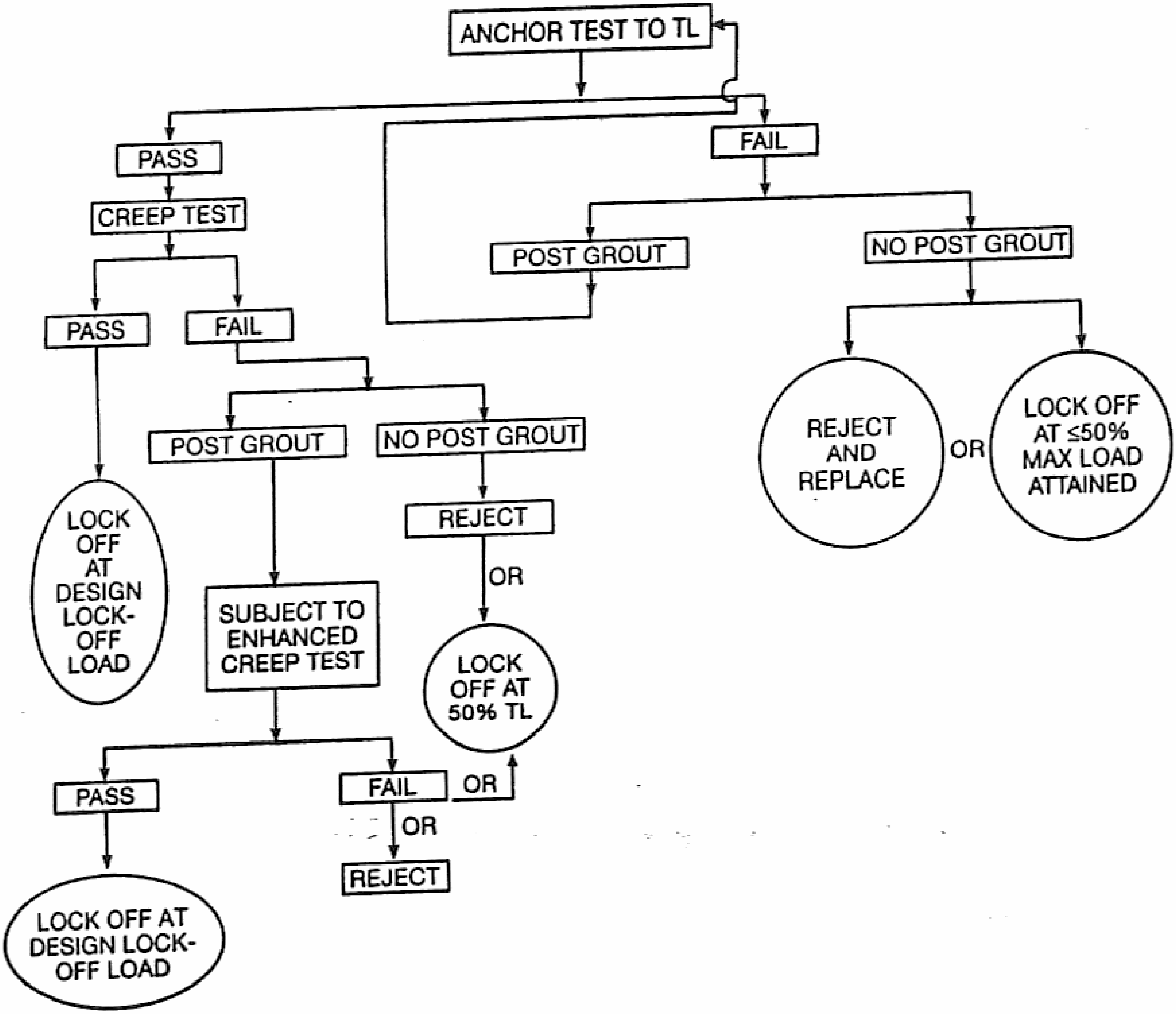

Figure 2. Decision diagram for acceptability testing of anchors (PTI, 1996).

insights and sources of knowledge is universally accepted. However, the PTI Committee intended to produce a document that clarified past inconsistencies and addressed future developments in a pragmatic fashion.

The Committee would like to believe that, upon the occasion of the next edition of the Recommendations being due, the changes will not be as extensive or fundamental as those occasioned by the developments and needs of the preceding ten years.

The market for anchors both temporary and permanent continues to grow, and is well served by specialty contractors, especially on the East and West Coasts, and the Mid West. These same contractors provide most of the innovation, especially in drilling systems, and sophisticated grouting methods such as the use of tubes á manchette to permit post grouting. Recent developments yet to be widely exploited include self-drilling anchor bars, removable tendons, and jet grouting principles.

Further data on rock anchors for dams is provided in Bruce (1997), while further information on anchors in weaker strata may be found in Bruce (1991).

3. MICROPILES

3.1 Historical perspective

Micropiles, defined as small diameter (less than 300 mm) bored, grouted in place piles incorporating steel reinforcement, were first installed in Italy in 1952. Following rapid international expansion in the subsequent two decades, the original Italian contractor, Fondedile, under the technical direction of Dr. Fernando Lizzi, established a presence in New England and began to introduce the technique to engineers throughout North America. Established U.S. specialty contractors (especially in anchors), and further European arrivals also began to exploit the potential of micropiles, particularly in urban engineering and slope stabilization projects.

The work was largely of the design-build nature, reflecting the fact that the technology was largely in the hands – and minds – of the contractors. The explosion of work around Boston in particular encouraged the creation of the first specifications (Commonwealth of Massachusetts, 1984) which began to be used – in the absence of any other micropile-specific documents – nationwide. A technology review (ASCE, 1987) focused on the traditional European types of piles, typically the " root pile" concept, wherein individual pile loads of 50 tons were considered the highest practical. By this time, however, individual pile loads of 100 tons were becoming commonplace on the East Coast, and were quickly being exploited on the West Coast, largely for seismic stabilizing applications in California (Bruce, 1992).

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 Dr. Lizzi, by then acknowledged as the "godfather" of micropiles as defined in this paper.

The activities of this group ensured that a comprehensive review of practice was conducted. However, the synergies of this group were also able to resolve a number of fundamental issues regarding various aspects of the classification, design, construction and performance of micropiles, issues which had been the cause of confusion and misunderstanding and which had held back their use in certain engineering communities in the United States.

Consequent to this study, the FHWA then funded the drafting of a complimentary Implementation Manual, focusing on the needs of owners in the individual State Departments of Transportation (FHWA, 2000), while the Deep Foundations Institute has a Micropile Committee, which organized an international workshop in 1997, and are also drafting standard Specifications.

The 1997 FHWA State of Practice review produced many innovative ideas on different aspects of micropile practice, and these are summarized below.

3.2 *New Classification of Micropiles*

It has been common to find micropiles sub-classified according to diameter, some constructional process, or by the nature of the reinforcement. However, given the definition of a micropile provided above, the FHWA team concluded that a new, rigorous classification be adopted based on two criteria:

- The philosophy of pile behavior, and
- The method of grouting.

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

- Classification based on Philosophy of Behavior. Micropiles are usually designed to transfer structural loads to more competent or stable strata. They therefore act as substitutes or alternatives for other conventional pile systems (Figure 3a). For axially loaded piles, the pile/ground interaction is in the form of side shear and so is restricted to that zone of ground immediately surrounding the pile. For micropiles used as in-situ reinforcements for slope stabilization, research by Pearlman et al. (1992) suggests that pile/ground interaction occurs only relatively close to the slide plane, although above this level, the pile group may also provide a certain degree of continuity to the pile/ground composite structure. In both cases, however, the pile (principally the reinforcement) resists directly the applied loads. This is equally true for cases when individual piles or groups of piles are used. In this context, a group is defined as a tight collection of piles, each of which is subjected to direct loading. Depending on prevailing codes relating to pile group design, the individual pile design capacity may have to be reduced in conformity with conventional "reduction ratio" concepts. These concepts were typically developed for driven piles, and so this restriction is almost never enforced for micropiles, given their mode of construction which tends to improve, not damage, the inter-pile soil.

  When axially-loaded piles of this type are designed to transfer their load only within a remote founding stratum, pile head movements will occur during loading, in proportion to the length and composition of the pile shaft between structure and the founding stratum, and the load. In this instance, the pile can be preloaded (Bruce et al., 1990) to ensure that the structure can be supported without further movements occurring. Equally, if suitably competent ground conditions exist all the way down from below the structure, then the pile can be fully bonded to the soil over its entire length and so movements under equivalent loads will be smaller than in the previous case.

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

  On the other hand, one may distinguish the small group of CASE 2 structures. Dr. Lizzi introduced the concept of micropiling when he patented the "root pile" (palo radice) in 1952. The name alone evokes the concept of support and stabilization by an interlocking, three-dimensional network of reticulated piles similar to the root network of a tree. This concept involves the creation of laterally confined soil/pile composite structure that can work for underpinning, stabilization and earth retention, as illustrated in Figure 3b. Here, the piles are not heavily reinforced since they are not individually and directly loaded: rather they circumscribe a zone of reinforced, composite, confined material that offers resistance with minimal movement. The piles are fully bonded over their entire length and so for this case to work, the soil, over its entire profile, must have some reasonable degree of competence. Lizzi's research (1982) has shown that a positive "network effect" is achieved in terms of load/movement performance, such is the effectiveness and efficiency of the reticulated pile/soil interaction in the composite mass.

  It is clear, therefore, that the basis of design for a CASE 2 network is radically different from a CASE 1 pile (or group of piles). Notwithstanding this difference, however, there will be occasions where there are applications transitional between these cases. For example, it may be possible to achieve a positive group effect in CASE 1 designs (although this attractive possibility is currently, conservatively, ignored for pile

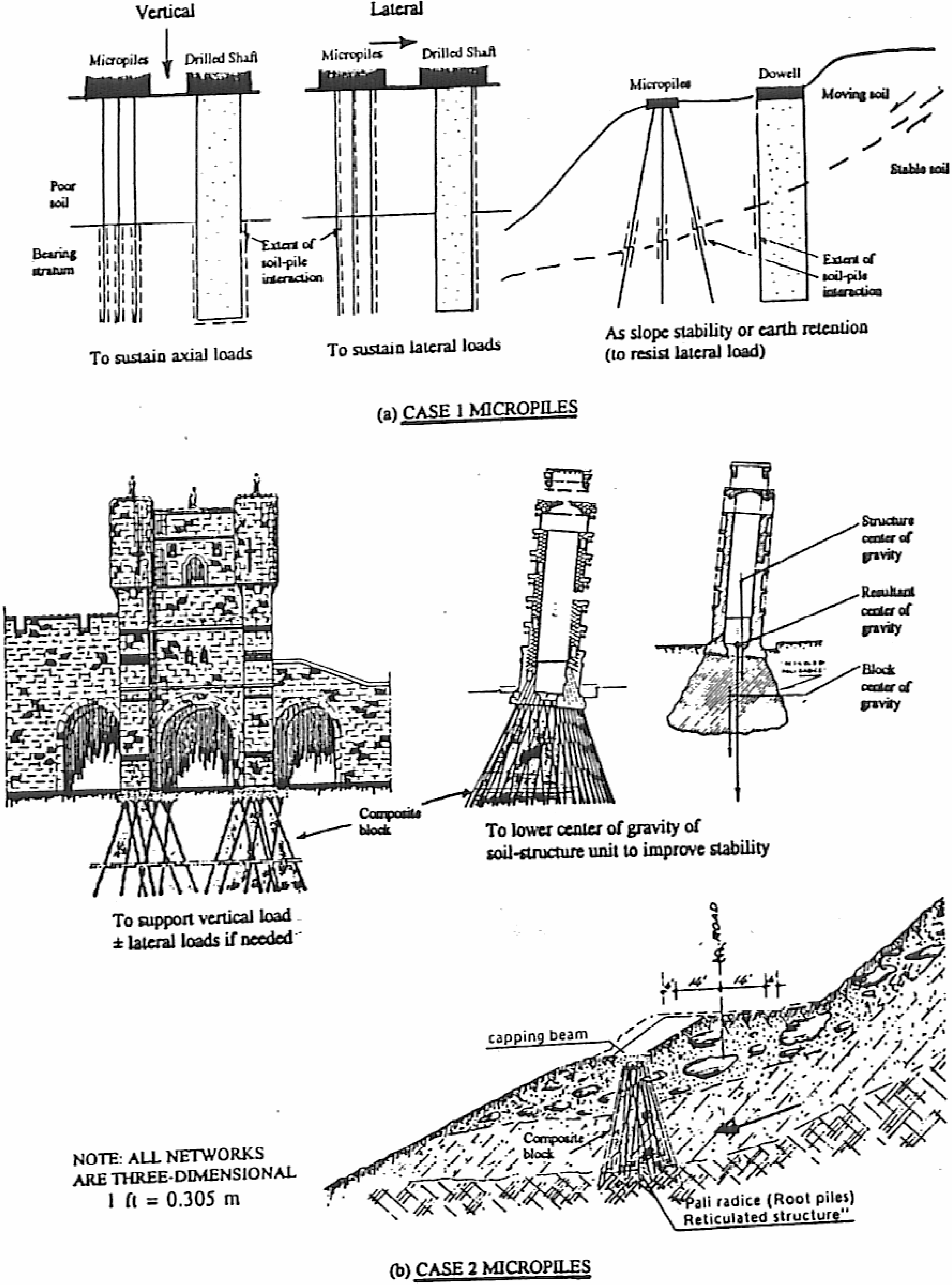

Figure 3. Fundamental classification of micropiles based on their supposed interaction with the soil.

groups), while a CASE 2 slope stability structure may have to consider direct pile loading conditions (in bending or shear) across well defined slip planes. By recognizing these two basic design philosophies, even those transitional cases can be designed with appropriate engineering clarity and precision.

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

- <u>Classification based on Method of Grouting</u>. The successive steps in constructing micropiles are, simply:
    - Drill;
    - Place reinforcement; and
    - Place and typically pressurize the grout (usually involving simultaneous extraction of the temporary steel drill casing).

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

    - <u>Type A</u>: Grout is placed in the pile under gravity head only. Since the grout column is not pressurized, sand-cement "mortars", as well as neat cement grouts, may be used. The pile drill hole may have an underreamed based (to aid performance in tension), but this is now very rare and not encountered in any other micropile type.
    - <u>Type B</u>: Neat cement grout is injected into the drilled hole as the temporary steel drill casing or auger is withdrawn. Pressures are typically in the range of 0.3 to 1 MPa, 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.
    - <u>Type C</u>: Neat cement grout is placed in the hole as for Type A. Between 15 and 25 minutes later, and so before hardening of this primary grout, similar grout is injected, once, via a preplaced sleeved grout pipe at a pressure of at least 1 MPa. This type of pile, referred to in France as IGU (Injection Globale et Unitaire), seems to be common practice only in that country.
    - <u>Type D</u>: Neat cement grout is placed in the hole as for Type A. Some hours later, when this primary grout has hardened, similar grout is injected via a preplaced sleeved grout pipe. In this case, however, a packer is used inside the sleeved pipe so that specific horizons can be treated, several times if necessary, at pressures of 2 to 8 MPa. This is referred to in France as IRS (Injection Répétitive et Sélective), and is common practice worldwide.

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

3.3 Applications

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

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

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

Table 2. Relationship between micropile application, design concepts, and construction type.

Applications	**Structural Support**	**In-situ Earth Reinforcement**			
Sub-Applications	Underpinning of Existing Foundations New Foundations Seismic Retrofitting	Slope stabilization and Excavation support	Soil Strengthening	Settlement Reduction	Structural Stability
Design Concept	CASE 1	CASE 1 and CASE 2 with transitions	CASE 2 with minor CASE 1	CASE 2	CASE 2
Construction Type	Type A (bond zones in rock or stiff clays) Type B and D in soil (Type C only in France)	Type A (CASE 1 and 2) and Type B (CASE 1) in soil	Type A and B in soil	Type A in soil	Type A in soil
Estimate of Relative Application	Probably 95% of total world applications	0 to 5%	Less than 1%	None known to date	Less than 1%

3.4 Further Comments

Micropiles are recognized nationally as an engineering tool of great value and flexibility for problems involving foundation enhancement and slope stabilization in both static and seismic cases. As with the case with anchors, consultants and government agencies have, to a large extent, caught up with the concept and are increasingly able to specify and codify it. As with anchors, however, there is still a great and correct reliance placed on the contractors to resolve the practical problems associated with the execution of the work, and so performance specifications and design-build concepts remain much more common than "traditional" prescriptive, specifications incorporating the rigid "low bid" mentality so common in other areas of American engineering practice.

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