

Anchors, Micropiles, Rock Grouting and Deep Mixing:
A Decade of Progress in the United States

Donald A. Bruce, M. ASCE

Abstract

Ground anchors have been used since 1934, and the history of micropiles began less than 20 years later. Rock grouting has been conducted in the U.S. for over 110 years, whereas its fellow ground treatment technology, deep mixing, has a local history dating from 1986. However, during the last decade of the twentieth century there were very significant advances in the application and understanding of each of these topics in North America and these are described in overview in this paper.

1. Introduction

The last ten years or so have seen the focus of the American specialty geotechnical construction industry continue to shift towards urban development and redevelopment, and the remediation of hydraulic structures (Bruce, 1995). This direction lags, by several years, similar trends in densely populated, more mature regions of Western Europe and Asia in particular, where the driving need to construct and upgrade transportation, hydraulic, sewage and commercial facilities has fostered the evolution of many varied innovative construction techniques and specialists. Direct consequences of contemporary urban engineering projects include the provision of structural and hydraulic security for deep excavations and high structures, the underpinning of existing buildings and the treatment of vast volumes of soft, compressible, permeable and/or liquefiable soils and fills. These applications have, in turn, driven developments in anchors, micropiles, grouting and deep mixing. Each of these techniques enjoys a large and rich technical literature, and is actively promoted via a variety of governmental and industry sponsorships. The purpose of this paper is to provide a brief insight into each of the technologies to illustrate the key developments which have occurred in the last ten years.

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 agencies, professional society engineering conferences, and “local” specifications by State Departments of Transportation and Corps of Engineers District Offices, 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. It has formed the basis of a more recent FHWA Geotechnical Engineering Circular on Ground Anchors (1999) which covers the wide ground of overall anchor system design as well.

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. 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 professional 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.

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

Chapter 2 on Definitions 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.

Chapter 3 on Specifications 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.

Chapter 4 on Materials 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 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.

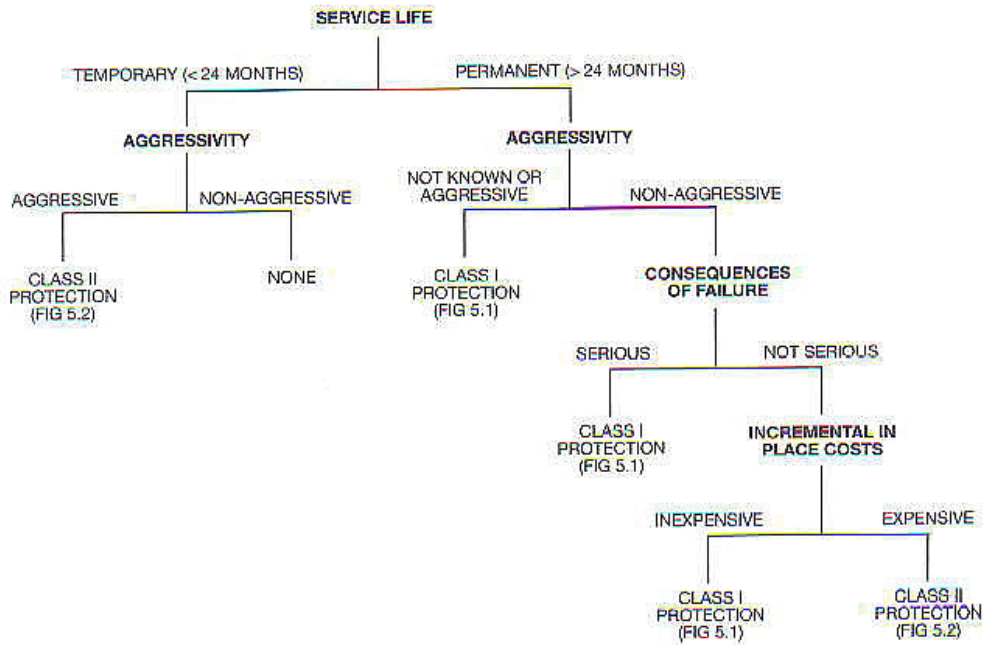


Figure 1. Corrosion protection decision tree (PTI, 1996).

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 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.

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. 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 Fpu 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 following comments are noteworthy.

1. 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. 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.
2. 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.
3. For much of the decade, the industry was very keen to exploit the technical and commercial advantages of epoxy protected strand, for dam anchors (e.g., Bruce and Bianchi, 1992; Bruce, 1997; and Bogdan, 2001). However, certain systematic problems were reported unofficially with the use of this material, especially as related to short term

load holding efficiency at the wedges. These concerns culminated, and came to general consideration, with the problems encountered at Wirtz Dam, Texas in 1999. Usage of the material rapidly declined, and a Task Force was established by the International Association of Foundation Drilling. The researches of this Task Force into material usage, problems encountered, and recommendations for future development are just beginning to be published (Bruce et al., 2002). One major step will be the writing of a supplement to PTI (1996) dealing solely with the special aspects of using epoxy protected strand.

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 newer 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 is drafting standard Specifications.

In 2001, the International Association of Foundation Drilling (Association of Drilled Shaft Contractors) also inaugurated a Micropile Committee and has been extremely active in organizing micropile seminars nationwide. This Committee has also raised funds for research, has taken over sponsorship of The International Workshop on Micropiles (the growing legacy of the International Panel established by FHWA in 1993), and is cooperating with the FHWA and the States on a variety of research and teaching initiatives.

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

3.2 New Classification of Micropiles

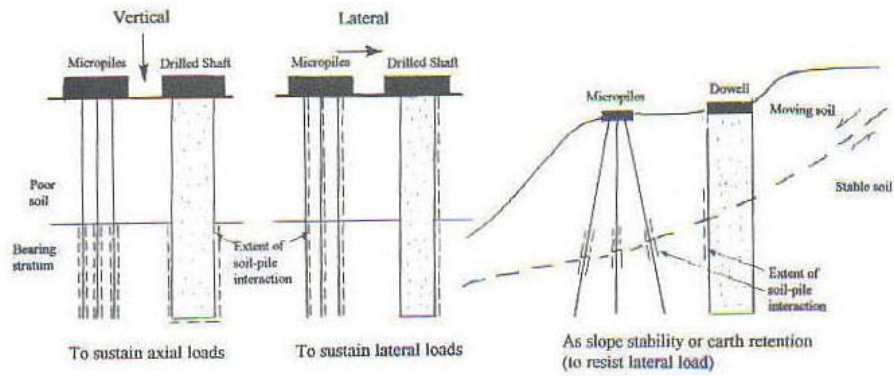
It has been common to find micropiles sub-classified according to diameter, some constructional process, or by the nature of the reinforcement. However, given the definition of a micropile provided above, the FHWA team concluded that a new, rigorous classification be adopted based on two criteria:

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

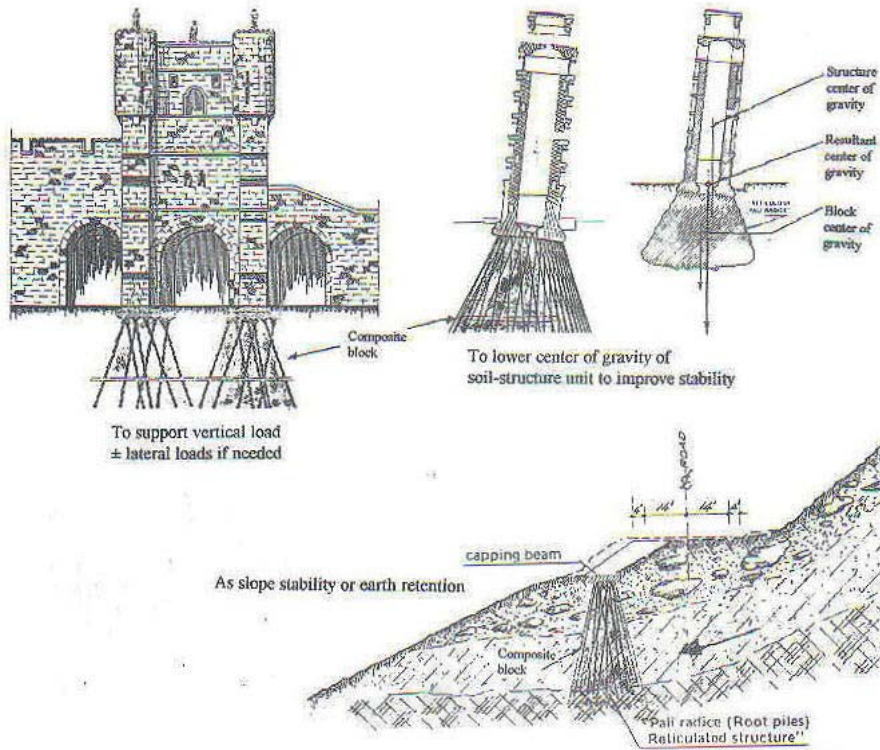
The former criterion dictates the basis of the overall design concepts, and the latter is the principal determinant of grout/ground bond capacity.

- 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 2a). 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, pile/ground interaction may occur 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 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



a. CASE 1 micropiles



b. CASE 2 micropiles

Figure 2. Fundamental classification of micropiles based on their supposed interaction with the soil.

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 2b. 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 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.

- Classification based on Method of Grouting. 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.

- Type A: Grout is placed in the pile under gravity head only. Since the grout column is not pressurized, sand-cement “mortars”, as well as neat cement grouts, 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.
 - Type B: 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.
 - Type C: 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.
 - Type D: 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.
- Combined 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.

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.

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			
		Slope stabilization and Excavation support	Soil Strengthening	Settlement Reduction	Structural Stability
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.

The rapidly growing interest shown by trade and governmental agencies in micropile technology is reflective of the national momentum which is being generated by the simple confluence of an excellent and flexible technology and a real set of market needs.

4. Rock Grouting

4.1 Background

The use of drilling and grouting methods to locate and seal fissures and voids in rock masses has been common throughout the world for over a century. While the goals of such programs have largely remained unchanged, the materials and methods have undergone remarkable change in response to technological advances and increasingly onerous site specific demands. These changes, however, have not been constant in their rate of evolution in any given part of the world. For example, little advance seems to have been made during the 50 year period of intense

activity on U.S. Federal dams from the 1920s onwards. One may cite extremely restrictive, prescriptive specifications as the main reason for the languid rate of innovation. During the last few years, however, the art of rock grouting has entered a new phase of progress, rapidly drawing it towards the status of an engineering science.

The following review provides a summary of current thinking with respect to materials, methods, QA/QC, and verification.

4.2 Materials

4.2.1 General Classification

There are four categories of materials (Bruce et al., 1997) which can be listed in order of increasing rheological performance and cost:

1. Particulate (suspension or cementitious) grouts, having a Binghamian performance.
2. Colloidal solutions, which are evolutive Newtonian fluids in which viscosity increases with time.
3. Pure solutions, being non evolutive Newtonian solutions in which viscosity is essentially constant until setting, within an adjustable period.
4. “Miscellaneous” materials.

Category 1 comprises mixtures of water and one or several particulate solids such as cement, flyash, clay, or sand. Such mixes, depending on their composition, may prove to be stable (i.e., having minimal bleeding) or unstable, when left at rest. Stable, thixotropic grouts have both cohesion and plastic viscosity increasing with time at a rate that may be considerably accelerated under pressure. Category 1 grouts are most common in rock grouting and are undergoing rapid development as a result of a markedly increased understanding of basic rheological and hydration principles.

Category 2 and 3 grouts are now commonly referred to as solution or chemical grouts and are typically subdivided on the basis of their component chemistries, for example, silicate based (Category 2), or resins (Category 3). They are rarely used in rock grouting, having application largely in “fast flow” sealing operations.

Category 4 comprises a wide range of relatively exotic grout materials, which have been used relatively infrequently, and only in certain industries and markets. Nevertheless, their importance and significance is growing due to the high performance standards which can be achieved when they are correctly used. The current renaissance in the use of hot bitumen grouts is a good example, in cases of extreme seepage conditions (Bruce et al., 2001).

4.2.2 Developments in Particulate Grouts

Due to their basic properties and relative economy, particulate grouts remain the most commonly used for both routine waterproofing and ground strengthening. The water to solids ratio is the prime determinant of their basic characteristics such as stability, fluidity, rheology, strength, and durability. Five broad subcategories can be identified:

1. Neat cement grouts
2. Clay/bentonite-cement grouts
3. Grouts with fillers (Including low mobility or “compaction” grouts, e.g. Bruce et al., 1998)
4. Grouts for special applications (Such as for antiwashout conditions)
5. Grouts for special applications
6. Grouts with enhanced penetrability.

It should be borne in mind that many particulate grouts alone are unsuited for sealing high flow, high head conditions: they will be diluted or washed away prior to setting in the desired location. However, the recent developments in rheology, stability, and hydration control technologies, and the major advances made in antiwashout additives have offered new opportunities to exploit the many economic, logistical, and long term performance benefits of cementitious compounds (Gause and Bruce, 1997). Water cement ratios are now typically in the range of 2 or 3 as a maximum, many times lower than the “traditional” mixes of the 1930s. These developments have drawn largely from experience with the wide range of additives developed primarily for the concrete industry. It is now common for a routine fissure grouting operation to feature a suite of grout mixes containing several components (in addition to cement and water), to satisfy site specific fluid and set property requirements, (Table 3), while the use of finer grind materials (e.g. DePaoli et al., 1992) has further enhanced penetrability efficiency. At the other end of the aperture spectrum, economic bulk infill mixes (e.g. for karsts, old mineral workings) are being refined using large volumes of relatively inexpensive materials such as flyash, and naturally occurring soils from gravels to clays. Admixture technology is again valuable in such mixes, providing stability, rheology and anti washout properties.

4.2.3 Developments in Other Grout Families

Given that the sodium silicate based grouts are never used in rock grouting, and that cost and environmental concerns rule out the regular use of most solution grouts in rock grouting (with the exception of certain acrylates), major developments have revolved around two groups of materials:

- Polyurethane
 - Water-reactive polyurethane: Liquid resin, often in solution with a solvent or in a elasticizing agent, possibly with added accelerator, reacts with groundwater to provide either a flexible (elastomeric) or rigid foam. Viscosities range from 50 to 100 cP. There are two subdivisions:
 1. Hydrophobic - react with water but repel it after the final (cured) product has been formed
 2. Hydrophillic - react with water but continue to physically absorb it after the chemical reaction has been completed.
 - Two component polyurethanes: Two compounds in liquid form react to provide either a rigid foam or an elastic gel due to multiple supplementing with a polyisocyanate and a polyol. Such resins have viscosities from 100 to 1,000 cP and strengths as high as 2 MPa.

Table 3. Contemporary typical cement grout additives
(Wilson and Dreese, 1998).

ADDITIVE	BENEFICIAL EFFECTS	ADVERSE EFFECTS	OTHER COMMENTS
Flyash Type C or Type F	Improves grain size distribution of cured grout, Cheap filler with pozzolanic properties. Can be used as a replacement for some of the cement and reacts with the free lime resulting from the cement hydration process. Increases durability and resistance to pressure filtration.	Increases viscosity and cohesion.	Concentrations of Type C flyash in excess of 20% by weight of cement should be avoided.
Bentonite	Reduces bleed and increases resistance to pressure filtration. Slight lubrication and penetrability benefits.	Increases viscosity and cohesion. Weakens grout.	Should be added as pre-hydrated suspension
Silica Fume	Fine grained powder which improves pressure filtration resistance and reduces bleed. Improves water repellency and enhances penetrability. Improves grain size distribution of cured grout.	Increases viscosity and cohesion.	Difficult to handle due to fineness.
Viscosity Modifiers (Welan Gum)	Makes the grout suspension more water repellent, provides resistance to pressure filtration, and reduces bleed.	Increases viscosity and cohesion.	At higher doses, provides some thixotropy to the grout which is helpful for artesian conditions.

- Hot Melts

- For certain cases seepage cut off applications, hot melts can be a particularly attractive option. Bitumens are composed of hydrocarbons of very high molecular weights, usually obtained from the residues of petroleum distillation. Bitumen may be viscous to hard at room temperature, and have relatively low viscosity (15 to 100 cP) when hot (say 200 degrees C plus). They are used in particularly challenging water-stopping applications (Bruce et al., 2001), remain stable with time, and have good chemical resistance.

Contemporary optimization principles require simultaneous penetration of the bitumen mass by stable particulate grouts to ensure good long-term performance. Although the concept is decades old, it is only in the last five years that the process has been completely “reinvented” to provide a tool of extraordinary value.

4.3 Methods

4.3.1 Drilling

There are three generic methods of rock drilling which have been used routinely in rock drilling (the rotasonic method has not yet met wide application for grouting):

- High Rotation Speed/Low Torque Rotary: relatively light drill rigs can be used to extract core samples, when using a core barrel system, or can also be used simply to drill holes, using “blind” or “plug” diamond impregnated bits. Typically used for holes up to 100 mm diameter.
- Low Rotational Speed/High Torque Rotary: used with heavier and more powerful rigs to drill holes of greater diameter to considerable depths. The penetration rate also depends on the thrust applied to the bit. Uses a variety of drag, roller, or finger bits depending on the rock, and relates closely to water well or oil field drilling technology.
- Rotary Percussive: the drill bit (cross- or button-) is both percussed and rotated. In general the percussive energy is the determinant of penetration rate. There are two different options:
 - Top drive: where the drill rods are rotated and percussed by the drill head on the rig.
 - Down-the-hole hammer: where the (larger diameter) drill rods are only rotated by the drill head, and compressed air is fed down the rods to activate the percussive hammer mounted directly above the bit.

In principle, the prime controls over choice of drilling method should ideally be related to the geology, the hole depth, and diameter, bearing in mind always the question of lineal cost. Hole linearity and drill access restraints may also have significant impact.

Overall in the United States, rock drilling is largely and traditionally conducted by rotary methods although the insistence on diamond drilling is no longer so prevalent. Top drive rotary percussion is growing in acceptance in certain quarters - with the increasing availability of higher powered diesel hydraulic drill rigs - as long as water or foam flush is used. Holes up to 100 mm in diameter to depths of 50m can be drilled economically. Somewhat perversely, certain specialists are beginning to allow air flushed rotary-percussive drilling for routine grout holes. Even when the air is “misted” with some inducted water, most specialists agree that this medium has a detrimental effect on the ability of the fissures to subsequently accept grout (Houlsby, 1990; Weaver, 1991). Such methods are still, of course, wholly applicable for drilling grout holes to locate and fill large voids such as karstic features. It is common to have drilling rigs instrumented to provide real time accurate data on those drilling parameters which in some way reflect directly the geology and ground water conditions.

4.3.2 Grouting

Rock grouting practice largely follows traditional lines (Ewert, 1985), although it would appear that more recent publications by specialists such as Houlsby (1990) and Weaver (1991) have had a refreshing and stimulating impact. There are three basic methods used for grouting stable rock masses:

- Downstage (Descending stage) with top hole packer;
- Downstage with down hole packer; and
- Upstage (Ascending stage).

Circuit grouting is now only very infrequently used.

The competent rock available on most dam sites is well suited for upstage grouting and this has historically been the most common method. Downstage methods have recently had more demand in the U.S. reflecting the challenges and difficulties posed more difficult site and geological conditions in the remedial and hazardous waste markets.

In some cases of extremely weathered and/or collapsing ground conditions, even descending stage methods can prove impractical, and the MPSP (Multiple Packer Sleeve Pipe) Method is now the method of choice. (Bruce and Gallavresi, 1988). This has particular application in remedial rock grouting operations.

The MPSP system is similar to the sleeved tube (tube à manchette) principle in common use for grouting soils and the softest rocks. The sleeve grout in the conventional system is replaced by concentric polypropylene fabric collars, slipped around sleeve ports at specific points along the tube (Figure 3). After placing the tube in the hole, the collars are inflated with cement grout, via a double packer and so the grout pipe is centered in the hole, and divides the hole into stages. Each stage can then be grouted with whatever material is judged appropriate, through the intermediate sleeved ports. Considerable use has been made of MPSP in loose, incompetent, or voided rock masses, especially karstic limestones in recent projects involving the authors in the Philippines, Canada, and the U.S. Such systems permit the use of a wide range of grouting materials, including the hot melts.

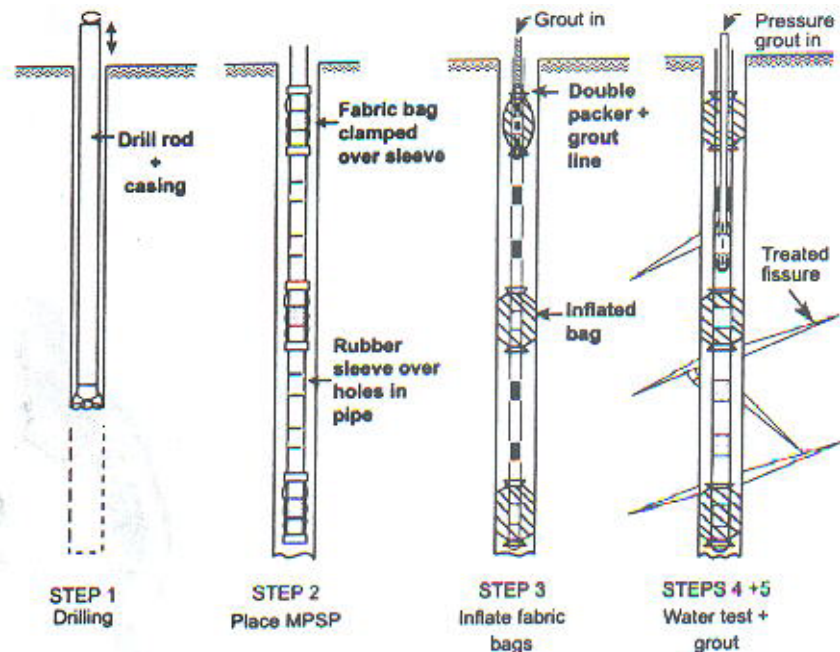


Figure 3. Multiple packer sleeve pipe system (Bruce and Gallavresi, 1988).

Regarding equipment, contemporary practice features the use of highly automated grout preparation and pumping stations. Mixers are high speed, high shear, high output and are capable of batching wide ranges of multicomponent particulate grouts with accuracy and consistency. Electricity is the power source of choice. Pumps must be capable of infinite stepping of injection rate and volume within their operating parameters and are usually electrically and/or hydraulically powered. Higher pressure operations (say above 2 MPa), require piston pumps, while progressive cavity pumps remain common for low pressure work. Other families of grouts require their own batching and delivery systems, usually provided by, or in conjunction with, the materials suppliers.

4.4 QA/QC and Verification

4.4.1 General

The fundamental approach to a correctly engineered grout curtain remains

- Investigate site and determine causes/paths of leakage;
- Execute grouting program; and
- Verify performance.

The traditional tools for investigation and verification such as coring, permeability testing, ground water characterization, dye testing, piezometric levels, and outflow monitoring have been supplemented by a range of geophysical tests in certain applications, and by sophisticated data collection, analysis and presentation instrumentation.

However, it is in the qa/qc programs now exercised during the execution of grouting works that the most significant progress is being made. As reported by Wilson and Dreese (1998), the potential of electronic measurement devices mated with computers was recognized almost as soon as widespread use of computers came into being in the early 1980s. The first trials were conducted at Ridgeway Dam by the U.S. Bureau of Reclamation (USBR). The problems with the first system were numerous, but it led to the USBR embarking on development of a comprehensive hardware and software system that would provide, generate, and record all the information that was needed for monitoring, control and analysis of grouting (Demming et al., 1985). That system was written in Basic programming language by a USBR software subcontractor, who retained the proprietary rights to the software, and the USBR implemented its use at Stillwater Dam in 1985. Since that time, there have been dramatic improvements in both the number and type of electronic measurement devices, computers and data management software.

At the simplest level, readings from flow meters and pressure transducers are transmitted to an X-Y recorder and manual calculations are then conducted. However, potentially significant head losses and gains from the system and the environment are ignored. The manual manipulation can be erroneous and is usually cumbersome when head difference allowances must be made. The next level allows for computer display of readings and spreadsheet calculations. Although head losses and gains are more easily accounted for, data entry from display to spreadsheet is still required.

The highest level is represented by some type of computer aided grouting and engineering system. The displayed data are automatically adjusted for all necessary correction factors to

reflect actual parameters within the stage being grouted. The displayed data include real time plots of the pressure and flow values, and a time plot showing Apparent Lugeon value. This is a calculated Lugeon value adjusted for the viscosity of the grout and which allows evaluation of the geologic formation response during grouting. The software also generates final hole records comprised of actual and adjusted measurements and scaled time plots of all parameters throughout the entire grouting operation. A final level of sophistication, which is not in general use, includes remotely activated control valves to allow adjustment of flows and pressures during grouting. Computer assisted grouting provides the knowledgeable grouting practitioner with real time data acquisition and a sound, scientific basis for decision making. As a consequence, every stage in every hole can be correctly brought to a natural refusal by informed manipulation of grout pressure, injection rate, rheology and grain size.

For example, such innovative methods and materials were used as the second part of the grout curtain at Penn Forest Dam, PA (Wilson and Dreese, 1998). The advantages recorded by these authors relative to “conventional” methodologies included:

- Real time data are obtained at much smaller time intervals (5 to 15 sec. frequency vs. 5 to 15 min. frequency).
- Eliminates potential for missing critical events such as pressure spikes.
- Data obtained are more accurate.
- Higher grouting pressures can be used with confidence.
- Formation response to procedure changes (mix or pressure) is shown instantly.
- Damage to formation due to over-pressuring can be easily detected and mitigated.
- Significant acceleration of pressure testing and grouting operations.
- More consistent grouting procedures due to central control location.
- Reduction in inspection manpower requirements.
- Provides detailed, permanent graphic records showing the entire time history for each operation on each stage.

The authors also found that the advanced system required less grout to reach the target permeability, largely as a result of the enhanced penetrability of these stable grouts. Financially, the construction cost savings were about 10%, the inspection cost savings 25%; and the construction schedule savings 25%, relative to those incurred during the previous, traditional grouting phase.

5. *Deep Mixing Method (DMM)*

5.1 Background

Deep Mixing Methods (DMM) have been used in the United States since 1986 but came to national prominence only from the mid-1990s as a result of the major applications in Boston, MA, in association with the Central Artery Project.

The Federal Highway Administration (FHWA) commissioned a global state of practice review from the author following the Tokyo Conference on Deep Mixing and Jet Grouting in May, 1996. that conference may be judged by our profession in retrospect to be one of the more significant expressions of technical knowledge on a narrow range of subjects to have impacted current and future U.S. specialty geotechnical construction practice. Not only were the historical

leaders of technology from Japan and Scandinavia present, but there was a significant proportion of attendees from North America and Europe to ensure that the rich volume of data openly presented would have a global impact in specialty geotechnical engineering circles. For the first time, these specialists communicated freely and openly in the English language about retrospective, introspective and prospective aspects of the industry. This was particularly welcome from the Japanese and Scandinavian practitioners, whose fundamental and excellent research and development findings had hitherto been available largely in their native language. For example, Terashi (1997) reported over 200 technical papers on Deep Mixing (DMM) were published each year in the Japanese language alone, so rendering their contents beyond the scope of occidental readers.

In the United States, there is a rapidly growing demand in a variety of markets for the benefits that DMM can provide. Mass Ground treatment schemes in Boston, MA for tunneling can be compared with earth retention projects in Milwaukee, WI, seismic retrofits for dams in the Rockies, and deep foundation systems in the San Francisco Bay area with respect to variety, intensity and technological ingenuity.

The FHWA study has so far been issued in three volumes (FHWA, 2000a, 2000b, and 2001); and forms the basis for this section on DMM. Since the late 1990s, the Deep Foundation Institute has sponsored a “Soil Mixing” Committee, and regularly ADSC seminars review Deep Mixing. Since early 2001, a State Pooled Fund Initiative on Deep Mixing has been developing coordinated by FHWA, and increasing number of technical papers are appearing in the technical press. In 2002, the Grouting and Ground Improvement Committees of the Geo-Institute combined to propose a Task Force on Deep Mixing – the first major step towards full Committee status.

5.2 Scope and Definition

DMM may be defined as an in situ soil treatment technology whereby the soil is blended with cementitious and/or other materials, either in dry or wet (slurry grout) form. The greatest amount of the work conducted globally involves vertical penetration by one or a number of mixing shafts to create discrete columns or panels. Depending on the application, these elements may be constructed to overlap to provide a variety of geometries of treat soil. The FHWA study addresses only these vertical, rotary methods.

However, there are an increasing number of methods under development which create either mass treatment by using inclined auger or conveyor technology or by using vertical beams with lateral jetting capabilities to provide thin, but continuous in situ membranes. Such applications mainly serve the environmental market – containment fixation, and retention, respectively – an are typically viable to relatively shallow depths (10m). Nevertheless, future studies of DMM should entertain these methods alongside our conventional groups of methodologies.

5.3 Applications

The main groups of applications remain:

- Hydraulic cutoff walls;
- Excavation support walls;

- Ground treatment;
- Liquefaction mitigation;
- In situ reinforcement, piles and gravity walls;
- Environmental remediation.

Globally, the novelty now arises when local methods are used for new applications, or when established methods are used in new geographic areas, often by contractors who are seeking to develop their own variant of the method in response to a particular project's challenges. Thus, we may anticipate in the next decade's technical press a plethora of case histories dealing with environmental and liquefaction mitigation, and in situ earth reinforcement from practitioners in countries as diverse as the U.K., Indonesia, and Australia, based on the authors' current project awareness.

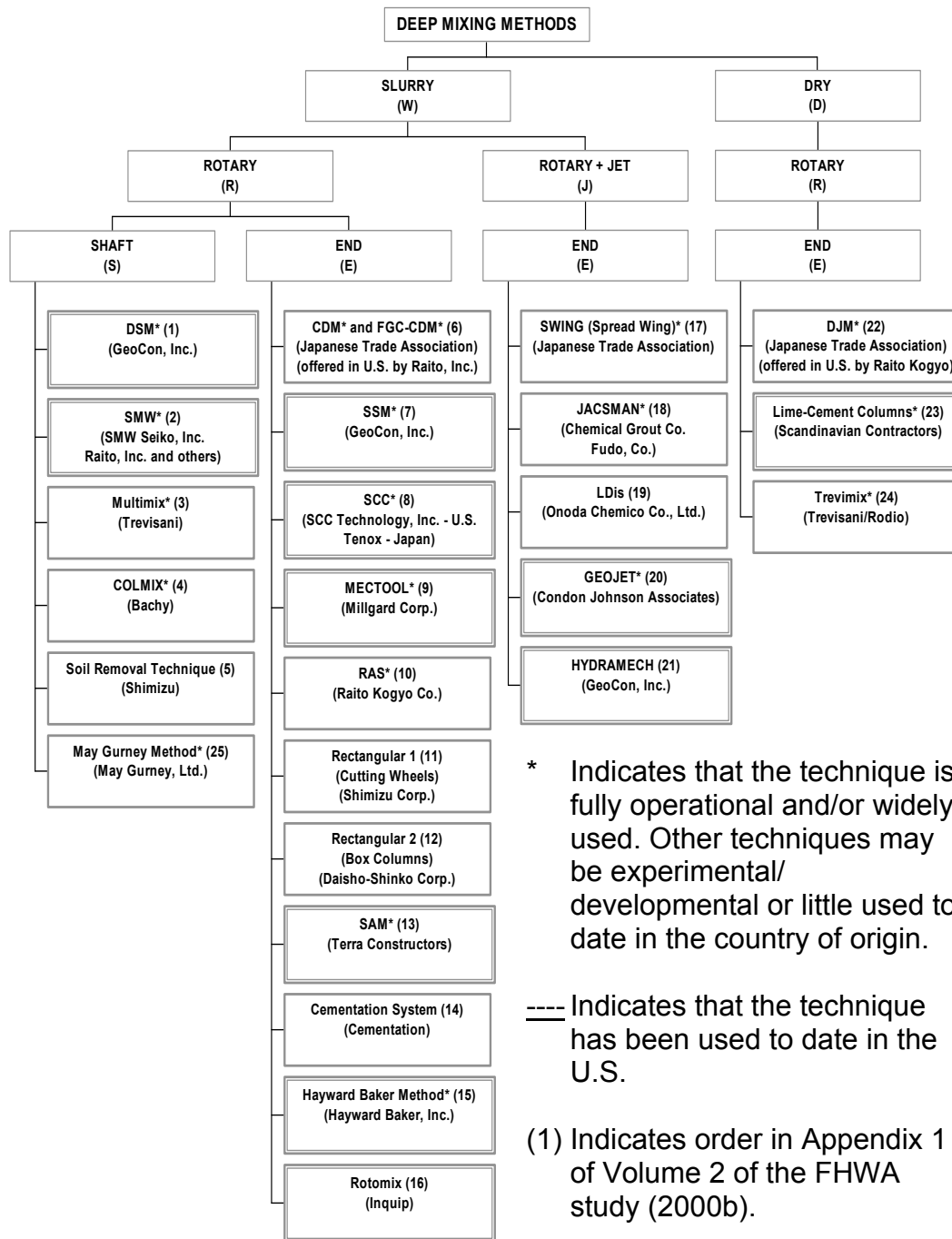
The viability, both technically and commercially of DMM in its various potential applications and settings will continue to be challenged by solutions based on other technologies and cultural preferences, and rightly so: deep mixing is not the panacea for all specialty geotechnical problems. However, when the goal is ground treatment, improvement or retention, the ground and site are relatively unobstructed, and the depth is limited to about 40m, then deep mixing will most probably be a viable option in countries with easy commercial access to the technology.

5.4 Classification of Methods

A total of 24 different methods – mostly fully operational and patented – were identified by the FHWA survey (Figure 4). The classification adopted is based on the nature of the “binder” (grout, or dry); the method of soil blending (rotary alone, or rotary with jet assistance); and the location at which most of the soil/binder blending occurs (along the shaft of a long auger, or only at the mixing tool located at the end of the rod). This classification, of course, only applies to those deep mixing systems employing vertical mixing principles (as discussed above). A new “arm” to this classification will be necessary to accommodate the “mass,” or “lateral jetting” variants.

The authors have received peer reviews of this proposed classification from specialists worldwide, and have monitored global practice for three years to date. The generic classification of Figure 4 has, in patent terms, “satisfied” these challenges, and so is considered appropriate.

Regarding the future, the constructional developmental trends are towards improving the quality of the mixing process (e.g., Systems 11 and 12); using less expensive binder components (e.g., System 6-FGC); obtaining larger diameter of treatment via jet assistance (e.g., Systems 18 and 21); and improving the level of computer assisted control (most systems, but especially in the U.S., Systems 3, 20, 23 and 24).



* Indicates that the technique is fully operational and/or widely used. Other techniques may be experimental/developmental or little used to date in the country of origin.

---- Indicates that the technique has been used to date in the U.S.

(1) Indicates order in Appendix 1 of Volume 2 of the FHWA study (2000b).

Figure 4. Classification of Deep Mixing Methods based on “binder” (Wet/Dry); penetration/mixing principle (Rotary/Jet); and location of mixing action (Shaft/End) (FHWA, 2000b).

5.5 Verification Methods for Treated Ground

The properties of treated ground are predicted and/or verified by the following broad groups of tests:

- Laboratory testing of laboratory samples (before construction).
- Wet grab sampling of fluid in situ material (during construction).
- Coring of hardened in situ material (after construction).
- Exposure and cutting of block samples (after construction).
- Miscellaneous, including geophysical testing (during and after construction).

It is reiterated that these properties are influenced in detail by many interactive factors, including soil type, amount and type of binder, water cement ratio, degree of mixing, curing conditions, environment, and age, although the soil characteristics themselves seem to be the most sensitive determinant of variations in strength. Excellent recent overviews of Scandinavian practice have been provided by Halkola (1999); Axelsson and Rehnman, 1999; and Holm et al., 1999).

5.5.1 Laboratory Testing

Such testing is a valuable basis for confirming basic design assumptions, and for demonstrating the effect and impact of the various materials used (both artificial and natural). It is also clearly useful in establishing base-line parameters, and for investigating in a controlled fashion the relationships between the various strength parameters and construction variables (Table 4). With respect to temperature, this is related to the size of the treated soil mass, as well as the quantity of binder introduced. In laboratory testing, there is no way to reliably vary and simulate factors III and IV from Table 4, except for the amount of binder and the curing time. Laboratory testing therefore standardizes these factors, with the result that the strength data obtained during such tests are “not a precise prediction” but only an “index” of the actual strength. Likely field strengths can then be estimated using empirical relationships from previous projects, and exercising engineering judgment. However, there is as yet no standard laboratory test procedure (in Japan).

Kamon (1996) summarized that unconfined compressive strength data from field cored samples are 20 to 50% those prepared in the laboratory findings largely supported by Kawasaki (1996). These data were determined from land projects whereas on the massive marine CDM projects, larger field than laboratory values are often obtained due to the “adiabatic temperature rise” in in-situ treated masses.

Mizutani et al. (1996) found that core strengths were 60 to 70% those of laboratory mixed samples and that 60 to 80% of the lab strength can be achieved in the field with “fairly good” quality control.

Taki and Bell (1998) also found a reduction in apparent strengths from laboratory to field, with a wider data scatter in the field data (Figure 5).

Table 4. Factors affecting the strength increase of treated soil (Terashi, 1997)

I	Characteristics of hardening agent	<ol style="list-style-type: none"> 1. Type of hardening agent 2. Quality 3. Mixing water and additives
II	Characteristics and conditions of soil (especially important for clays)	<ol style="list-style-type: none"> 1. Physical chemical and mineralogical properties of soil 2. Organic content 3. pH of pore water 4. Water content
III	Mixing conditions	<ol style="list-style-type: none"> 1. Degree of mixing 2. Timing of mixing/re-mixing 3. Quality of hardening agent
IV	Curing conditions	<ol style="list-style-type: none"> 1. Temperature 2. Curing time 3. Humidity 4. Wetting and drying/freezing and thawing, etc.

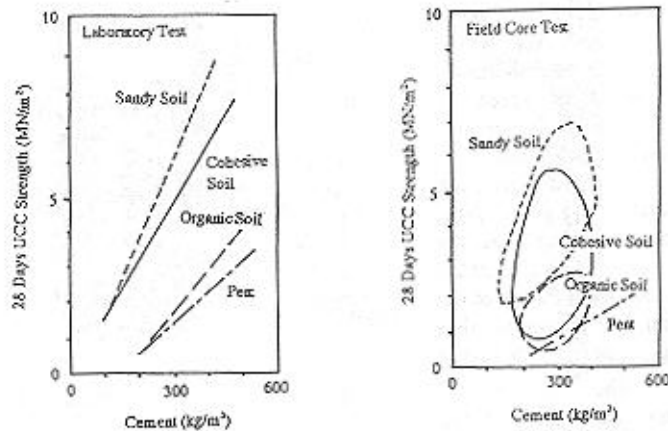


Figure 5. Relationship of unconfined compressive strength, cement factor and soil type (data developed with water/cement ratio of 0.6 to 1.2, and a volume ratio of 23 to 75%) (Taki and Bell, 1998, after Hibino, 1989).

5.5.2 Wet Grab Sampling

The concept simply is to obtain samples from the treated ground before the mix reaches such a strength that a sampler cannot be introduced easily or without causing significant sample disturbance. Such samples are then used to make cubes or cylinders for later laboratory testing. Wet grab sampling may be faced with a number of systematic and logistical problems. For example, the sampling device must be able to reach a prescribed depth, take a representative sample from that depth, and allow it to be retrieved without contamination. This places great emphasis on the efficiency of the sampling tool and how expedient it is to introduce and withdraw.

If the deep mixing efficiency has not been high, the presence of unmixed native material may prevent the sampler from functioning correctly, and/or from obtaining a wet sample whose composition is truly representative of the overall mixed volume. In this regard, it is typical to screen wet samples, prior to casting samples for testing, and screens with 6- to 12-mm aperture are common.

5.5.3 Coring

Given that coring is an energetic, local and invasive technique, even when conducted with the best equipment, skill, and methods (the triple tube core barrel is widely recommended), it is notable that most contractors cite core samples as their prime source of data on treated ground properties in general, and unconfined compressive strength in particular.

Druss (1998) noted several key elements which promote good and representative core sampling. These include using experienced drillers and logging engineers; taking large diameter cores (greater than 76 mm in diameter); using triple tube methods, coarse diamond bits to minimize sample washout, and appropriate drilling flush; and ensuring that the inside surface of the sample tube is well lubricated. The Japanese Committee on DM strength evaluation (Hosoya et al., 1996) recommends a minimum core diameter of 150 mm.

Taki and Bell (1998) wrote that core locations can be randomly selected, but additional core drilling and testing should be performed when questionable soil conditions or mix conditions are observed during installation. Uniformity of mixing can be evaluated from the inspection of core samples. The depths of core samples should include intervals containing the weakest soil layers and should be more than 95% in sandy soil and more than 90% in cohesive soil. There is healthy debate as to the relationship of unconfined compressive strengths measured from cores, and those cast from wet grab samples. It is perceived in some quarters that core samples will provide lower strengths given the distress caused to the core during drilling and extraction. (However, it is typical to select only the “better” core samples for testing and this will automatically provide higher actual test data). Taki and Yang (1991) produced data (Figure 6) from various soil types which show that the core strengths were about twice those obtained by samples made from wet grabs. Their view is supported by Burke (1998), who found that on a DMM project in soft clays the core samples always gave higher strengths than wet grab samples, but were only 50% of laboratory strengths and showed a wider variation. Although pre- and post-construction CPTs are possible for in situ strengths less than 7 MPa, if in situ strength is expected to average over 3.5 MPa, Burke considers coring feasible (minimum diameter 76 mm). Recovery rates can vary from 25 to 100% depending on mixing parameters and soil strengths. They found that, with their particular DMM technique (Method 15), the grout:treated soil strength ratio was about 4.

Taki and Yang (1991) published data illustrating the relationship of unconfined compressive strength, cement factor, and soil type, (Figure 6), which highlighted also the difference in value and quality between laboratory and core data.

Isobe et al. (1996) conducted several field tests using a WRE method in sands and silts at Kunishima, Japan. Cores were taken at various positions across the face of different sized columns. Strengths were higher in centrally cored samples, but decreased by up to 50% toward the perimeters of the DMM columns.

Okumura (1996) stated that for large DMM projects in Japan, it is typical to core one hole per every 10,000 m³ of treated soil (marine projects) and 1 per 3,000 m³ (land projects).

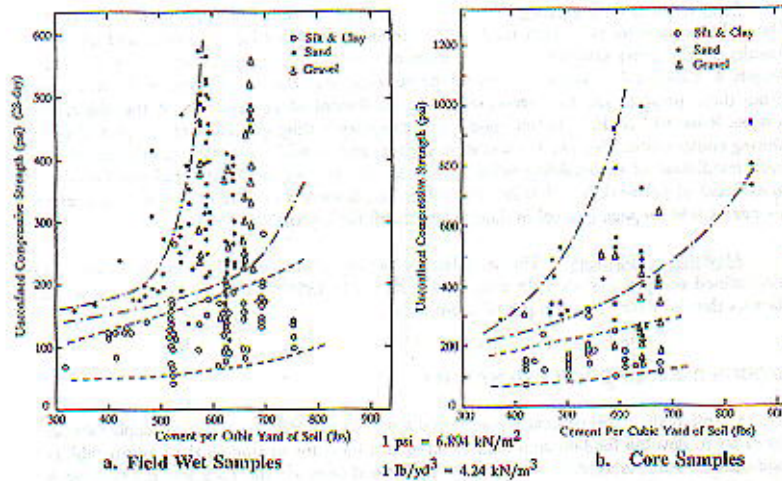


Figure 6. Strength of soils treated by SMW Method (Taki and Yang, 1991).

Regarding future developments, Sugawara et al. (1996) produced a most interesting paper on their attempts to produce an improved core sampler. However, even this can only reduce disturbance to the core samples, although the use of triaxial as opposed to uniaxial testing will generally give higher and more consistent results.

5.5.4 Exposure and Block Sampling

The opportunity to expose the treated ground allows all parties to observe column shape, homogeneity, diameter, nature of overlap and so on. It also permits samples to be taken with different shapes, sizes and orientations from those that can be obtained by vertical coring. The value of this kind of testing is underlined when it is recalled that important technical goals of any DMM operation are to provide a uniformly treated mass, with minimal lumps of soil or binder, a uniform moisture content, and a uniform distribution of binder throughout the mass. Exposed treated soil can be sprayed with phenolphthalein solution to indicate the presence of cement in the mass.

Single columns can be fully exposed, and even extracted (e.g., Method 23), while multiple columns can be installed in a circular shaft, or box, arrangement to allow a self-supporting excavation to be completed.

Again, the major drawbacks to such exercises are principally cost, time, and site logistics, but on certain projects of critical size, complexity and significance, exposure is a vital element in verification, both as a pre-production measure, but also as a special demonstration during construction. Burke (1998) is of the opinion that the most efficient method of evaluation is to drill a shaft into overlapping columns to allow visual observation of integrity, homogeneity and sampling of the mixed soils, and therefore to put into perspective any apparent anomalies identified by coring.

5.6 Miscellaneous

Depending on the nature, purpose, and extent of the treatment, a variety of miscellaneous methods have been reported. For example, Methods 11 and 19 both have been developed to reduce adjacent ground and structural movements: inclinometer and borehole extensometer testing results have therefore been reported. Similarly, in those methods (e.g., 15 and 23) focusing on very soft clays and low strength treatment, CPT/SPT testing may be conducted before and after mixing although the Japanese Committee (1996) regards SPT values as “coarse” but of “some merits”. Most recently, Esrig (1999) described the value of routine pressuremeter testing to indicate in situ shear strengths. In higher strength materials (e.g., Method 4), “sonic velocity measurements in three dimensions” have been conducted to verify quality of treatment.

Hane and Saito (1996) reported on the use of shear wave seismic tomography to explore a treated sandy soil mass, underwater. In this case, the increase in velocity was from 200 to 500 m/s (untreated) to 950 to 1200 m/s (treated), with a very small velocity contrast indicating homogeneity of treatment. These data were consistent “with other mechanical tests”. Similarly, Hiraide et al. (1996) were able to relate shear wave data to unconfined compressive strength and E value.

Regarding other geophysical techniques, Imamura et al. (1996) investigated borehole resistivity for treated soil quality, and Nishikawa et al. (1996) experimented – also successfully – with PS logging and SPT testing for predicting unconfined compressive strength. Tamura et al. (1996) reported on low strain sonic integrity testing, while Barker et al. (1996) illustrated the value of a portable cone resistance testing apparatus. Halkola (1999) noted that CPT methods are used almost exclusively “even for the testing of semi-strong and soft” columns in Helsinki. At the other end of the test scale spectrum, full-scale load testing (vertical or lateral) can be conducted on entire DMM elements (Druss, 1998).

For low strength DMM installations, such as Lime Cement Columns, a range of column vane penetrometers have been designed and tested in Scandinavia (Rathmayer, 1996; Halkola, 1999; Holm et al., 1999). Both push down and inverted versions are available. Significantly Rathmayer stated that “methods applied for integrity testing of concrete piles do not work” (for LCC). Therefore “the only reliable test method today is total sampling, managed by lifting upon the entire column”.

Experiments have also been made in Finland and in Japan with, respectively, “measurement while drilling” (MWD) or “factor of drilling energy” tests, which, according to Halkola (1999) relate the records of various drilling parameters to the strength properties of the treated soil. A useful summary of these various methods was provided by Hosoya et al. (1996) (Table 5).

5.7 The Future of DMM in the United States

Notwithstanding the benefits and advantages which contemporary DMM techniques can offer, there remain a number of factors, often interrelated, which act as potential barriers to market entry for prospective contractors, and/or controls over market growth. These include:

- Demand for the product: given the national trends towards urban construction and redevelopment, seismic retrofit and environmental clear up – all challenges to be solved in situ – then demand for DMM will continue to increase.

Table 5. In-situ tests for evaluating treated soil (Hosoya et al., 1996).

Type of test	Method	Test method and results	Comment on quality control method
Sounding test	Standard penetration test	Let 67.5kgf hammer fall free from 75cm height and count the number of strokes (N-value) to penetrate 30 cm.	Most common method on natural soil. However, only a few applications for stabilized soil are available. There is correlation with unconfined compressive strength.
	Dynamic cone penetration test	Let 5kgf hammer fall from 50cm height and count the number of strokes (Nd-value) for cone to penetrate 10cm.	Easy transportation and operation. Practical for unconfined compressive strength of $q_u=2\sim 5$ kgf/cm ² . $q_u=0.29Nd-2.58$ (kgf/cm ²)
	Electric static cone penetration test	Let the cone penetrate at uniform speed and measure the resistance at the end and surrounding surface and pore-water pressure in sequence.	Applicable to measure the improvement of low strength stabilized soil in sequence. However, not applicable for firm stabilized soil.
	Rotary penetrating test	Measure the bit pressure, torque, and muddy water pressure by the sensor at the end of the boring rod to observe the soil strength in sequence.	Greater mobility compared with core sampling and in-situ strength can be measured. However, correlation with the unconfined compressive strength must be compared from site to site.
Test of utilizing bore-hole	PS logging	Measure the velocity of P and S waves. Then calculate rigidity and Poisson's ratio of stabilized soil. There are two testing methods. Down hole method and Suspension method.	There is some correlation with unconfined compressive strength although it is not so uniform. Suspension method is better to evaluate the stabilized soil.
	Electrical logging	Supply electricity to stabilized soil and measure electric current and voltage through an electrode. Then calculate the specific resistance. Lately, analysis is made by tomography in some cases.	The correlation with unconfined compressive strength is low.
	Density log	Measure the gamma rays emitted from a probe inserted into the hole by the detector installed at a certain distance. Then convert the data into density.	Since it is influenced by hole diameter and water inside the hole, calibration is important. There is no correlation with unconfined compressive strength.
Loading test	Bore-hole lateral load test	Press rubber tube toward the bore-hole wall in stages and measure the strength and deformation modulus of stabilized soil. Measurement apparatus is Pressiometre and LLT.	Deformation modulus rather than strength is often the objective of the tests. Vertical measurement is costly so it is used as representative value of stabilized soil.
	Plate loading test	Place a loading plate (round plate of 30cm diameter) on the stabilized soil and put on load in stages. Bearing capacity and deformation characteristics can be obtained directly from the load and settlement curves.	Bearing capacity and deformation characteristics can be obtained directly. However, the evaluation of stabilized soil is possible only down to the depth of 2 to 3 times of (load) plate diameter.
	Stabilized pile loading test	Place the (load) plate of the same diameter as the stabilized pile on the surface of the leveled stabilized pile. Then put on load in stages. This is a sort of vertical load test of a pile where bearing capacity characteristics are obtained from the load and settlement curves.	Bearing capacity characteristics of a stabilized pile can be directly obtained. However, testing equipment is costly and the number of tests available is limited.
Non-destructive test	Integrity test	Safely stroke the surface of a stabilized pile with a hammer and measure the reflected wave of the vibration by the accelerometer installed on top of the pile. Length and discontinuity of the stabilized pile is measured.	Simple method. However, evaluation standard for a stabilized pile has not been established yet.
	Elastic wave exploration	Emit P and S waves to measure the velocity distribution of stabilized soil. In the case of stabilized soil, measurement of S wave is preferred.	Stabilized condition is measured by velocity distribution of the S wave. The measurement is made in the bore-hole and on the ground surface. Tomography is used to improve accuracy of the test.
Other test	Penetration test	Use pocket type pin penetration apparatus and measure the penetration resistance of stabilized soil on the job site. Then estimate the unconfined compressive strength.	Easy and simple method. A lot of tests can be done. However, only the surface of the stabilized soil can be tested.

- Awareness of the product: a wider range of active specialty contractors and consultants, more prolific technical publications, short courses and the coincidence of several high profile DMM projects nationwide have combined to elevate awareness of DMM in general engineering circles, and will so continue to increase demand.

- Bidding methods/responsibility for performance: the authors believe that the interests of a rapidly developing and complex technology like DMM in the U.S. are best served by “design-build” concepts. Thus, the rate of growth of DMM will be influenced strongly by the rate at which innovative contract procurement and administration vehicles are adopted nationwide.
- Geotechnical limitations: DMM has been developed to treat relatively soft, unobstructed soils and fills in sites with good access. There are other practical limitations as to depth, strength, and durability of the treatment. Extreme care should be taken not to overextend the limits of DMM capability without due regard to a true appreciation of the fundamentals of its evolution. Otherwise, inappropriately applied, designed, and constructed work will lead to owner disappointment, or worse.
- Technology protection: most of the 24 methods shown on Figure 4 are protected in their technology by Patent or similar. Thus, new potential contractors must either invent their own system, or acquire a foreign license. The latter seems more realistic, given the timetables and costs involved in conducting basic research and development.
- Capital cost of startup: given the high levels of technical sophistication, and large physical scale of most systems, startup costs are high. In addition, the larger projects may require several machines and so committed capital expenditures may easily rise to several million dollars. The equipment must also be regularly maintained and upgraded leading to the general conclusion that DMM is a “cash hungry” technology for the contractors who offer it – although the potential return on investment is high. Thus, the field of potential contractors is practically limited by the levels of their own financial resources.

6. *Final Remarks*

In the four technologies described in this paper, there have been major technological advances within the last 10 years. Given the increasing market demands, and the matching interest shown by practitioners and owners alike, there is every evidence that this same rate of growth will continue to be observed in the 10 years to come. It is intended that this paper will constitute a “base line” of knowledge at this hectic time.

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