Recent Developments in Specialty Geotechnical Construction Processes

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ABSTRACT: There are a large number of specialty geotechnical construction techniques which are used in ground treatment, ground improvement, and ground reinforcement and retention. This review paper highlights newer developments in two areas of ground treatment, namely rock grouting, and deep mixing, and in two areas of ground reinforcement and retention, namely micropiles and anchors.

1 BACKGROUND

Technical Committee 17 (TC-17) of the International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE), of which the author was the inaugural secretary, defines the group of techniques which constitute the geotechnical engineering market as follows:

1. Ground Treatment
   - 1.1 Rock Grouting
     - 1.1.1 Fissure
     - 1.1.2 Bulk/Large Voids (Natural and Artificial)
   - 1.2 Soil Grouting
     - 1.2.1 Permeation
     - 1.2.2 Compaction
     - 1.2.3 Claquage/Hydrofracture (including Compensation)
     - 1.2.4 Jet
   - 1.3 Deep Mixing
     - 1.3.1 Wet (slurry) methods
     - 1.3.2 Dry methods
2. Ground Improvement
   - 2.1 Mechanical Densification/Consolidation
     - 2.1.1 Dynamic Consolidation
     - 2.1.2 Vibro techniques
     - 2.1.3 Preloading
     - 2.1.4 Blasting
   - 2.2. Drainage/Dewatering
     - 2.2.1 Wicks/Band Drains
     - 2.2.2 Sand Drains
     - 2.2.3 "RODREN"
   - 2.3 Thermal
     - 2.3.1 Freezing
     - 2.3.2 Vitrification
3. In Situ Reinforcement and Ground Retention
   - 3.1 Anchors
   - 3.2 Micropiles
     - 3.2.1 Structural Support
     - 3.2.2 In Situ Reinforcement
   - 3.3 Soil Nails
   - 3.4 Diaphragm Walls (Slurry Walls)

The annual total construction value of this market in the United States is around $1 billion, whereas the corresponding value for the piling market (including driven piles, continuous flight auger piles, and large-diameter drilled shafts, or caissons) is 70 to 100 percent larger.

These specialty geotechnical construction techniques find a wide range of applications within the broad market segments of construction, mining, quarrying, and environmental remediation. Depending on geological, logistical, financial and indeed historical factors, the techniques however have distinct geographical applicability. Thus, as examples, compaction grouting cannot be used in hard rock environments, permeation grouting is not feasible in silts and clays, deep mixing is not practical in low headroom situations, and micropiles are not typically competitive in urban areas underlain by great thicknesses of softer clays.

As evidenced by the wealth of case history data presented at conferences and in technical publications, technological advances continue to be made across the whole range of specialty construction processes. However, it may be argued that the most dramatic enhancements appear to have been made in four techniques in particular: rock grouting, deep mixing, anchors, and micropiles. It is no coincidence that from a usage point of view, these four techniques are also enjoying rapid growth and in a variety of applications internationally.

2 ROCK GROUTING

2.1 Materials

2.1.1 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 and soil 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.

2.1.2 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 (e.g., Figures 1 and 2). 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)
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 1), while the use of finer grind materials (e.g. DePaoli et al., 1992) has further enhanced penetrability efficiency (Figure 3). 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,

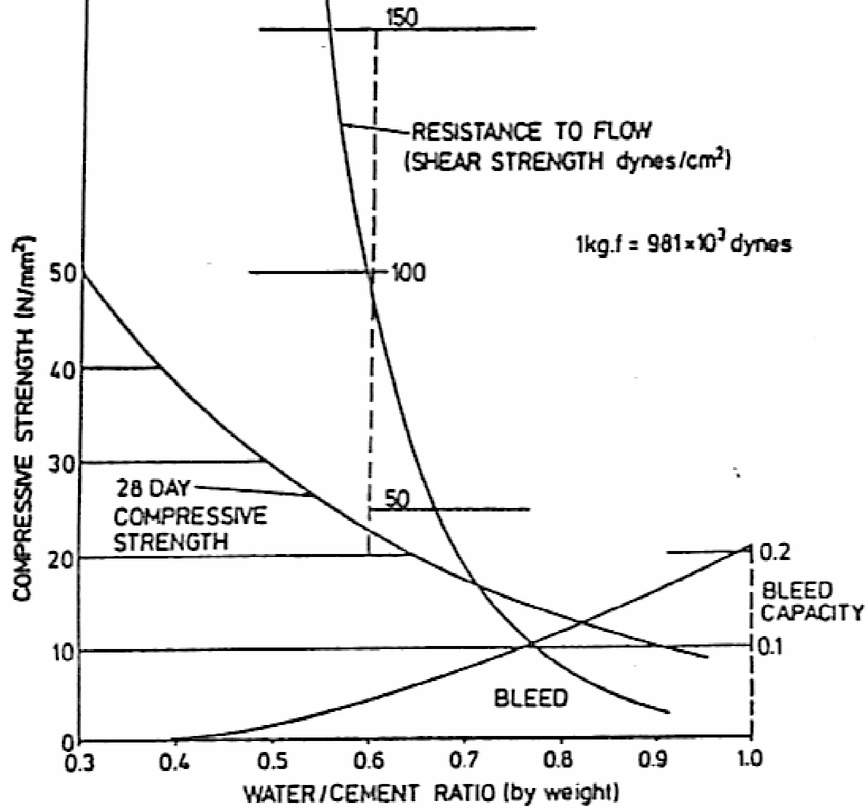

Figure 1. Effect of water content on grout properties (Littlejohn and Bruce, 1977).

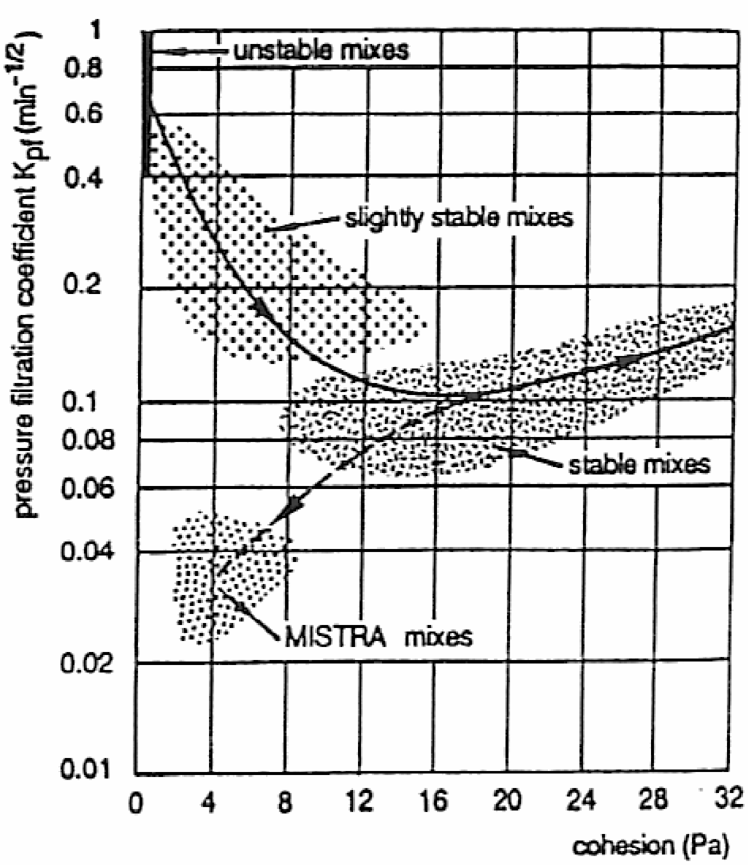

Figure 2. Relationship between stability under pressure and cohesion for different types of mixes (DePaoli et al., 1992a).

Table 1. 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. [12]
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. [12]	Increases viscosity and cohesion.	Difficult to handle due to fineness.
Viscosity Modifiers (Welan Gum)	Makes the grout suspension more water repellant, 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.
Dispersants or Water Reducers (Superplasticizer)	Overprints solid particles with a negative charge causing them to repel one another. Reduces agglomeration of particles thereby reducing grain size by inhibiting the development of macro-flocs. Also reduces viscosity and cohesion.	Depending on chemistry chosen, may accelerate or retard hydration process. This is not necessarily negative.	Dispersants have a distinct life span. Working life depends on dispersant chemistry chosen.

	grain size (µm)					
	D 95	D 85	D 60	D 50	D 15	D 10
CEMILL® 6	15.0	9.0	6.0	5.0	1.3	0.9
CEMILL® 9	9.0	5.5	3.5	2.5	0.6	0.4
CEMILL® 12	6.0	4.0	3.0	2.2	0.4	0.3
ONODA MC-500	8.0	60.0	4.5	4.0	2.5	2.0
Portland 525	40.0	22.0	11.0	8.0	2.5	2.0
bentonite	60.0	40.0	15.0	10.0	1.7	1.2

a b c sands for injection tests
a $\gamma = \gamma_{max} = 1.713$ g/cm^3
b $\gamma = \gamma_{max} = 1.701$ g/cm^3
c $\gamma = \gamma_{max} = 1.690$ g/cm^3

d bentonite
e Portland 525 cement
f ONODA MC-500 cement
g h i CEMILL® mixes

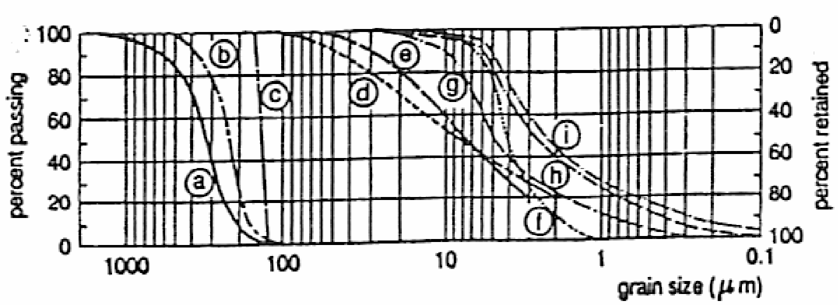

Figure 3. Grain size distribution curves for sands, dry grout materials, and grouts (DePaoli et al., 1992b).

providing stability, appropriate rheology and anti washout properties.

2.1.3 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. A thorough description of these grouts was provided by Naudts (1995).
- 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., 1998), remain stable with time, and have good chemical resistance.

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

2.2 *Methods*

2.2.1 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 100mm 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 cost. Hole linearity and drill rig 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 100mm 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.

2.2.3 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 4). 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.

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.

2.3 *QA/QC and Verification*

2.3.1 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

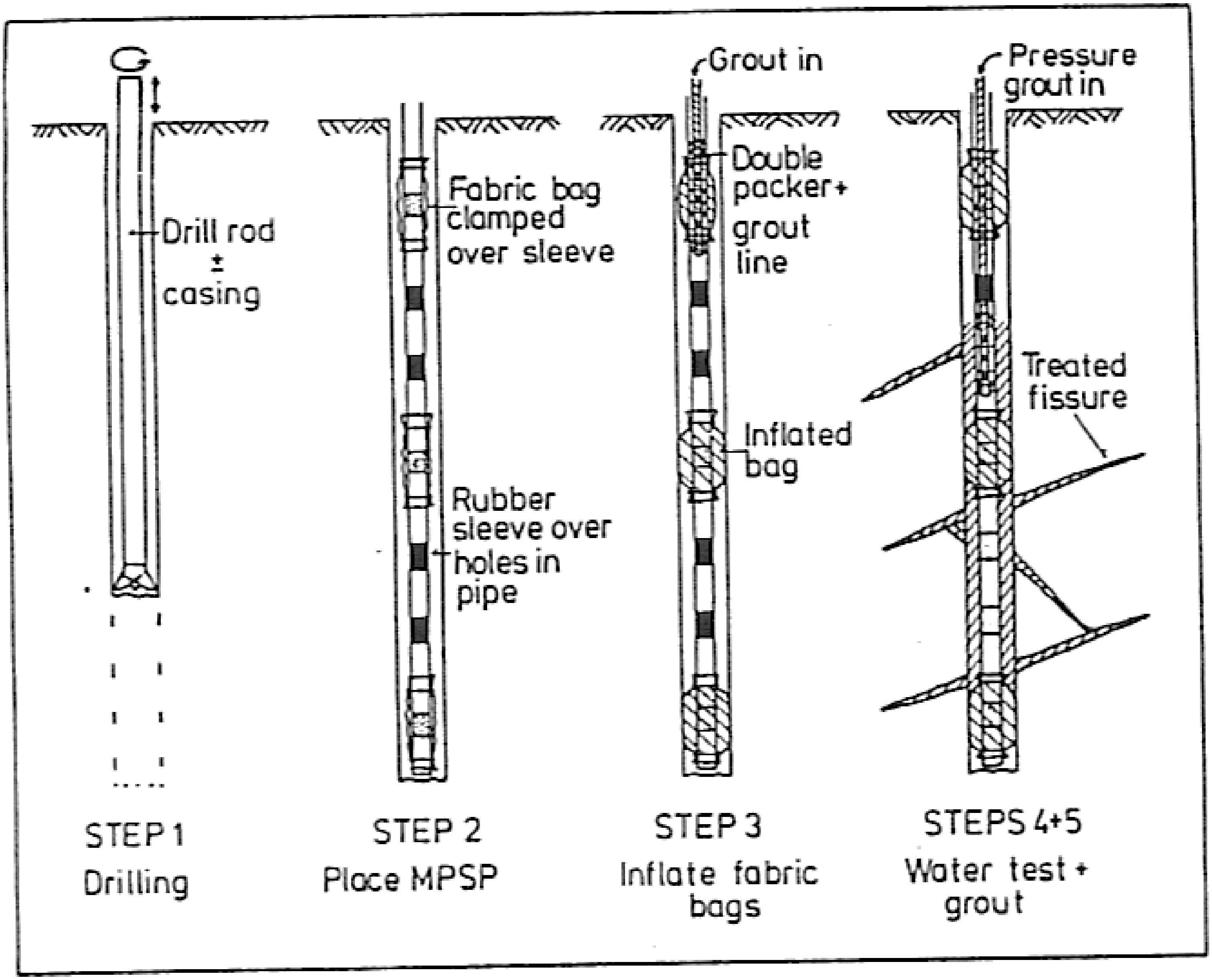

Figure 4. Multiple packer sleeve pipe system.

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 CAGES - 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 combined with the application of the Apparent Lugeon Theory and Amenability Theory (Naudts, 1995) 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.

3 DEEP MIXING

3.1 Introduction

The Deep Mixing Method (DMM) encompasses a group of technologies that provide in situ soil treatment. Materials of various types, but usually of cementitious nature, are introduced and blended into the soil through hollow, rotated shafts equipped with cutting tools, and mixing paddles or augers that extend for various distances above the tip. The materials may be injected in either slurry (wet) or dry form. The treated soil or fill mass that results generally has a higher strength, lower compressibility and (usually) lower permeability than the virgin soil, although the exact properties obtained will reflect both the characteristics of the native soil, and the construction techniques and variables that are selected.

Although the original concept appears to have been developed in the United States in 1954, current practice reflects the intense efforts of researchers, backed by strong federal resources and demand, in both Japan and Scandinavia since 1967. During the last decade, however, domestic challenges to the specialty ground engineering community in the arenas of urban infrastructure development, seismic mitigation and environmental remediation, have led to a rapid growth in the use of such techniques in the United States also.

Recent international conferences, such as in Tokyo in 1996, Logan, UT (ASCE, 1997) and Boston, MA (ASCE, 1998) have highlighted that there exist a surprisingly large number of different DMM techniques, each one typically proprietary to one, or a group of, specialty contractors. It is also clear that each technique has its own particular advantages and limitations, technically, logistically, and environmentally.

3.2 Historical Evolution

Table 2 provides a chronology of the major events in the ongoing development of the DMM techniques. It refers to a large number of these techniques by name, bearing in mind that the details of these techniques themselves are provided in Appendices 1 and 2. Table 2 highlights the commitment and energy of engineers in Scandinavia and Japan for over 30 years, initially pursuing similar paths, but soon following different directions in response to

Table 2. Highlights of Historical Development of DMM (continues).

Year	Event
1954	Intrusion Prepakt Co. (U.S.) develop the Mixed in Place (MIP) Piling Technique (single auger), and see sporadic use in the U.S., although widespread use continues in Japan till early 1970s.
1967	Port and Harbor Research Institute (PHRI, Ministry of Transportation, Japan) begin laboratory tests, using granular lime for treating soft marine soils (DLM). Research continued by Okumura, Terashi et al. through early 1970s to a) investigate lime-marine clay reaction and b) develop appropriate mixing equipment (U.C.S. of 0.1 to 1 MPa achieved) Early equipment used on first marine trial near Hareda Airport (10m below water surface).
1967	Laboratory and field researches begin on Swedish Lime Column method for treating soft clays under embankments using unslaked lime (Kjeld Paus, Linden - Alimak AB, in cooperation with Swedish Geotechnical Institute (SGI), EurocAB and ByggproduktionAB).
1974	PHRI report that the Deep Lime Mixing method (DLM) has commenced full scale application in Japan. First applications in reclaimed soft clay at Chiba. (DLM continues to be popular until late 1970s when CDM and DJM supersede it.)
1975	Following researches from 1973 to 1974, PHRI develop the forerunner of the Cement Deep Mixing method (CDM) using cement grout and employ it for the first time in large scale projects in soft marine soils offshore. (Original variants include DCM, CMC (still in use from 1974), then DCCM, DECOM, Demic, etc.)
1976	Public Works Research Institute (PWRI) (Ministry of Construction, Japan) begins researches on the Dry Jet Mixing method (DJM) using dry powdered cement (or less commonly, quick-lime); "first practical stage" completed in late 1980. Representatives of PHRI also participate.
1976	SMW method used commercially for first time in Japan.
1977	First "practical use" of CDM in Japan (marine and land uses).
1980	First commercial use in Japan of DJM (land use only).
1981	Prof. Mitchell presents General Report at ICSMFE in Stockholm on lime-cement columns.
1985	SGI (Sweden) publishes 10 year progress review. (Åhnberg and Holm).
1986	SMW Seiko Inc. commence operations in U.S. under license from Seiko Kogyo.
1987 - 1988	SMW method used in massive ground treatment and improvement program for seismic retrofit at Jackson Lake Dam, WY.

Table 2. Highlights of Historical Development of DMM (concluded).

Year	Event
1988 - 1989	Development by Geocon, Inc. in U.S. of DSM (Deep Soil Mixing) and SSM (Shallow Soil Mixing) techniques.
1989	Start of exponential growth in use of Lime Cement Columns in Sweden and Finland.
1992 - 1994	SMW method used for massive earth retention and ground treatment project at Logan Airport, Boston.
1992 - 1993	First SCC installation in U.S. (Richmond, CA).
1993	CDM and DJM Research Institutes publish Design and Construction Manuals (in Japanese).
1994	First commercial application of original Geojet system in the U.S. (Texas) following several years of development by Brown & Root.
1995	Swedish government sets up new "Swedish Deep Stabilization Research Center" at SGI (1995-2000: $8 -10 M): "Svensk Djupstabilisering". Consortium includes owners, Government, contractors, universities, consultants, research organizations, co-coordinated by Holm of SGI.
1995	Finnish government sets up new research consortium for the ongoing Road Structures Research Programme ("TPPT") - till 2001.
1995	Swedish Geotechnical Society publishes new design guide for lime and lime-cement columns (P. Carlsten).
1996	SGI (Sweden) publish 21-year experience review.
1996	First commercial use of lime columns in the U.S. (by Stabilator in Queens, NY).
1996 - 1997	Hayward Baker install 1.2 to 1.8m diameter columns for foundations, earth retention and ground improvement.
1997	SMW method used for huge soil treatment project at Fort Point Channel, Boston, MA (Largest DMM project to date in North America), and other adjacent projects. Input at design stage to U.S. consultants by Dr. Terashi (Japan).
1997	First commercial use in U.S. of modified Geojet system (by Condon Johnson Associates at San Francisco Airport, CA).
1997	Major lime-cement column application (I-15, Salt Lake City) proposed by Swedish contractor, Stabilator.
1997	Raito Kogyo (Japan) establish U.S. subsidiary in California.
1997 - 1998	Master Builders Technologies develop families of dispersants for soil (and grout) to aid DMM penetration and mixing efficiency.

particular national demands. Also apparent is the accelerating rate of progress in other regions, principally the United States and Western Europe, over the last 10 years.

3.3 Applications and Applicability of DMM

Six basic groups of applications can be identified for contemporary deep mixing methods:

1. Hydraulic cutoffs: DMM walls to prevent water movement through or under water retaining structures, such as dams or levees and into deep basements excavated below the water table.
2. Structural walls: DMM walls containing steel elements to resist lateral earth pressures in the construction of deep excavations, such as for cut and cover tunnels and deep basements.
3. Ground treatment: Block treatment to strengthen in a uniform manner large volumes of foundation soil in conjunction with deep excavations, and structural foundations.
4. Ground improvement: Discrete DMM elements (columns or panels) used as reinforcing elements to improve the overall performance of large, compressible soil masses under relatively lightly loaded structures, such as road or railway embankments.
5. Liquefaction mitigation: Interlocking DMM box or cellular structures to reduce the tendency for mass liquefaction and lateral spreading during seismic events under large embankments or buildings.
6. Hazardous materials: DMM walls to contain, or DMM block treatment to fix, environmentally unacceptable materials.

3.4 Classification of Methods

A generic classification of the numerous methods used internationally can be made on the following simple basis:

- Is the cementitious material injected in a slurry or wet (W) form, or in a dry (D) state?
- Is this binder mixed with the soil via rotary energy only (R) or is the mixing enhanced/ facilitated by high pressure jet (J) grout type methods?
- Is the mixing action only occurring near to the drilling tool (E), or is it continued along the shaft (S) for a significant distance above it, via augers and/or paddles?

The classification shown on Figure 5 has therefore been developed by the authors, and four categories of methods - WRS, WRE, WJE and DRE - have been identified. No methods have been found in the DRS, DJE, or DJS categories since dry injection methods only feature end mixing with relatively low pressure binder injection pressures via compressed air, and jetted methods only feature end mixing (hence no WJS).

3.5 Features of Methods

The FHWA Reports (1998 and 1999) provides extensive data on each of the methods identified in Figure 5. Space restrictions prevent more than a brief summary of the more significant methods being provided in these publications (Appendices 1 and 2). The following general points must be borne in mind while considering these data:

- New methods, refinements of existing methods, and developments in materials (e.g., use of flyash, gypsum and slag in slurries; clay dispersants to aid penetration and improve mixing efficiency) are continually underway.
- As noted by Taki and Bell (1997), the technical goal of any DMM technique is to provide a uniformly treated mixed body, with no discrete lumps of binder or soil, a uniform moisture content, and a uniform distribution of binder throughout the mass. The most important requirements for installation are therefore: thorough and uniform mixing of the soil and binder; appropriate water/cement ratio; and appropriate grout injection ratio (e.g., volume of grout: volume of treated soil).
- The table includes methods not conventionally or nationally regarded as DMM, for example, the SMW Method (used only for walls in Japan), while the Scandinavian practitioners do not conventionally address their Lime-Cement Column Method as DMM.

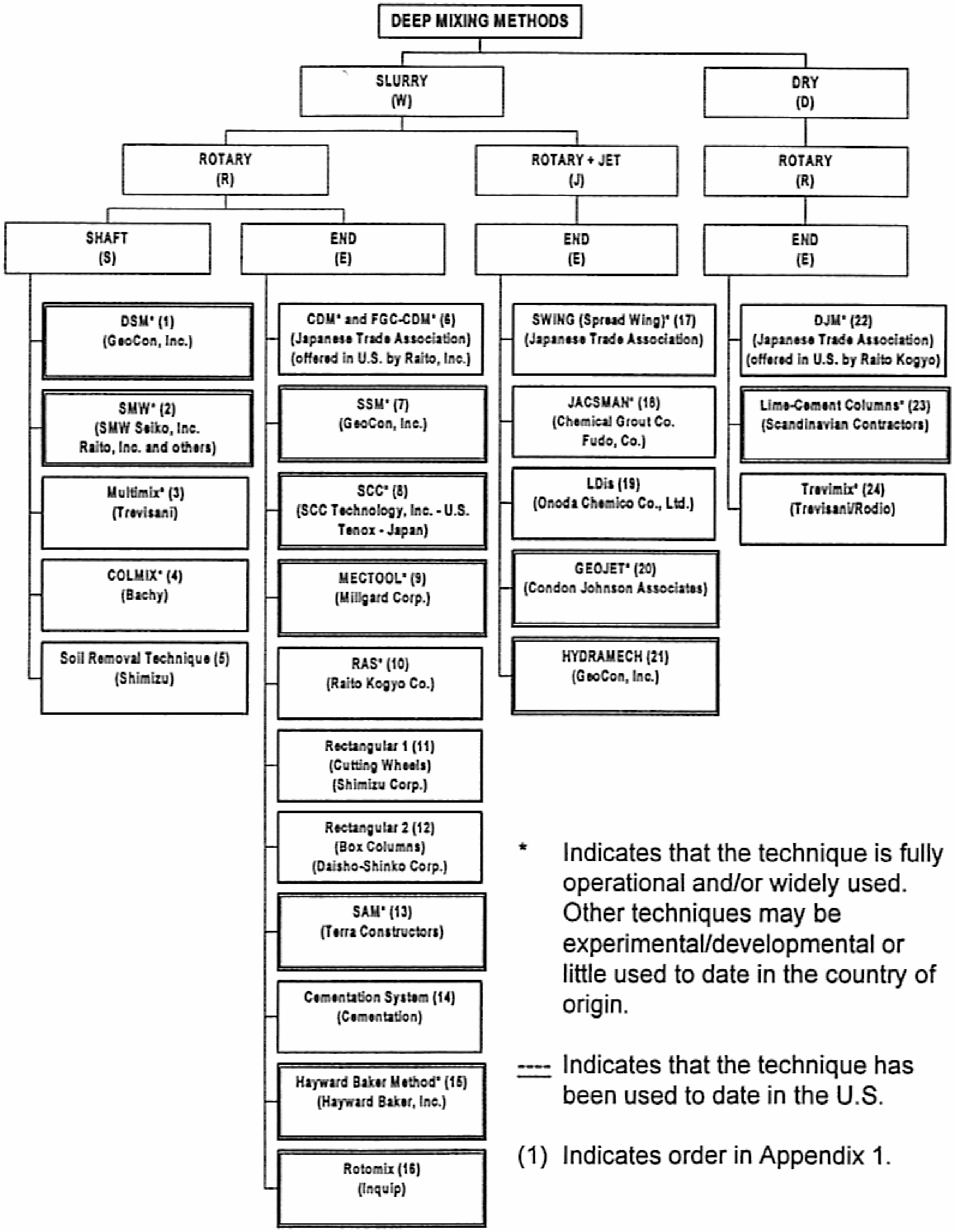

Figure 5. Generic classification of DMM techniques.

- Despite their generic similarity, there are major and significant regional and procedural variations. For example, unconfined compressive strengths (U.C.S.) of treated soil using WRE, WRS, and WJE are typically higher than 1 MPa, except (e.g., FGC-CDM) where lower strengths are deliberately engineered. For DRE methods in Japan (e.g., DJM) a minimum U.C.S. of 0.5 MPa is obtained, whereas for the comparable DRE Scandinavian method (Lime-Cement Columns), rarely are strengths in excess of 0.15 MPa designed and/or achieved. Furthermore, treated soils in Scandinavia may be considered as providing vertical drainage, whilst similar soils in other countries, by other methods, may be regarded as relatively impermeable.
- Table 3 (Terashi, 1997) summarizes the factors influencing the strength of treated soil. In laboratory testing, there is no way to simulate factors III and IV except for the amount of binder and the curing time. Thus laboratory testing features standardization of these factors, and so it must be realized that the strength data provided by such tests is "not a precise prediction" (Terashi, 1997) but only an "index" of the actual strength. Field testing is essential, and invariably appears to provide, for a number of reasons, inferior and more variable strength data.
- Deep mixing is, of course, not a panacea for all soft ground treatment, improvement, retention and containment problems, and in different applications it can be more or less practical, economic or preferable than competitive technologies. In the most general terms, DMM may be most attractive in projects where the ground is neither very stiff nor very dense, nor contains boulders or other obstructions; to depths of less than about 30m; where there is relatively unrestricted overhead clearance; where a constant and good supply of binder can be assured; where a significant amount of spoil can be tolerated; where a relatively vibration-free technology is required; where treated or improved ground volumes are large; where "performance specifications" are applicable; or where treated ground strengths have to be closely engineered (typically 0.1 to 5 MPa). Otherwise, and depending always on local conditions, it may prove more appropriate to use jet grouting, diaphragm walling, sheet piling, caissons, beams and lagging, driven piles, wick drains, micropiles, soil nails, vibrodensification, compaction grouting, deep dynamic consolidation, bioremediation, or vapor extraction.
- The materials injected are tailored to the method used, their local availability, the ground to be treated and the desired or intended result. Generally, for the methods using a fluid grout, the constituents include cements, water, bentonite, clay, gypsum, flyash, and various additives. Water cement ratios typically range from less than 1 to over 2, although the actual in place w/c ratio will depend on any "predrilling" activities with water, or other fluids. Most recently, dispersants (Gause, 1997) can be used, both to breakdown cohesive soils, and also to render more efficient the grout injected. For dry injection methods, cement and/or unslaked lime are the prime materials used.
- For wet methods (mechanically simpler and so preferable in "difficult" geographic locations), the cement injected is typically in the range of 100 to 500 kg per cubic meter of soil to be treated. The ratio of volume of fluid grout injected to soil mass treated is typically about 20 to 40%. (A lower injection ratio is preferable, to minimize cement usage and spoil).
- For dry methods, (in soils of 60 to over 200% moisture content), typically 100 to 300 kg of dry materials per cubic meter of treated soil are used, providing strengths of 0.2 to 20 MPa, depending very much on soil type (low strengths and solids contents in Scandinavia), with minimal spoil or heave potential.
- Treated soil properties (recalling that cohesive soils require more cement to give equivalent strengths than cohesionless soils) are usually in the ranges shown in Table 4.
- It must be remembered that different techniques are intended specifically to provide higher strengths, or lower permeabilities and so the figures cited above are gross ranges only, and that the data provided by the individual corporations supersede those presented above for specific applications.

Table 3. Factors affecting the strength increase (Terashi, 1997).

I	Characteristics of hardening agent	
	1.	Type of hardening agent
	2.	Quality
	3.	Mixing water and additives
II	Characteristics and conditions of soil (especially important for clays)	
	1.	Physical chemical and mineralogical properties of soil
	2.	Organic content
	3.	pH of pore water
	4.	Water content
III	Mixing conditions	
	1.	Degree of mixing
	2.	Timing of mixing/re-mixing
	3.	Quality of hardening agent
IV	Curing conditions	
	1.	Temperature
	2.	Curing time
	3.	Humidity
	4.	Wetting and drying/freezing and thawing, etc.

Table 4. Typical data on soil treated by deep mixing.

U.C.S.	0.2 - 5.0 MPa (0.5 - 5 MPa in granular soils) (0.2 - 2 MPa in cohesives)
k	10^{-6} - 10^{-9} m/s (lower if bentonite is used)
E	350 to 1000 times U.C.S. for lab samples and 150 to 500 times U.C.S. for field samples
Shear strength (direct shear, no normal stress)	40 to 50% of U.C.S. at U.C.S. values < 1 MPa, but this ratio decreases gradually as U.C.S. increases.
Tensile strength	Typically 8 - 14% U.C.S.
28-day U.C.S.	1.4 to 1.5 times the 7-day strength for silts and clays 2 times the 7-day strength for sands
60-day U.C.S.	1.5 times the 28-day U.C.S., while the ratio of 15 years to 60 days U.C.S. may be as high as 3 to 1. In general, grouts with high w/c ratios have much less long term strength gain beyond 28 days, however.

3.6 Commercial Aspects

In the United States, there are at least nine companies who offer, or claim to offer, deep mixing services. Four (GeoCon, Condon Johnson, Terra, and Millgard) appear to have no links with foreign ownership or licensees, having developed their own systems. The others (Hayward Baker, Raito, Seiko, Stabilator, and SCC) are either U.S. operations with foreign ownership or use methods under foreign license. Based on the authors' investigations, it would seem that from 1986 to 1992, the annual value of deep mixing work conducted was in the range of $10 to 20 million, increasing by over 50 percent to 1996. Since then, as a result of massive works in Boston, Salt Lake City and the West Coast, this annual volume is probably now in the range of $50 to 80 million. For DMM used in environmental applications, the annual market may be around $20 to 30 million, increasing at about 5 to 10 percent annually.

Large scale systems may cost $80,000 to $200,000 to mobilize (much lower for methods such as Lime-Cement Columns). Typical prices for treatment are $100 to 250/m^2, or $50 to 100/m^3.

In Japan, the CDM Association claims to have treated over 26 million m^3 of soil from 1977 to 1995 (30 percent in the period 1992 to 1995) with about 60 percent being offshore. The DJM Association records 16 million m^3 of soil treatment from 1980 to 1996, involving 2345 separate projects, and an annual volume now approaching 2 million m^3 (Figure 6). By 1994, SMW Seiko, referring to their deep mixing wall system, had recorded 4,000 projects worldwide for a total treatment of 12.5 million m^2 (7 million m^3).

In Scandinavia, Åhnberg's data (Figure 7) illustrate the rapid growth in Swedish applications, while similar data illustrate a strong but smaller and steadier market in Finland (about 250,000 m^3 per year, 80 percent of which is lime-cement columns.) Markets in Norway and the Baltic countries are much smaller but have considerable growth potential. Selling prices in Scandinavia are typically in the range of $7 to 12/lin. m.

Similar data have not been found for other European countries, but there is no evidence that levels of activity in countries like U.K., France, Germany and Italy currently approach those in the U.S.

4 ANCHORS

4.1 The New U.S. Recommendations

In June 1996, the revised Recommendations for Prestressed Rock and Soil Anchors were published in the United States by the Post-Tensioning Institute (PTI) based in Phoenix, Arizona. These new recommendations are an extensive revision of the previous version published in 1986 although they still cover only prestressed cement and resin grouted anchors. (Nierlich and Bruce, 1997).

The committee formed by PTI to work on the revised edition comprised representatives of all parties involved in anchor work. On the part of 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 in Dallas, Texas. The result of this cooperation between these often conflicting interests is a set of guidelines for the design, installation, and testing of anchors that are intended to be realistic and practical, while still satisfying the concerns for reliability and safety, which are recognized worldwide.

With respect to typical international anchor specifications, the PTI Recommendations may be considered to occupy a middle ground somewhere between the European Standard: "Execution of Special Geotechnical Work: Ground Anchors" (1994) with its precise and factual separation into statements, requirements, possibilities, recommendations, permissions, and application rules, and the "British Standard Code of Practice for Ground Anchors" (1989) which is viewed to be a most systematic and comprehensive exposé on the subject "Anchors" as well as constituting a formal standard. The PTI Recommendations limit themselves to the design of the anchor only, and do not address the design of the entire retained structure.

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

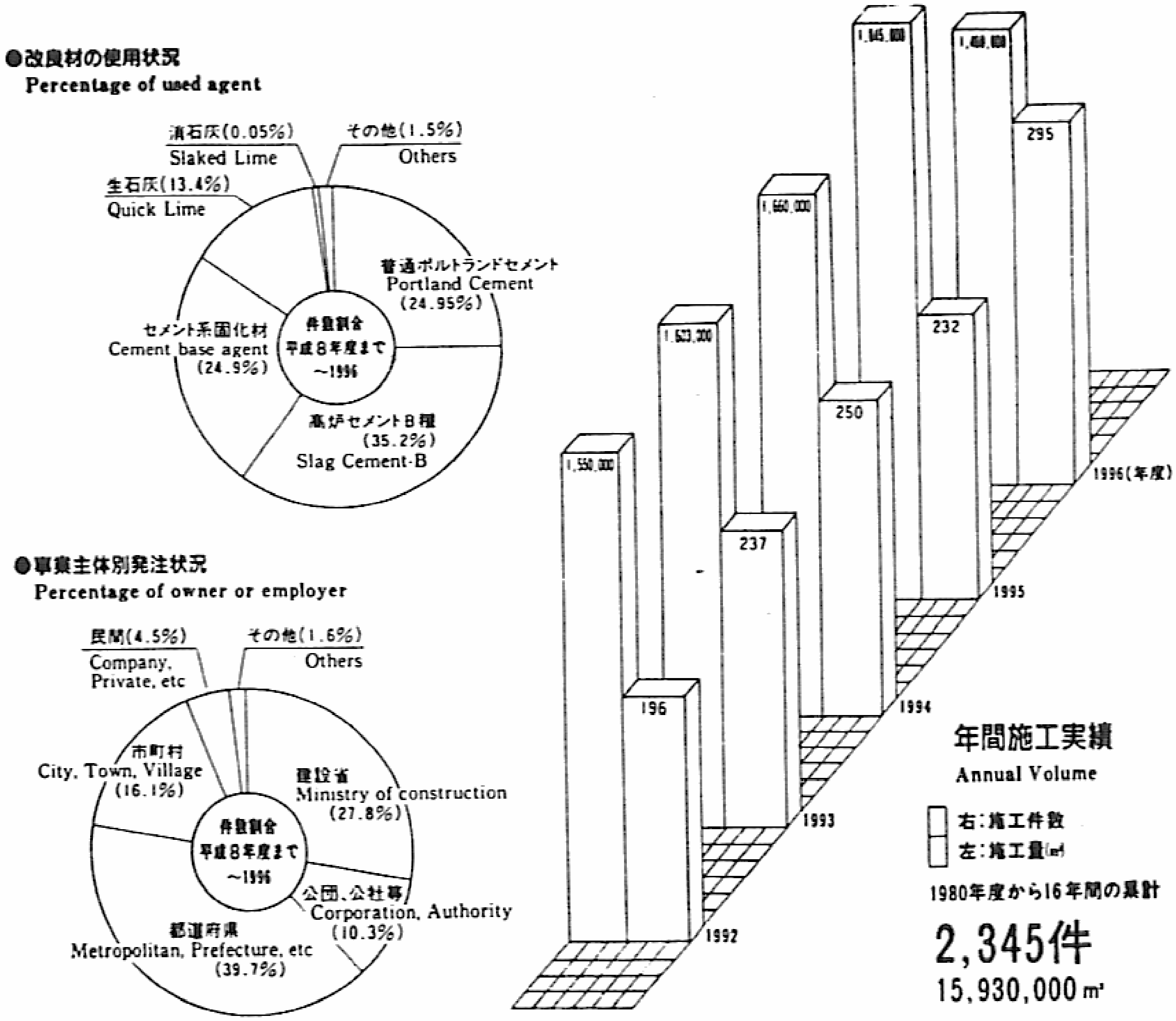

Figure 6. Data DJM usage in Japan. Left bar represents volume (m^3) and right bar represents number of projects (DJM Association, 1996).

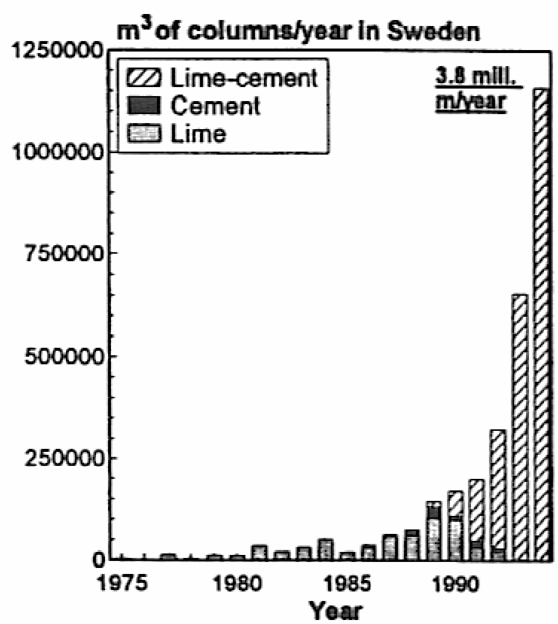

Figure 7. Use of different stabilizing agents for deep stabilization of soils in Sweden 1975-1994. (Åhnberg, 1996).

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 (Table 5). Anchor contractors in the United States often observe 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 new 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 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, 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 as 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 8 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,

Table 5. Tasks and responsibilities to be allocated for anchor works.

1.	Site investigation, geotechnical investigation, site survey and potential work restrictions.
2.	Decision to use a anchor system, requirement for a precontract testing program, type of specification and procurement method and levels of prequalification.
3.	Obtaining easements.
4.	Overall scope of the work, design of the anchored structure, and definition and qualification of safety factors.
5.	Definition of service life (temporary or permanent) and required degree of corrosion protection.
6.	Anchor spacing and orientation, minimum total anchor length, free anchor length and anchor load.
7.	Anchor components and details.
8.	Determination of bond length.
9.	Details of corrosion protection.
10.	Type and number of tests.
11.	Evaluation of test results.
12.	Construction methods, schedule, sequencing and coordination of work.
13.	Supervision of the work.
14.	Maintenance and long-term monitoring.
15.	Requirements for QA/QC Program.

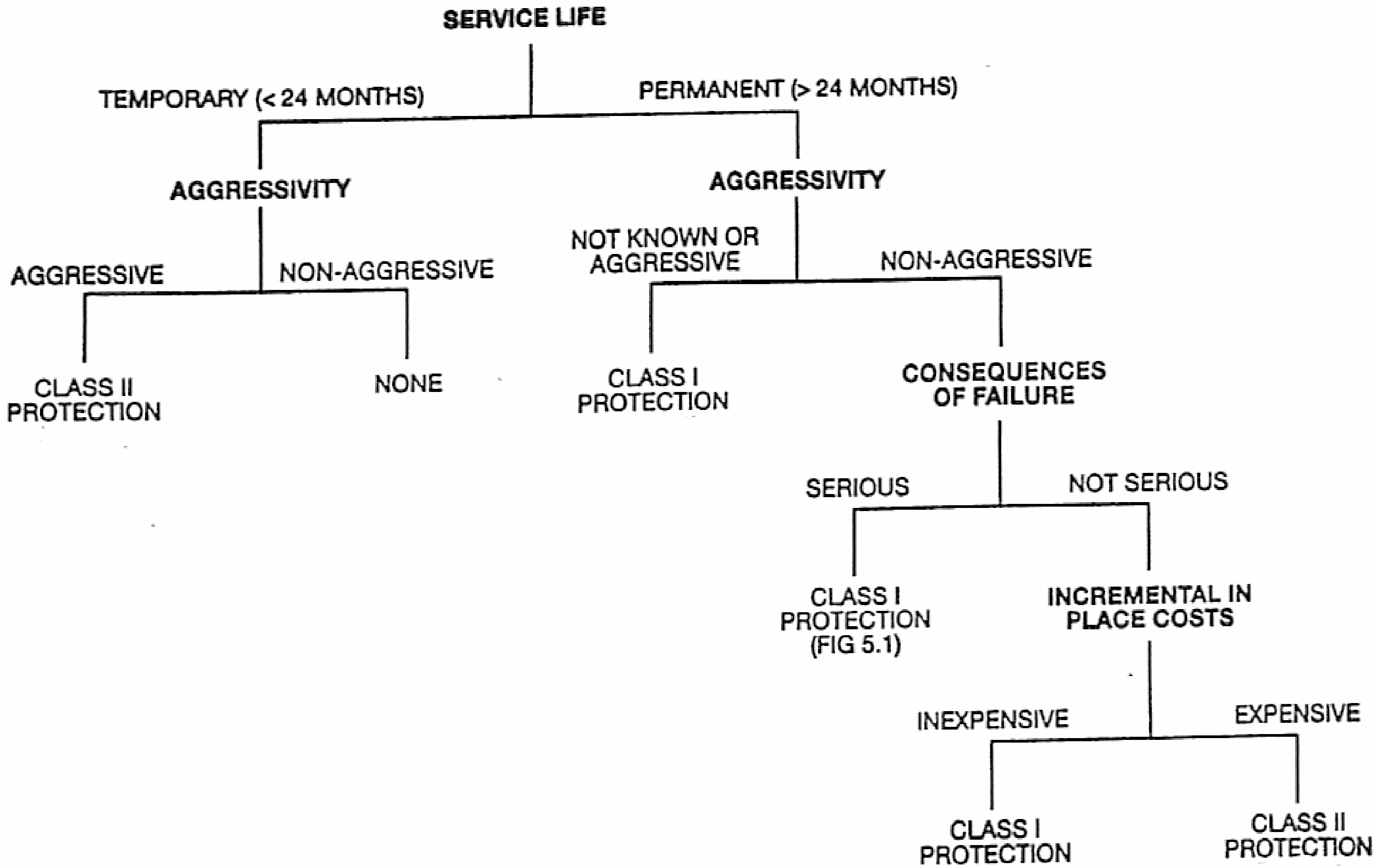

Figure 8. Corrosion protection decision tree.

Table 6. Corrosion protection requirements.

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

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 6 and Figures 9 and 10. 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 follows 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 separation 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 much 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 11). 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 separation of the total movement into residual and elastic movement is possible.

Another new section "Procedures in the Event of Failure during Testing" allows 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

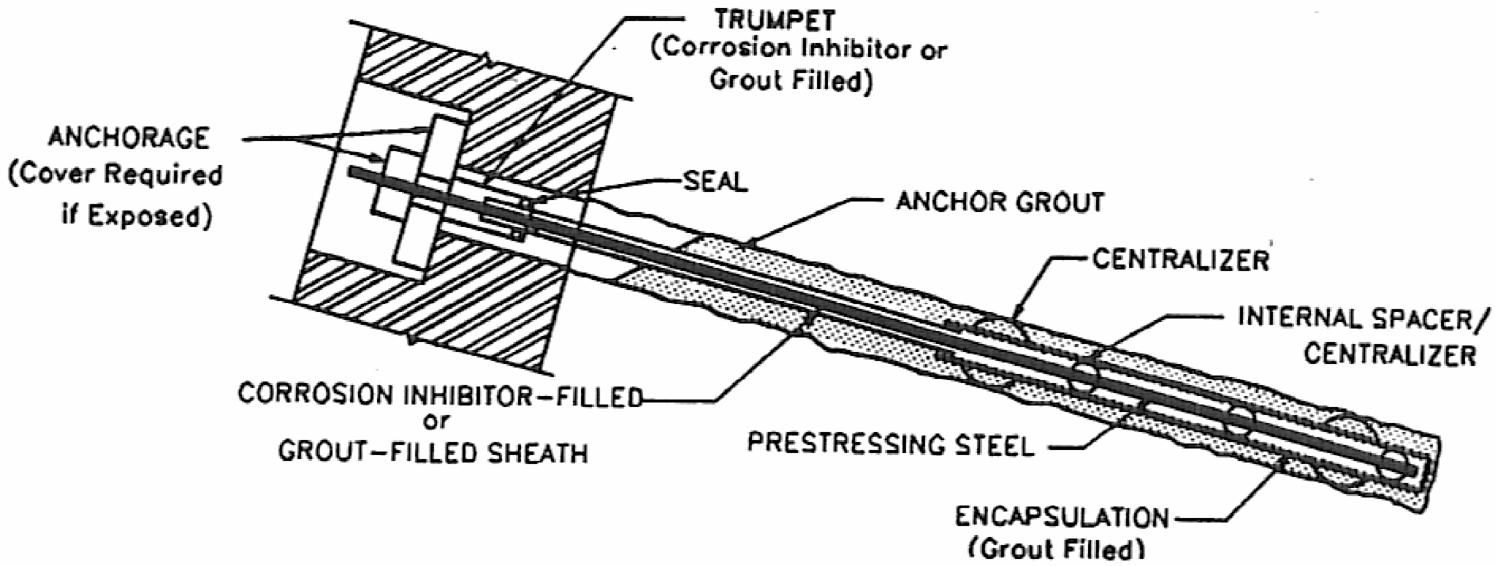

Figure 9. Class I protection – encapsulated anchor.

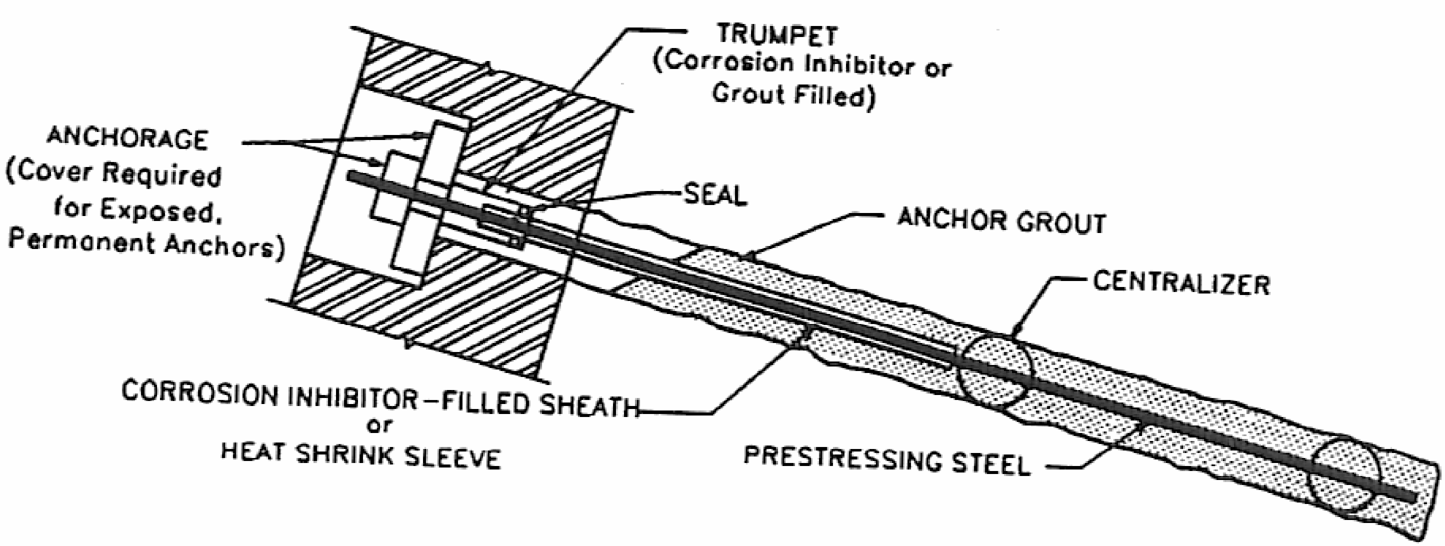

Figure 10. Class II protection – grout protected anchor.

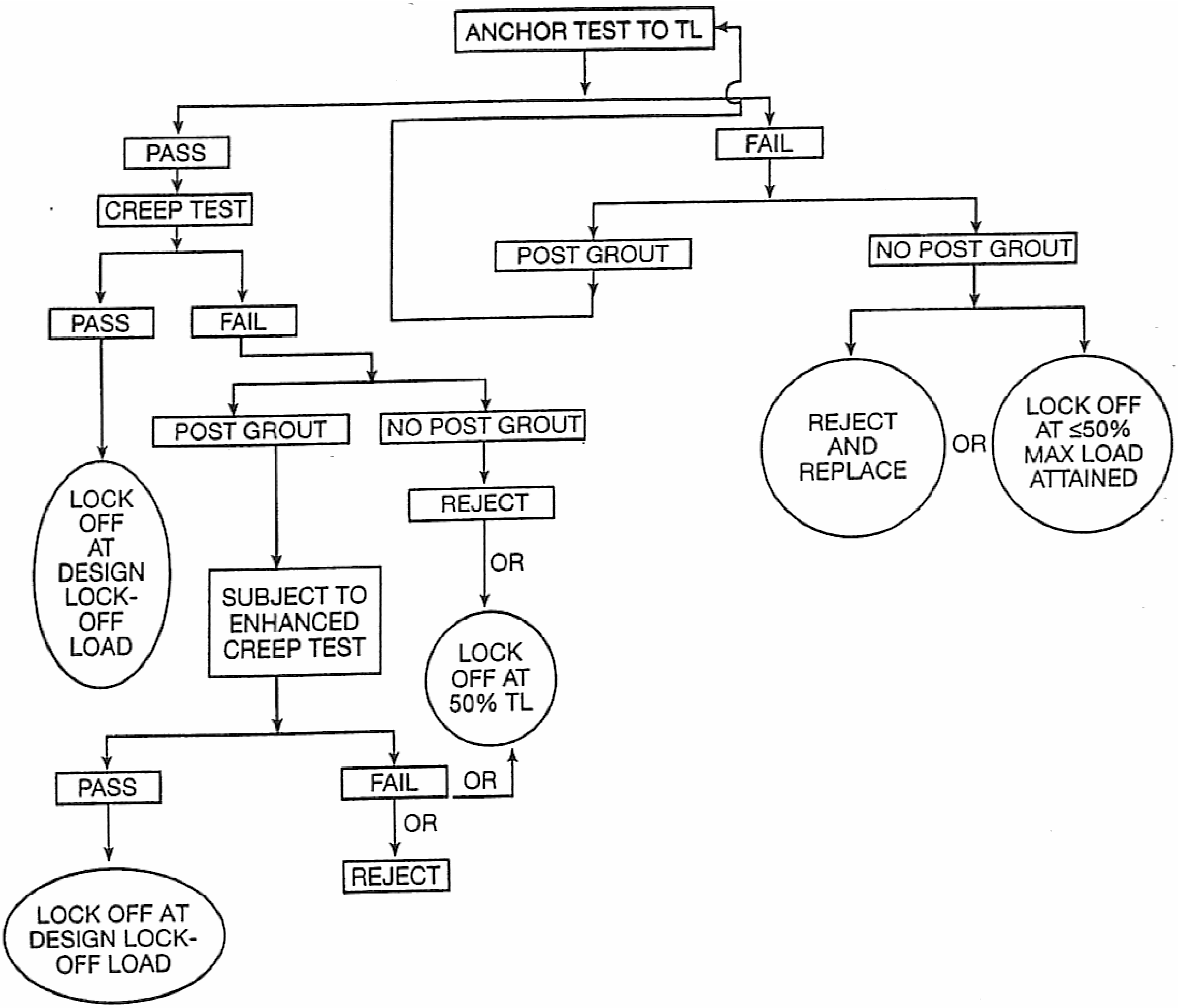

Figure 11. Decision diagram for acceptability testing of anchors.

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 rapidly 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 shall 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 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 these Revisions.

4.2 Further Comments

The new 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 insights and sources of knowledge is universally accepted. However, the Committee feel that they have produced a document which clarifies past inconsistencies and addresses 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.

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

5 MICROPILES

5.1 Introduction and History

5.1.1 Background

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

The activities of this group clearly ensured that a comprehensive review of practice was conducted. However, the synergies of this group more importantly were 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.

5.1.2 Scope

Micropiles are, generically, small-diameter, bored, grouted-in-place piles incorporating steel reinforcement. They have been used throughout the world for various purposes, and this has spawned a profusion of national and local names, including pali radice, micropali (Italian), pieux racines, pieux aiguilles, minipieux, micropieux (French), minipile, micropile, pin pile, root pile, needle pile (English), Verpresspfähle and Wurzelpfähle (German) and

Estaca Raiz (Portuguese). All, however, refer to the "special type of small diameter bored pile" as discussed by Koreck (1978).

Such a pile can withstand axial and/or lateral loads, and may be considered as either one component in a composite soil/pile mass or as a small diameter substitute for a conventional pile, depending on the design concept. Inherent in their genesis and application is the precept that micropiles are installed with methods that cause minimal disturbance to structure, soil and environmental.

5.1.3 Historical Note

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

5.2 Fundamental Concepts

5.2.1 Characteristics and Definitions

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

The development of highly specialized drilling equipment and methods also allows micropiles to be drilled through virtually every ground condition, natural and artificial, with minimal vibration, disturbance and noise, and at any angle below horizontal. Micropiles are therefore used widely for underpinning existing structures, and the equipment can be further adapted to operate in locations with low headroom and severely restricted access.

All of these observations of its traditionally recognized characteristics therefore lead to a fuller definition of a micropile: a small-diameter (less than 300 mm - note that in France the limit is set as 250 mm), replacement, drilled pile composed of placed or injected grout, and having some form of steel reinforcement to resist a high proportion of the design load. This load is mainly (and initially) accepted by the steel and transferred via the grout to the surrounding rock or soil, by high values of interfacial friction with minimal end bearing component, as is the case for ground anchors and soil nails. They are constructed by the type of equipment used for ground anchor and grouting projects, although micropiles often must be installed in low headroom and/or difficult access locations. They must be capable of causing minimal damage to structure or foundation material during installation and must be environmentally responsive. The majority of micropiles are between 100 and 250 mm in diameter, 20 to 30 m in length, and 300 to 1000 kN in compressive or tensile service load, although far greater depths and much higher loads are not uncommon in the United States.

5.2.2 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 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 12a). 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 author refers 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 12b. 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 is CASE 2 piles, was understandably an opponent of the practice of preloading high capacity micropiles, such as those 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.

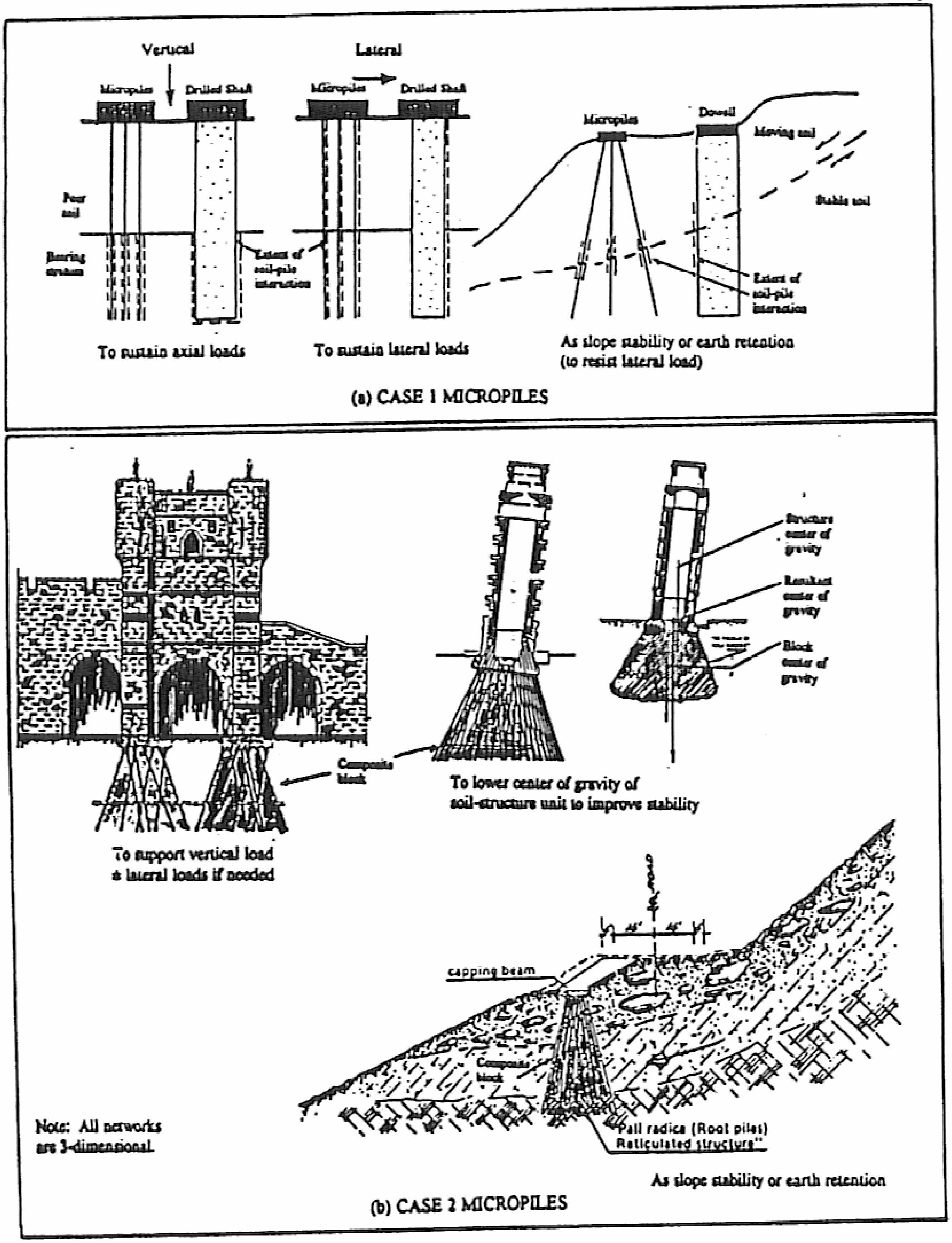

Figure 12. Fundamental classification of micropiles based on their supposed interaction with the soil.

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

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

There is no question that the *drilling* method and technique will affect the magnitude of the grout/ground bond which can be mobilized. On the other hand, 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., British Code 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. It is shown schematically in Figure 13.

- 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 stabilizing network is Type 2A.

5.2.3 Applications

Micropiles are used in two basic applications: as structural support and as in situ reinforcement (Figure 14). 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

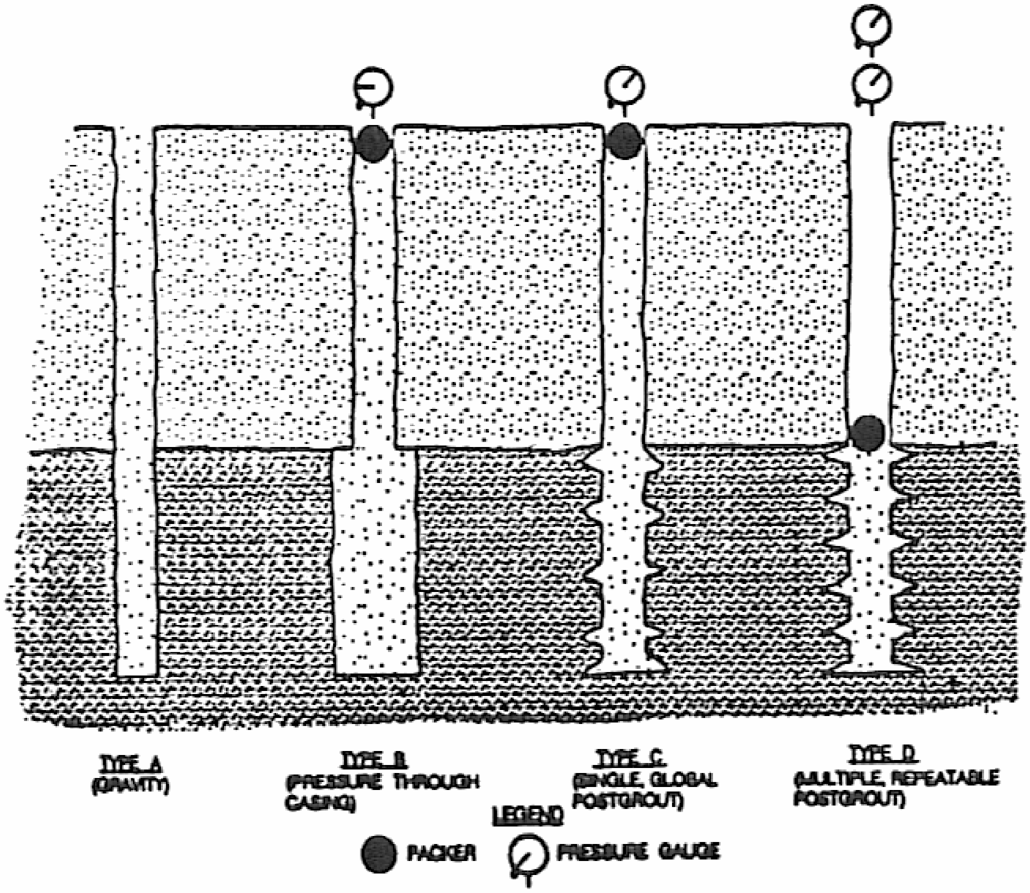

Figure 13. Classification of micropile based on type of grouting.

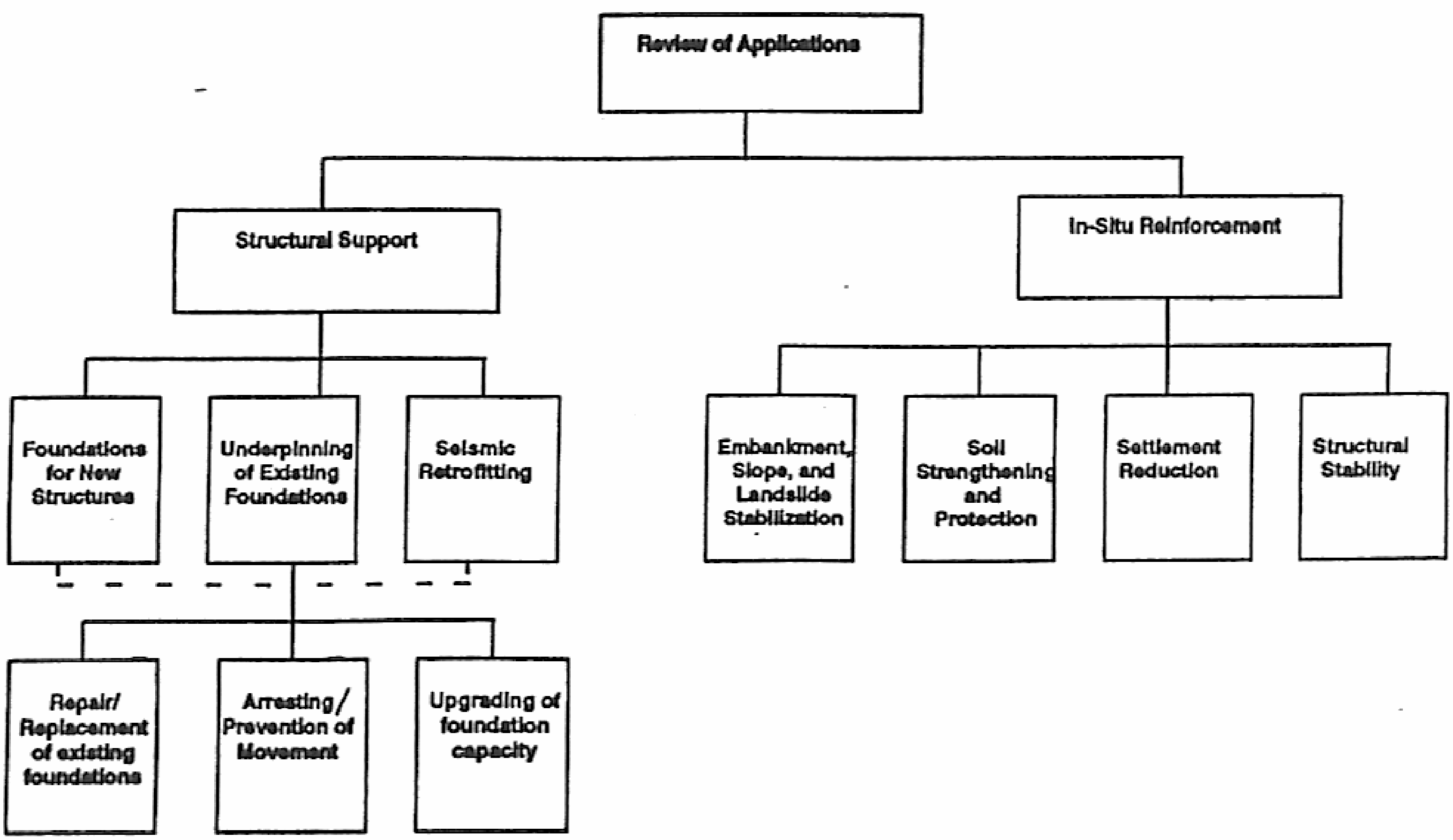

Figure 14. Classification of micropile applications.

in France. Table 7 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.

5.3 Construction

Figure 15 illustrates the standard successive steps in the construction of a Type B micropile. As noted in Section 2.2, Type A piles are not subjected to excess pressure during Primary grouting while Types C and D are pressure grouted at some point after the initial installation is completed. Highlights of the successive steps are as follows.

5.3.1 Drilling

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

Thereafter, the technical and economic success of the job is largely dependent on the contractor's ability to drill through the overburden, any obstructions (natural and artificial), and into the bedrock if that is where the pile is to be founded. There are fundamentally six methods of drilling overburden, as summarized in Table 8, and the most appropriate method is selected with respect to the site, the subsurface conditions, and the type and size of the pile.

Drilling rigs are typically diesel- or electro-hydraulic, and may be crawler or frame mounted. Special rigs have been developed for very restricted site conditions, and these rigs, although they may be relatively small in width and/or height, do provide considerable rotary power - essential for overburden drilling.

Drilling is most commonly conducted with water flush, although foam flush is frequently used in very difficult drilling conditions (Bruce et al., 1992). Air flush should never be permitted in urban environments for fear of causing pneumatic fissuring of the ground.

5.3.2 Reinforcement

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

Alternatively the reinforcement can be in the form of a pipe section, with or without additional central reinforcement for whole or part of the length.

5.3.3 Grouting

Grouts used in the Primary phase are stable, and have high strengths - typically in excess of 25 MPa. In the United States, neat water/cement mixes of w/c = 0.40 to 0.50 are common, whereas in other countries, sand/cement mixes are used, especially where grout takes into the surrounding formation (e.g., karstic limestone conditions) may be excessive. Special ground and/or ground water chemistries may require the use of special cements, but usually a Type I or II is sufficient - Type III if higher early strength is required. Additives are rarely necessary, although plasticizers are useful in very hot conditions or when pumping distances are substantial. Mixing is best conducted in high shear mixers.

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

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

5.4 QA/QC and Testing

5.4.1 During Installation

Full details are to be maintained through all the construction processes to ensure the final quality of the product. Of particular importance is the recording of all relevant grout pressure-volume-depth-time data, since to a large extent, the grouting

Table 7. Relationship between micropile application, design concept, and construction type.

APPLICATION	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%

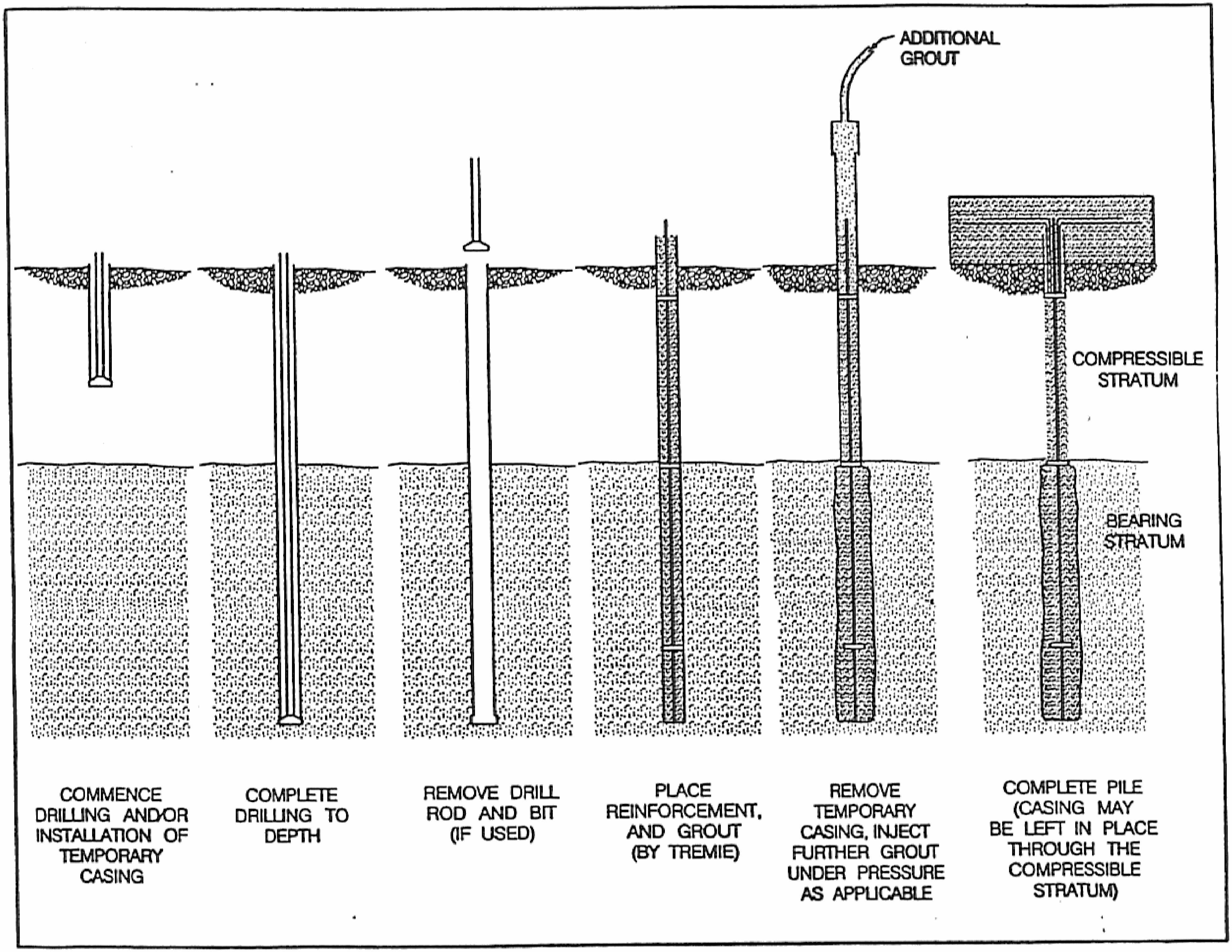

Figure 15. Typical construction sequence for a Type A and B micropile.

Table 8. Overburden drilling methods (Bruce, 1989).

DRILLING METHOD	PRINCIPLE	COMMON DIAMETERS AND DEPTHS	NOTES
1. Single tube advancement			
a) Drill drilling	Casing, with "lost point" percussed without flush.	50 - 100 mm to 30 m	Obstructions or very dense soils problematical.
b) External flush	Casing, with shoe, rotated with strong water flush.	100 - 200 mm to 60 m	Very common for anchor installation. Needs high torque and powerful flush pump.
2. Rotary duplex	Simultaneous rotation and advancement of casing plus internal rod, carrying flush.	100 - 200 mm to 70 m	Used only in very sensitive soil/site conditions. Needs positive flush return. Needs high torque.
3. Rotary percussive concentric duplex	As 2, above, except casing and rods percussed as well as rotated.	89 - 175 mm to 40 m	Useful in obstructed/bouldery conditions. Needs powerful top rotary percussive hammer.
4. Rotary percussive eccentric duples	As 2, except eccentric bit on rod cuts oversized hole to ease casing advance.	89 - 200 mm to 60 m	Somewhat obsolescent and technically difficult system for variable overburden.
5. "Double head" duplex	As 2 or 3, except casing and rods rotate in opposite senses.	100 - 150 mm to 60 m	Powerful, new system for fast, straight drilling in very difficult soils.
6. Hollow stem auger	Auger rotated to depth to permit subsequent introduction of reinforcement through stem.	150 - 400 mm to 30 m	Obstructions problematical; care must be exercised in cohesionless soils to avoid cavitation and/or loosening. Prevents application of higher grout pressures.

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

process is the major construction determinant of the grout/ground bond capacity. Good contractors also favor tests of the fluid grout (e.g., specific gravity, fluidity) prior to injection, to ensure that the injected grout meets the specifications. Samples for 7-, 14-, and 28-day strength testing give retrospective proof of the ability of the grout to reach specified quality.

5.4.2 After Installation

For axially-loaded CASE 1 piles, load tests are conducted on a representative number of elements. It is common to use ASTM D 1143-81 (Compression) and ASTM D 3689-87 (Tension), although the information yield from both can be greatly expanded by incrementally cycling the load, in the fashion of anchor testing (PTI, 1996). Such testing permits the total pile movements to be partitioned into permanent and elastic components, so allowing fundamental investigations into load transfer mechanisms. CASE 1 piles subjected to lateral loading can be testing according to ASTM D 3966-81.

There is no common, absolute set of acceptance criteria for CASE 1 axially-loaded piles, although many "solutions" based on geometric analyses of load-total movement curves have been proposed (Kulhawy et al., 1991). Basically, the acceptance criteria should be selected project by project, with respect to short-term movement, and creep performance. Analyzing pile load test data to meet these criteria is best conducted with the full insight afforded by cyclic loading programs.

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

6 FINAL REMARKS

The continuously changing and ever-increasing demands of contemporary ground engineering projects are naturally leading to significant developments in certain construction technologies. In North America, the days of massive, but straight-forward "green site" construction projects have been superseded by an emphasis on urban engineering and infrastructure rehabilitation projects, typically and usually executed against challenging geotechnical, environmental logistical and scheduling factors. This change has occurred some two or three decades after similar trends in other, older more densely populated areas such as in Western Europe and South East Asia.

The end of the twentieth century therefore sees rapid advances in many ground engineering techniques throughout the world, fostered by specialists who constantly compete with each other to create innovative "best quality" solutions on the one hand, and who efficiently exchange information via contemporary technology transfer vehicles.

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Appendix I. Summary of mixing equipment and pertinent information for each technique.

Name		DSM 1	SMW 2
Classification		W-R-S	W-R-S
Company		Geo-Con, Inc.	SMW Seiko, Inc.; Raito, Inc., and others
Geography		N. America	Southeast Asia, U.S.
General Description of Most Typical Method		Multiple discontinuous augers on hanging leads rotate in alternate directions. Most of grout injected on downstroke to create panels. Neither air nor water typically used during penetration. Reverse rotation during withdrawal.	Multiple discontinuous augers on fixed leads rotate in alternate directions. Water, air or grout used on downstroke and/or grout on upstroke
Special Features / Patented Aspects		Lower 3 m usually double-stroked. Strong QA/QC by electronic methods. Patent pending on VERTWall concept.	Special electric head and gear box patented. Double-stroking "oscillation" common, especially in cohesive soils. Discontinuous auger flights and paddles are positioned at discrete intervals to reduce torque requirements. Good control over verticality feasible. Auger type varies with soil.
Details of Installation	Shafts	1-6, usually 4	2-5, usually 3
	Diameter	0.8 to 1.0 m, usually 0.9 m	0.55 to 1.5 m, usually 850-900 mm
	Realistic max. depth	45 m possible, 27 m common	60 m claimed, 35 m practical
	rpm	15-25	15-20 during penetration, depending on soil; higher during withdrawal
	Productivity/ output	0.6-1.0 m/min penetration (slower in clays and dense sands); 2 m/min withdrawal/mixing; 100-150 m^2/shift industrial	0.5-1.5 m/min penetration; 1.5-2 m/min withdrawal/mixing; 100-200 m^3 per shift, i.e., 100-150 m^2 per shift
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement grout ± bentonite ± clay and other materials and additives, such as ash, slag	Cement grout ± bentonite and other additives such as ash, slag
	w/c ratio	1.2-1.75 (typically 1.5 on penetration and 1 to 1.25 during withdrawal)	1.25-1.50 (sands) - 2.5 (cohesives)
	Cement factor (kg_{cement}/m^3_{soil})	120-400 kg/m^3	200-750 kg/m^3
	Volume ratio ($Vol_{grout}:Vol_{soil}$)	15-40%	50-100%
Reported Treated Soil Properties	U.C.S.	0.3-7 MPa (clay strengths approx. 40% of those in sands); In sands, 2+ MPa	0.3-1.3 MPa (clays) 1.2-4.2 MPa (sands)
	k	1×10^{-7} to 1×10^{-9} m/s	1×10^{-7} to 1×10^{-10} m/s
	E	300 to 1000 x U.C.S.	350 to 1350 x U.C.S.
Specific Relative Advantages and Disadvantages		Economical, proven systems; mixing efficiency can be poor in stiff cohesive soils (especially SMW Seiko); can generate large spoil volumes, proportional to volume ratio required for mixing efficiency and treated soil requirements	
Notes		First DSM application at Bay City, MI in 1987.	Developed by Seiko in 1972: first used 1976 in Japan, 1986 in U.S. Trade Association in Japan.
Representative References		Ryan and Jasperse (1989, 1992); Day and Ryan (1995); Nicholson et al., 1998	Taki and Yang (1989, 1991); Yang (1997)

*ND = No data; NA = Not applicable.

Appendix I. Summary of mixing equipment and pertinent information for each technique (continued).

Name		Multimix (Trevimix) 3	CDM 6
Classification		W-R-S	W-R-E
Company		Trevisani	More than 48 members of CDM Association in Japan
Geography		Italy, U.S.	Japan, China
General Description of Most Typical Method		Multiple cable-suspended augers rotate in opposite directions. Grout injected during penetration. Prestroked with water in clays. Auger rotation reversed during withdrawal. Mixing occurs over 8- to 10-m length of shaft.	Fixed leads support shafts with 4-6 mixing blades above drill bit. Grout injected during penetration and (mainly) withdrawal. Also a 2- to 8-min mixing period at full depth.
Special Features / Patented Aspects		Pre-drilling with water ± additives in very resistant soils. Process is patented by TREVI. Developed especially for cohesionless soils of low/medium density, and weak clays.	Comprises numerous subtly different methods all under CDM Association
Details of Installation	Shafts	1-3, typically 3. Configuration varies with soil.	2-8 (marine): 1-2 (land) (each with 4-6 blades) (12 have been used)
	Diameter	0.55-0.8 m at 0.4 to 0.6-m spacings	1-2 m (marine): 0.7-1.5 m (land)
	Realistic max. depth	25m	70 m (marine): 40 m (land)
	rpm	12-30	20-30 (penetration); 40-60 (withdrawal)
	Productivity/ output	0.35-1.1 m/min penetration (typically 0.5) 0.48-2 m/min withdrawal	0.5-2 m/min (avg. 1 m/min) (penetration) 1-2 m/min (withdrawal) (1000 m^3/shift for marine; 100-200 m^3/shift on land)
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement grout mainly, plus bentonite in sands; additives common, even in predrilling phase	Wide range of materials, including portland or slag cement, bentonite, gypsum, flyash, using fresh or seawater; plus various additives.
	w/c ratio	Typically low, i.e., 0.6-1.0 (especially in cohesives)	0.6-1.3, typically 1.0
	Cement factor (kg_{cement}/m^3_{soil})	200-250 kg/m^3 typical (80-450 kg/m^3 range)	100-300 kg/m^3, typically 140 to 200 kg/m^3
	Volume ratio ($Vol_{grout}:Vol_{soil}$)	15-40%	20-30%
Reported Treated Soil Properties	U.C.S.	0.5-5 MPa (sands); 0.2-1 MPa (silts, clays); up to 20 MPa in very hard soils	Strengths can be closely controlled, by varying grout composition, from < 0.5-4 MPa (typically 2-4)
	k	$< 1 \times 10^{-8}$ m/s	1×10^{-8} to 1×10^{-9} m/s
	E	ND*	350 to 1000 x U.C.S. (lab) 150 to 500 x U.C.S. (field)
Specific Relative Advantages and Disadvantages		Goals are to minimize spoils (10-20%) and presence of unmixed zones within and between panels	Vast amount of R&D information available. Specifically developed for softer marine deposits and fills, now also used for land-based projects.
Notes		Developed jointly in 1991 by TREVI and Rodio.	Association founded in 1977. Research initiated under Japanese Government (1967). Offered in the U.S. by Raito, Inc.
Representative References		Pagliacci and Pagotto (1994)	CDM (1996); Okumura (1996)

*ND = No data; NA = Not applicable.

Appendix I. Summary of mixing equipment and pertinent information for each technique (continued).

Name		SCC 8	HBM (Single Axis Tooling) 15
Classification		W-R-E	W-R-E
Company		SCC Technology, Inc.	Hayward Baker Inc., a Keller Co.
Geography		SCC (U.S.); Tenox (Japan)	U.S. (but with opportunities for sister companies worldwide)
General Description of Most Typical Method		Grout is injected from shafts on fixed leads during penetration. A "share blade" is located above tip (non-rotating). At target depth, 1 minute of additional injection plus oscillation for 1.5-3 m. Withdrawal with counter rotation and no further grout injection.	Cable-suspended shaft rotated by bottom rotary drive table. Grout injected usually during penetration, followed by 5 minutes mixing and oscillation at full depth, and rapid extraction with injection of "backfill grout" only (1-5% total).
Special Features / Patented Aspects		Very thorough mixing via "share blade" action, which is patented.	Method proprietary to Keller.
Details of Installation	Shafts	Single with 3 pairs of rotated mixing blades plus "share blade". Double shafts are possible for ground stabilization; single shaft for piles.	Single with 2 or 3 pairs of mixing paddles above drill bit.
	Diameter	0.6-1.5 m; 1.2 m for double shafts.	0.5-3.5 m, typically 2.1 and 2.4 m
	Realistic max. depth	20 m max	20 m max.
	rpm	30-60	20-25 (penetration); higher upon withdrawal
	Productivity/ output	1 m/min penetration and withdrawal 100 m^2 of wall up to 400 m of piles/8-h shift	0.3-0.5 m/min (penetration); faster upon withdrawal. In excess of 500 m^3/shift
Mix Design *(depends on soil type and strength requirements)*	Materials	Typically cement grout, but others, e.g., ash, bentonite, possible.	Varied in response to soil type and needs
	w/c ratio	0.6-0.8 (clays) to 1.0-1.2 (sands)	1-2 (typically at lower end)
	Cement factor (kg_{cement}/m^3_{soil})	150-400 kg/m^3 cement	150 kg/m^3
	Volume ratio ($Vol_{grout}:Vol_{soil}$)	25-35%	15-30%
Reported Treated Soil Properties	U.C.S.	3.5-7 MPa (sands) 1.3-7 MPa (cohesives)	3.5-10 MPa (sands) 0.2-1.4 MPa (clays)
	k	1×10^{-8} m/s	1×10^{-10} m/s possible
	E	180 x U.C.S.	ND*
Specific Relative Advantages and Disadvantages		Low spoil with minimal grout loss claimed, due to low w/c and minimized injected volume. Very efficient mixing.	Good mixing; moderate penetration capability; low spoils volume. Dry binder method also available.
Notes		Used since 1979 in Japan and 1993 in U.S.	In development since 1990. Commercially viable since 1997.
Representative References		Taki and Bell (1997)	Burke et al., 1998

*ND = No data; NA = Not applicable.

Appendix I. Summary of mixing equipment and pertinent information for each technique (continued).

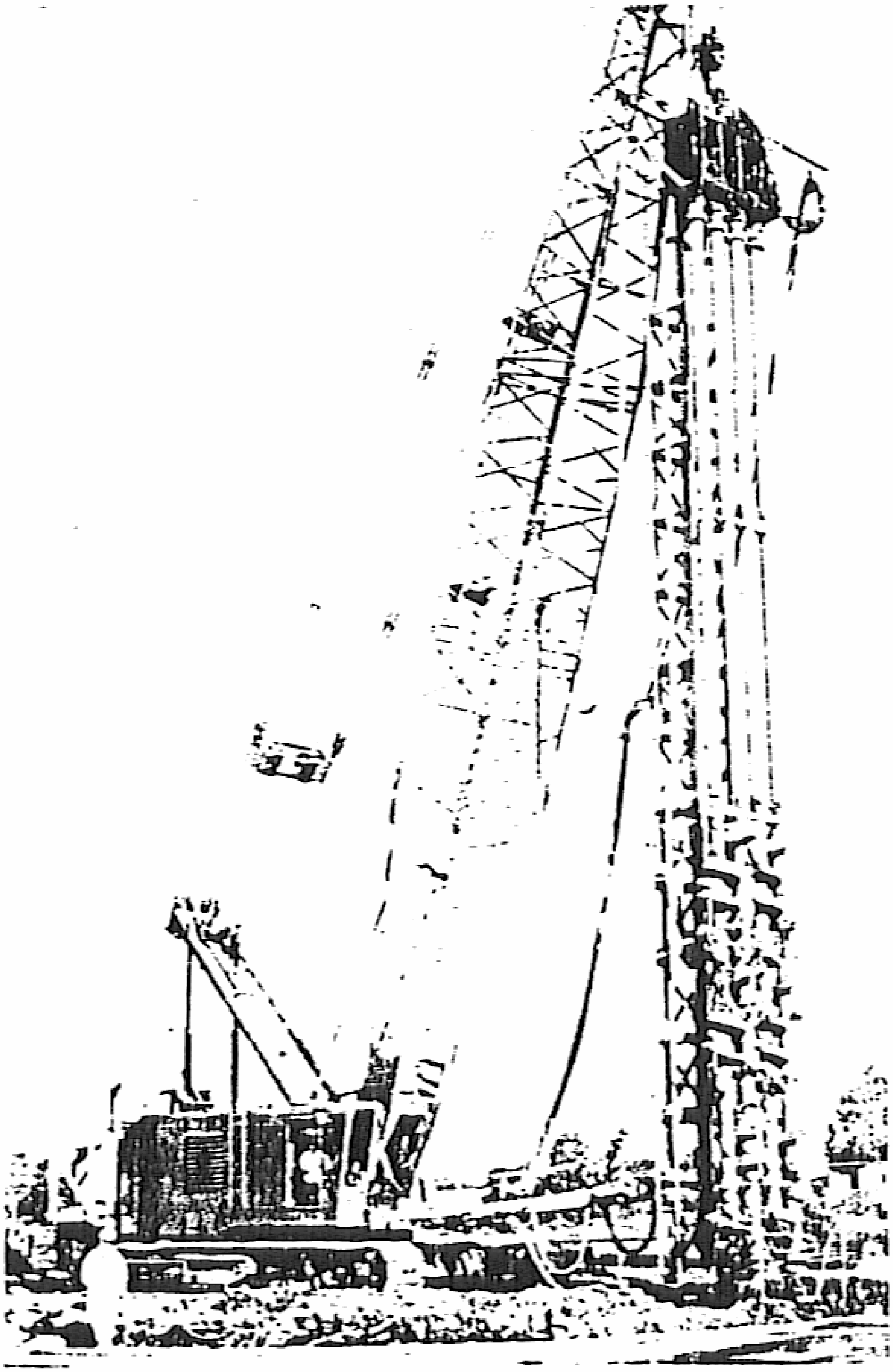
Name		JACSMAN 18	GeoJet 20
Classification		W-J-E	W-J-E
Company		Chemical Grout Co., Fudo Co., & others	Condon Johnson and Associates
Geography		Japan	Western U.S.
General Description of Most Typical Method		Twin counter-rotating shafts, grout injected at low pressure from cutting blades during penetration. During withdrawal, inclined, crossed jets on upper two pairs of blades are used at high velocities to increase diameter and enhance mixing efficiency	Grout is jetted via ports on a "processor" during rapid penetration. The wings cut the soil and the jetted grout blends it.
Special Features / Patented Aspects		The combination of DMM and jet grouting ensures good joints between adjacent columns, and columns of controlled diameter and quality. Column formed is nominally 1.9 m x 2.7 m in plan. Patented process. Trade association.	Combination of mechanical and hydraulic cutting/mixing gives high-quality mixing and fast penetration. Licensed by CJA for five western states. Trevi-ICOS for the remainder. Very low environmental impact.
Details of Installation	Shafts	2 shafts at 0.8-m spacing each with 3 blades.	1 shaft with pair of wings or similar "processor"
	Diameter	1 m (blades at 0.8-m spacing along shaft)	0.6-1.2 m
	Realistic max. depth	20 m	45 m max (25 m typical)
	rpm	20	150-200 (recent developments focusing on 80-90 rpm)
	Productivity/ output	1 m/min penetration 0.5-1 m/min withdrawal	2-12 m/min (penetration) (6 m/min typical) 15 m/min (withdrawal); 150 m of piles/h possible
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement grout	Cement grout; additives if necessary
	w/c ratio	1.0	0.5-1.5 (typically 0.8-1.0)
	Cement factor (kg_{cement}/m^3_{soil})	200 kg/m^3 (jetted); 320 kg/m^3 (DMM). Air also used to enhance jetting	150-300 kg/m^3
	Volume ratio ($Vol_{grout}:Vol_{soil}$)	200 L/min per shaft during DM penetration; 300 L/min per shaft during withdrawal (jetting); i.e., 20-30%	20-40%
Reported Treated Soil Properties	U.C.S.	2-5.8 MPa (silty sand and clay) 1.2-3 MPa (silty sand)	0.7-5.5 MPa (Bay mud) 4.8-10.3 MPa (Beaumont clay)
	k	ND*	ND*
	E	ND*	ND*
Specific Relative Advantages and Disadvantages		New system combining DMM and jet-grouting principles to enhance volume and quality of treatment; jetting provides good overlap between columns.	Computer control of penetration parameters excellent. High strength. Low spoil volumes. High repeatability. Excellent mixing. High productivity.
Notes		Name is an acronym for Jet and Churning System Management.	Developed since early 1990s. Fully operational in Bay Area. Five patents on "processor", system, and computer control; three patents pending.
Representative References		Miyoshi and Hirayama (1996)	Reavis and Freyaldenhoven (1994)

*ND = No data; NA = Not applicable.

Appendix I. Summary of mixing equipment and pertinent information for each technique (continued).

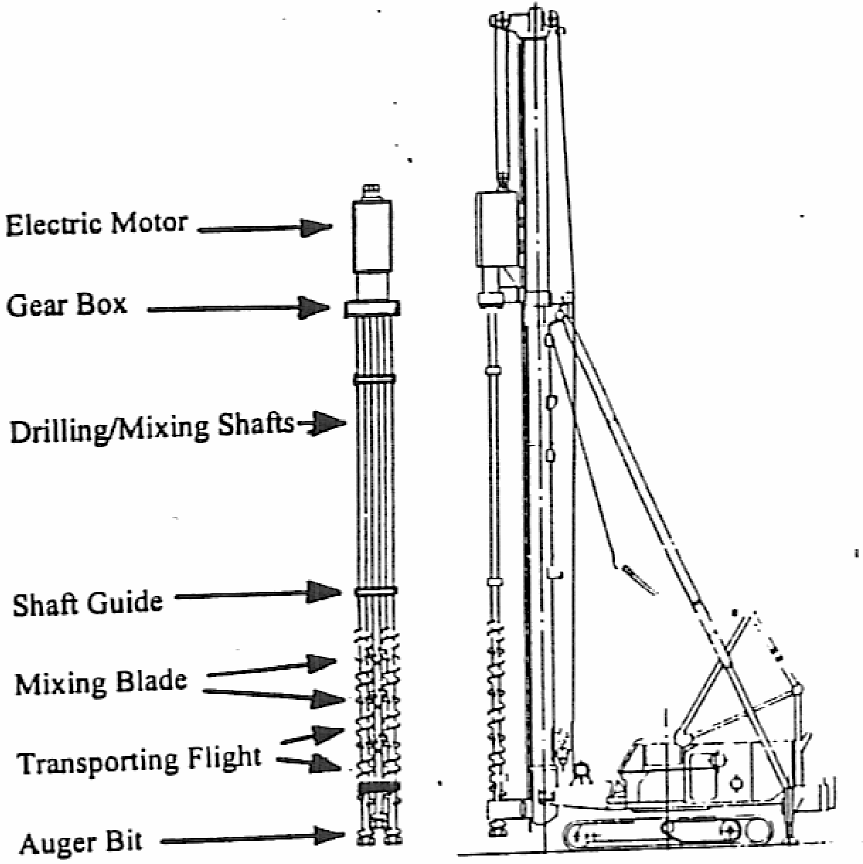
Name		Dry Jet Mixing 22	Lime Cement Columns 23
Classification		D-R-E	D-R-E
Company		DJM Association (64 companies)	Various (in Scandinavia/Far East). Stabilator alone in U.S.
Geography		Japan	Scandinavia, Far East, U.S.
General Description of Most Typical Method		Shafts are rotated while injecting compressed air from the lower blades to avoid clogging of jet nozzles. Dry materials are injected during withdrawal via compressed air, and with reverse rotation. Air vents to surface around the square section shafts.	Shaft is rotated while injecting compressed air below mixing tool to keep injection ports clear. Dry materials are injected during withdrawal via compressed air, and reverse rotation. Requires sufficient free water to hydrate binder, e.g., sand >15%; silt >20%; clay >35%.
Special Features / Patented Aspects		System is patented and protected by DJM Association. Two basic patents (blade design and electronic control system). Many supplementary patents.	Very low spoil. High productivity. Efficient mixing. No patents believed current. Strong reliance on computer control. Close involvement by SGI.
Details of Installation	Shafts	1-2 shafts adjustably spaced at 0.8 to ~1.5 m, each with 2-3 pairs of blades	Single shaft, various types of cutting/mixing blades.
	Diameter	1 m	0.5-1.2 m, typically 0.6 or 0.8 m
	Realistic max. depth	33 m max.	30 m max. (20 m typical)
	rpm	24-32 during penetration. Twice as high during withdrawal.	100-200, usually 130-170
	Productivity/ output	0.5 m/min penetration; 3 m/min withdrawal.35-45% lower in low-headroom conditions	2-3 m/min (penetration) 0.6-0.9 m/min (withdrawal) 400-1000 lin m/shift (0.6 m diameter)
Mix Design *(depends on soil type and strength requirements)*	Materials	Usually cement, but quicklime is used in clays of very high moisture content	Cement and lime in various percentages (typically 50:50 or 75:25)
	w/c ratio	NA*	NA*
	Cement factor ($kg_{cement}/m^3{}_{soil}$)	100-400 kg/m^3 (sands and fine grained soil using cement); 200-600 kg/m^3 (peats and organics using cement); 50-300 kg/m^3 (soft marine clays using lime)	23-28 kg/m (0.6 m diameter), typically 40 kg/m (0.8 m diameter); overall 20-60 kg/m i.e., 80-150 kg/m^3
	Volume ratio (Vol_{grout}:Vol_{soil})	NA*	NA*
Reported Treated Soil Properties	U.C.S.	Greatly varies depending on soil and binder, 1-10 MPa	Varies, but typically 0.2-0.5 MPa (0.2-2 MPa possible). Shear strength 0.1-0.30 MPa (up to 1 MPa in field)
	k	"Higher than CDM permeabilities"	For lime columns, k = 1000 times higher than the k of the clay; for lime-cement columns, the factor is 400 to 500.
	E	E_{50} = 50 to 200 x U.C.S.	50 to 200 x U.C.S.
Specific Relative Advantages and Disadvantages		Heavy rotary heads remain at bottom of leads, improving mechanical stability of rigs, especially in soft conditions. Very little spoils; efficient mixing. Extensive R&D experience. Fast production on large jobs.	Same as for DJM. Excellent Swedish/Finnish research continues.
Notes		Sponsored by Japanese Government and fully operational in 1980. (First application in 1981.) Offered in the U.S. by Raito, Inc. since 1998.	Developed by Swedish industry and Government, with first commercial applications in mid 1970s, and first U.S. application in 1996.
Representative References		DJM Brochure (1996); Fujita (1996); Yang et al., 1998	Holm (1994); Rathmeyer (1996)

*ND = No data; NA = Not applicable.

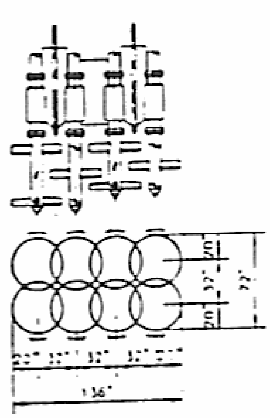
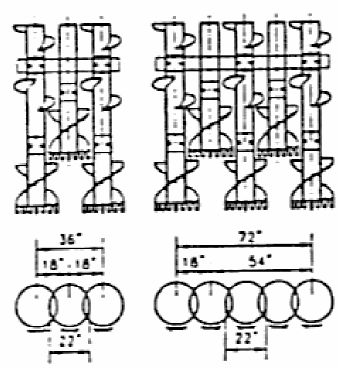


DSM system (Courtesy: GeoCon)

Appendix 2. Illustrations of various Deep Mixing Methods described in Appendix 1 (continues).

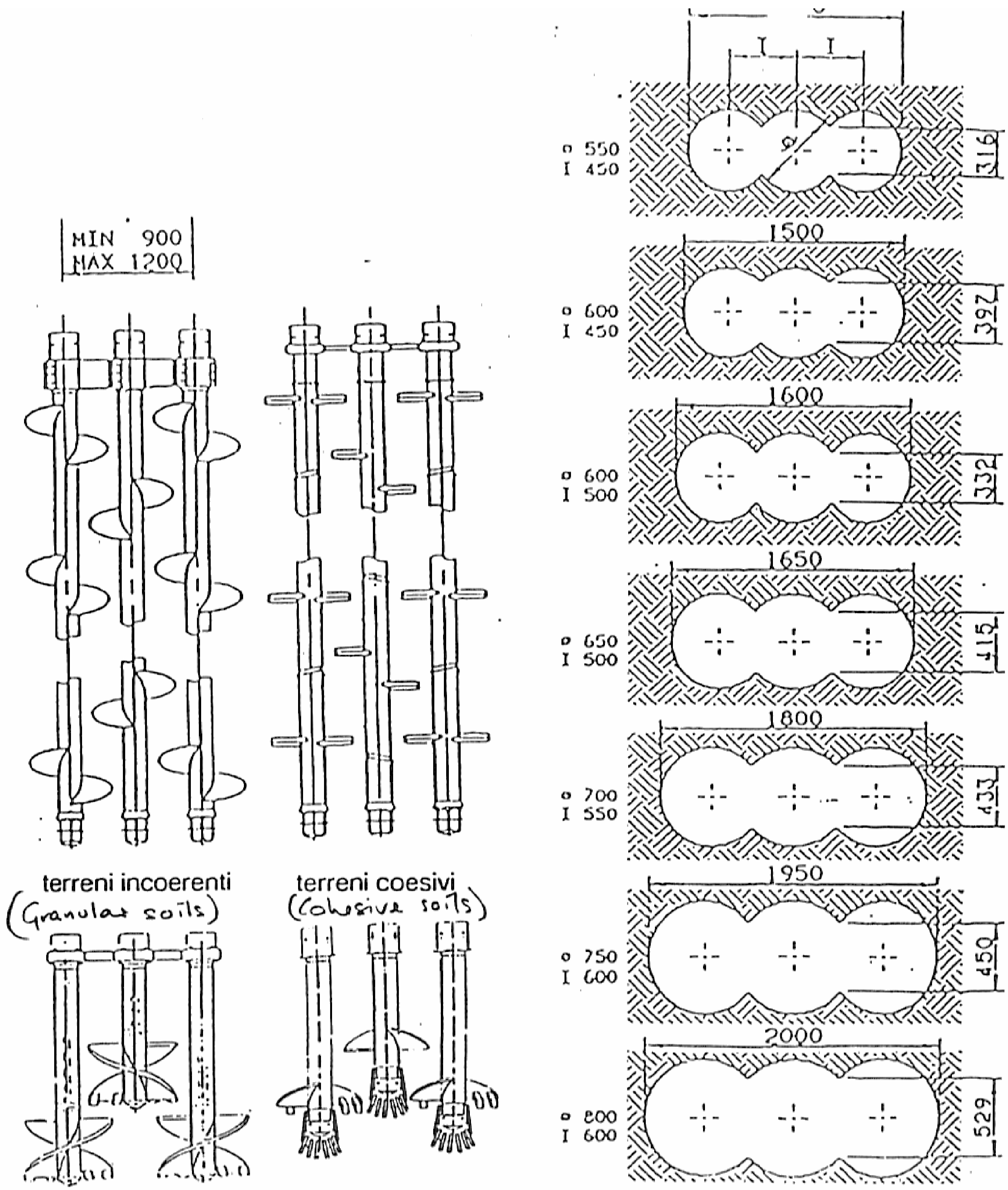


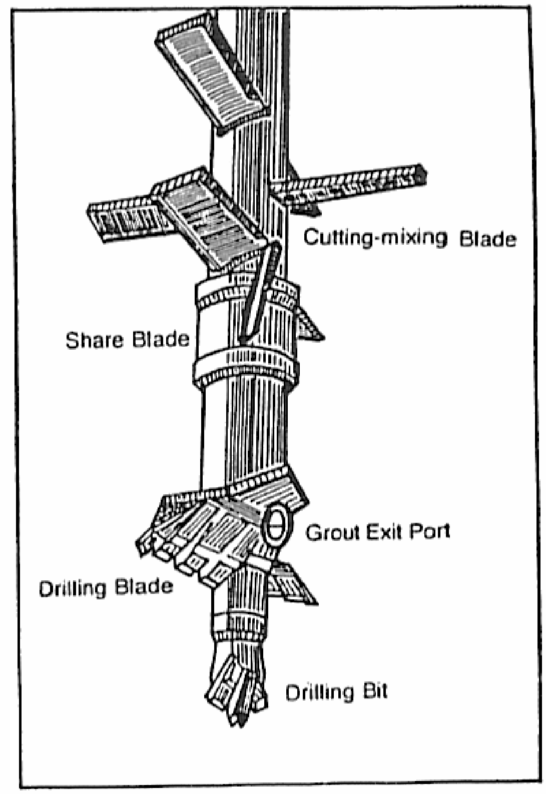
Schematic of SMW system (Taki and Bell, 1997)

Title	CDM Cement Deep Mixing	SMW Soil Mix Wall
Sketches of Repre-sentative Mixing Mechan-isms	 ø = 39" to 63" available 1 ft = 0.305 m	 ø = 22" to 40" available 1 ft = 0.305 m
Descriptions	Rotation of multiple axis shafts create relative movement and shear in soil for soil-reagent mixing	Uses multiple auger, paddle shafts rotating in alternating directions to mix in situ soil with cement grout or other reagents to form continuous soil-cement walls.

Comparisons of SMW and CDM shaft arrangements (Yang, 1997)

Appendix 2. Illustrations of various Deep Mixing Methods described in Appendix 1 (continues).

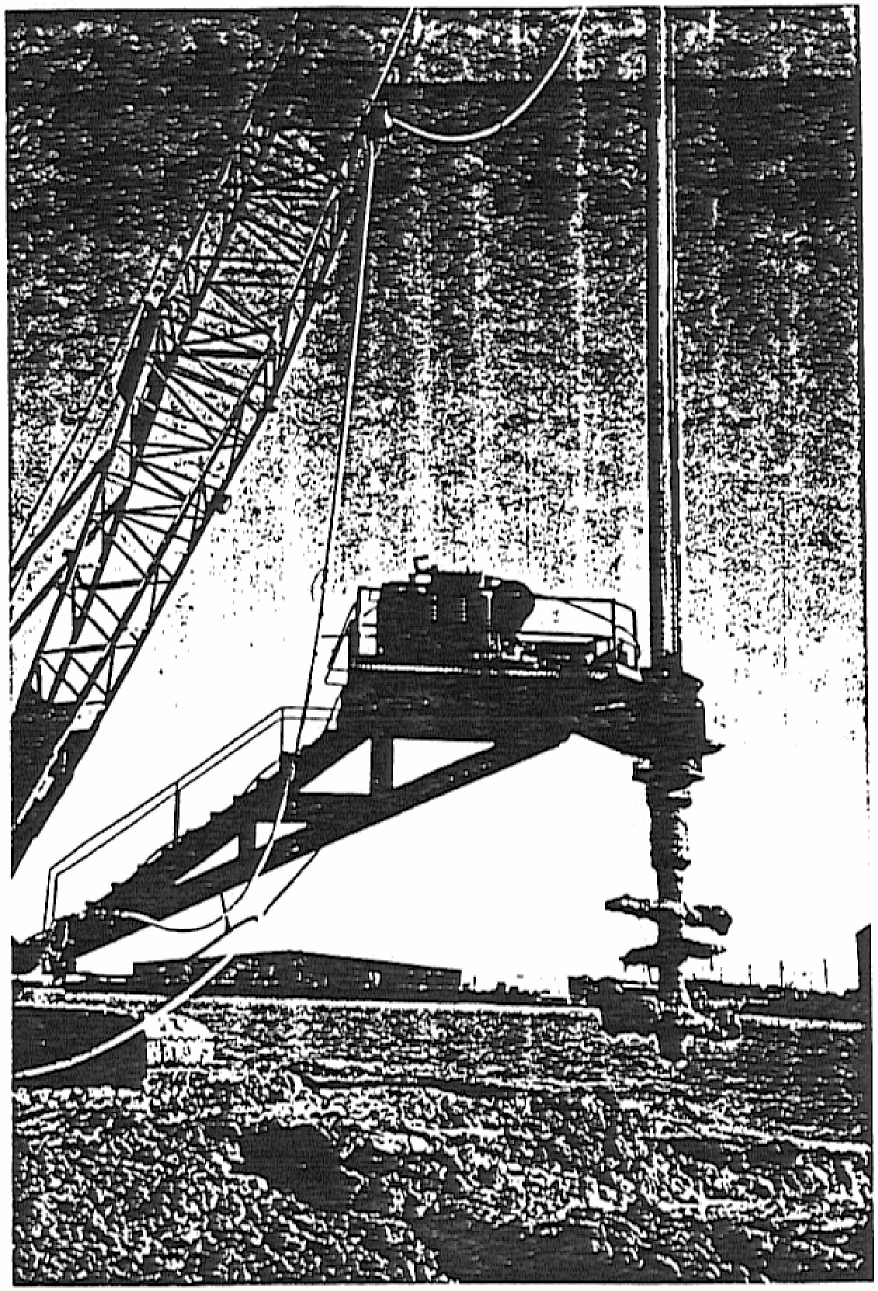




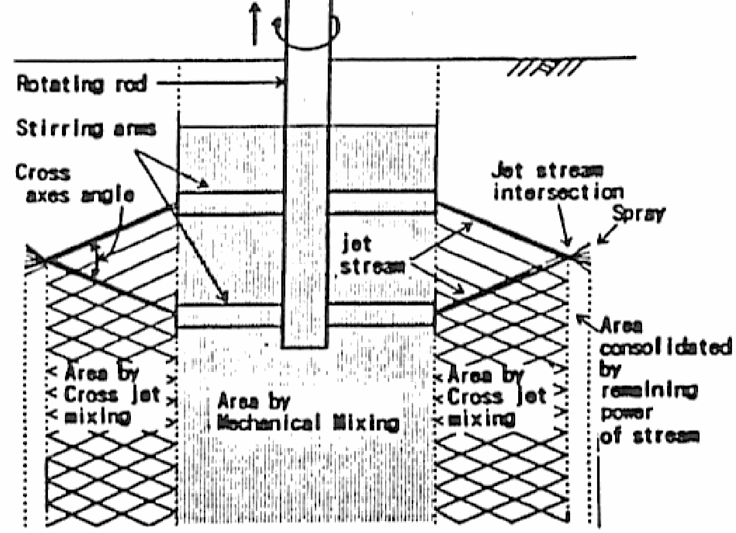
Details of SCC mixing tool (above)
(Taki and Bell, 1997).

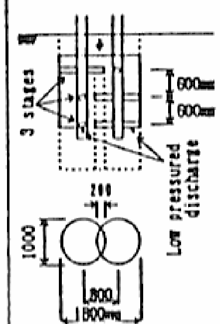
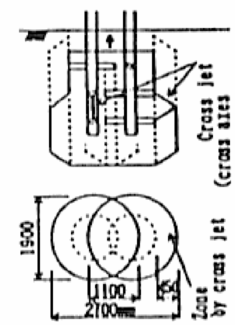
Details of Multimix (left) (Courtesy: Rodio)

Appendix 2. Illustrations of various Deep Mixing Methods



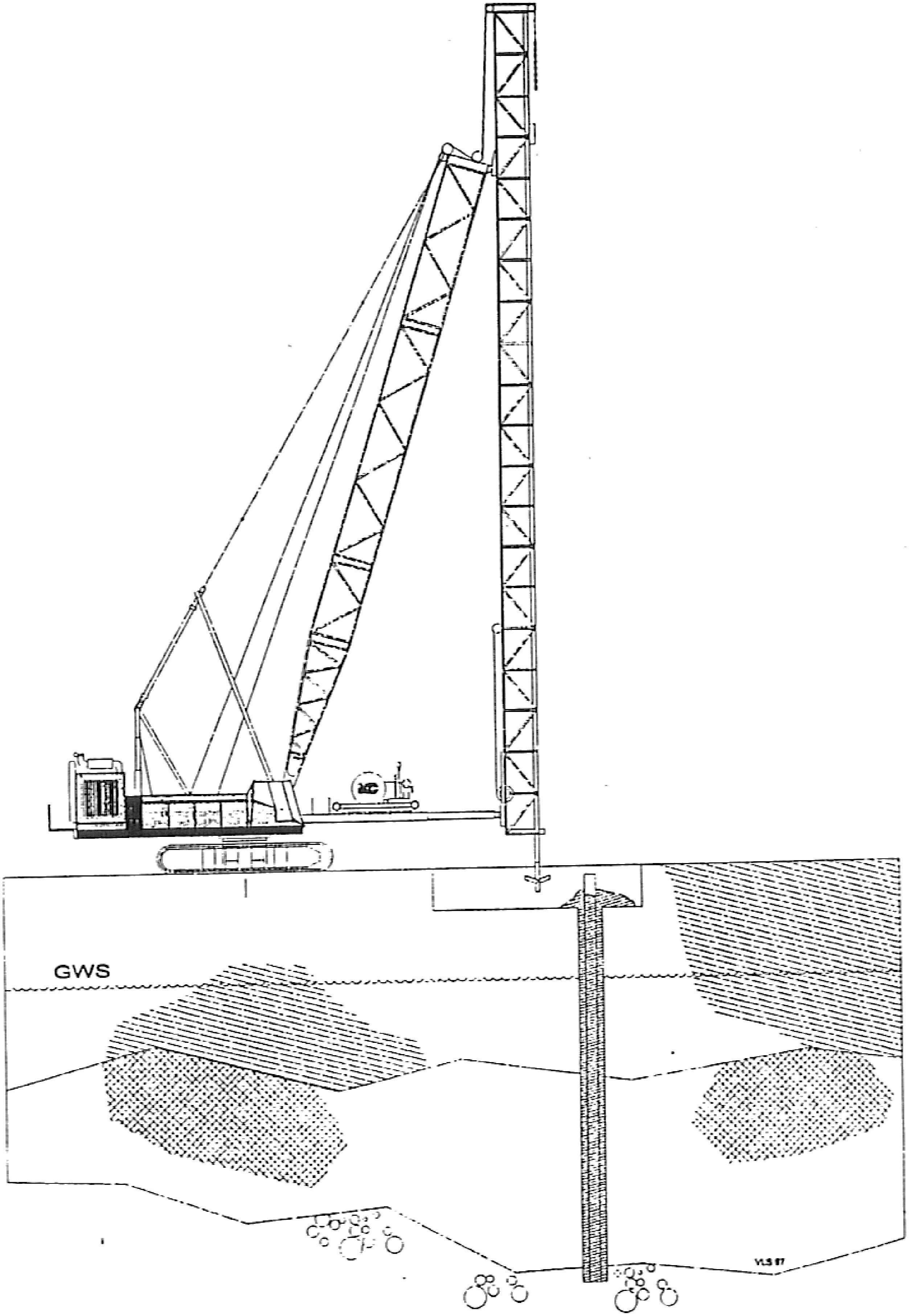
HBM tooling
(Courtesy: Hayward Baker, Inc.)



		Penetration (mechanical mixing)	Lifting (cross jet operated)
Method of work	2-rods, 80cm apart		
Elevation velocity		V = 1.0 m/min	V = 0.5 m/min
Rotation of rod		R = 20rpm	R = 20rpm
Diameter		ϕ_P = 1m(A_P =0.785m^2) × 2	ϕ_P = 1.9m(A_P =2.83m^2) × 2
Quantity of cement slurry		q=200 ℓ/min× 2	pressure q=150 ℓ/min × 4 nozzle P= 3000N/cm^2 (air vol:q=1m^3/min × 4 nozzle)

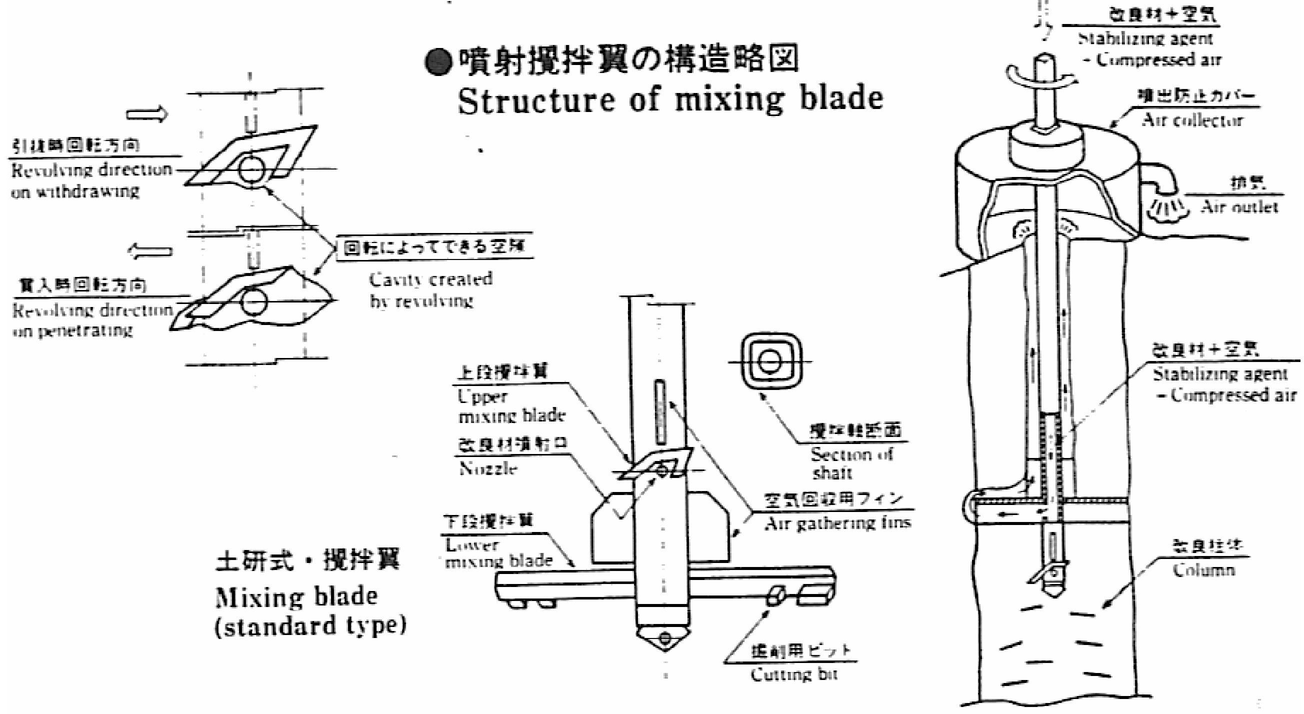
Details of JACSMAN system
(Miyoshi and Hirayama, 1996)

Appenc' ?. Illustrations of various Deep Mixing Methods

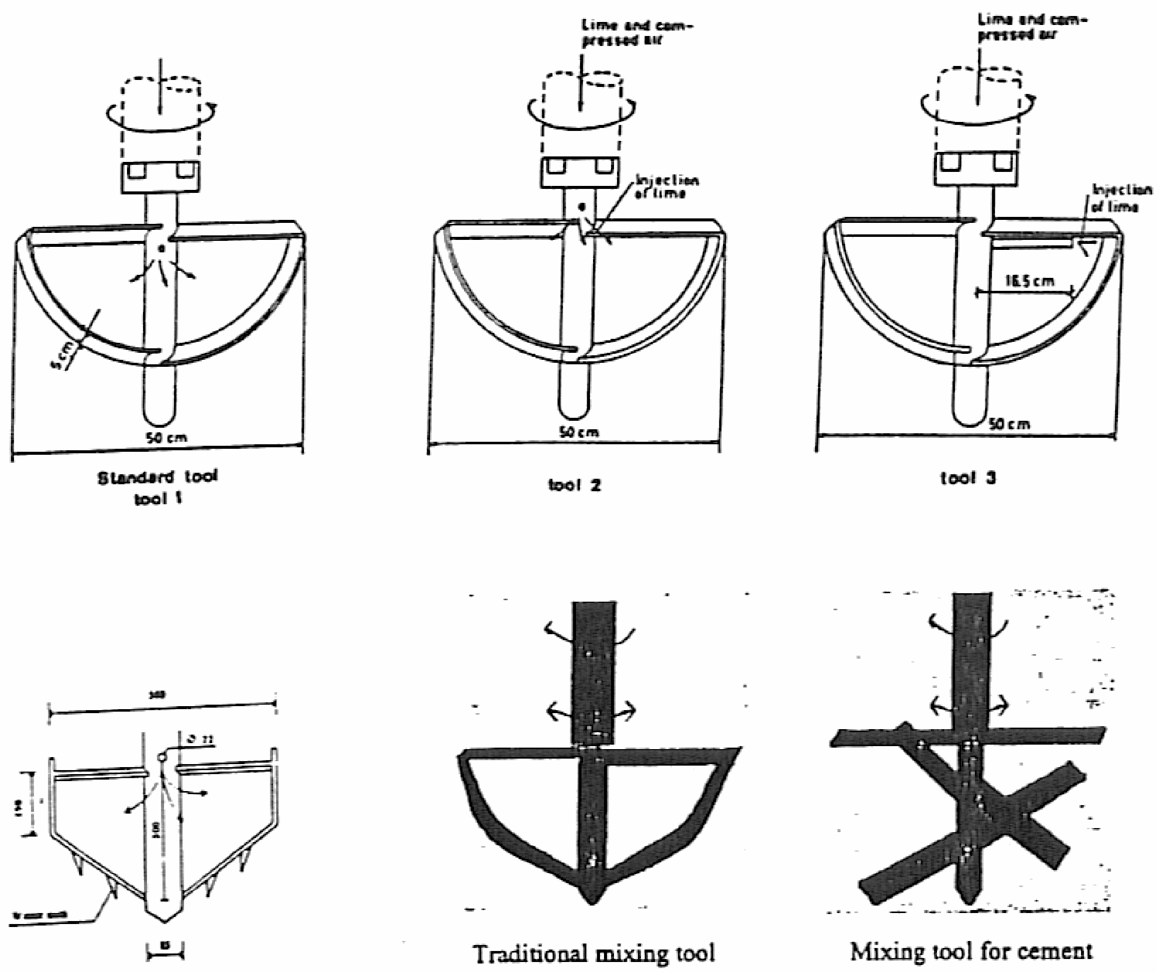


GeoJet system (Courtesy: Condon Johnson Associates)

Appendix 2. Illustrations of various Deep Mixing Methods described in Appendix 1 (continues).



Details of DJM system (DJM Association, 1994)



Details of Mixing tools for Lime-Cement columns (Courtesy: Stabilator)

Appendix 2. Illustrations of various Deep Mixing Methods described in Appendix 1 (concluded).