PB2002-100376

An Introduction to the Deep Mixing Methods as Used in Geotechnical Applications, Volume 3: The Verification and Properties of Treated Ground

FHWA-RD-99-167

October 2001

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
FHWA-RD-99-167		

4. Title and Subtitle	5. Report Date
AN INTRODUCTION TO THE DEEP MIXING METHODS AS USED IN GEOTECHNICAL APPLICATIONS – VOLUME III: THE VERIFICATION AND PROPERTIES OF TREATED GROUND	October 2001
	6. Performing Organization Code
7. Author(s) Donald A. Bruce, Ph.D.	8. Performing Organization Report No.
9. Performing Organization Name and Address ECO Geosystems, L.P. P.O. Box 237 Venetia, PA 15367	10. Work Unit No. (TRAIS)
	11. Contract or Grant No. DTFH61-95-2-00042
12. Sponsoring Agency Name and Address Office of Infrastructure Research and Development Federal Highway Administration 6300 Georgetown Pike McLean, VA 22101-2296	13. Type of Report and Period Covered Final Report 1997 - 2000
	14. Sponsoring Agency

15. Supplementary Notes

Contracting Officer's Technical Representative (COTR): A.F. DiMillio, HRDI-08

16. Abstract

The Deep Mixing Method (DMM) is an in situ soil treatment technology whereby the soil is blended with cementitious and/or other materials. This third report focuses closely on the properties of soils treated by DMM and aspects of quality control, quality assurance and verification. This report expands on the illustrative details previously provided in Volumes I and II. The engineering properties of treated soils, as reported in the international technical press, are reviewed together with the construction parameters and materials used to treat them. Sources of information include data from routine production tests, special laboratory and field tests, and more qualitative statements made in overview by specialists in the DMM technology. Methods of process control and verification of performance are also discussed since these are key issues in the minds of current and potential users.

This volume is the third in a series. The other volumes in the series are:

FHWA-RD-99-138 Volume I: An Introduction to the Deep Mixing Methods as Used in Geotechnical Applications
FHWA-RD-99-144 Volume II: An Introduction to the Deep Mixing Methods as Used in Geotechnical Applications: Appendices

17. Key Words	18. Distribution Statement
Deep mixing, verification, quality control and assurance, soil properties, state of the practice, testing	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	455	

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

TABLE OF CONTENTS

VOLUME 3

TABLE OF CONTENTS (Continued)

VOLUME 3

TABLE OF CONTENTS (Continued)

VOLUME 3

LIST OF FIGURES

VOLUME 3

LIST OF FIGURES (Continued)

VOLUME 3

LIST OF FIGURES (Continued)

VOLUME 3

LIST OF FIGURES (Continued)

VOLUME 3

LIST OF FIGURES (Continued)

VOLUME 3

LIST OF FIGURES (Continued)

VOLUME 3

LIST OF FIGURES (Continued)

VOLUME 3

LIST OF FIGURES (Continued)

VOLUME 3

LIST OF FIGURES (Continued)

VOLUME 3

LIST OF FIGURES (Continued)

VOLUME 3

LIST OF TABLES

VOLUME 3

LIST OF TABLES (Continued)

VOLUME 3

LIST OF TABLES (Continued)

VOLUME 3

ACKNOWLEDGMENTS

This volume has benefited from the support and input of a number of specialists in Deep Mixing, as did the earlier two volumes in this series. However, particular thanks are due to the following engineers

Jerry DiMaggio (FHWA)
Al DiMillio (FHWA)
Victor Elias (SaLUT)
Mel Esrig (Stabilator)
Göran Holm (Swedish Geotechnical Institute)
Masaaki Terashi (Nikken Sekkei, Japan)
Tom O'Rourke (Cornell University)
Mark Myers (USACOE)
David Yang (Raito, Inc.)

In addition, the author especially acknowledges the contributions of Jim Lambrechts of Haley and Aldrich, Inc., who drafted much of Chapters 2 and 6.

1. INTRODUCTION

1.1 Background

The two companion reports "An Introduction to the Deep Mixing Methods Used in Geotechnical Applications," and the associated appendices, were first submitted in April 1999. They were then reviewed, revised, and resubmitted in August 1999. The former was published in March 2000, and the latter is due for publication in mid 2000. The first report provides separate chapters on the historical development of the Deep Mixing Method (DMM), its applications, its relative competitiveness, a classification and description of the various DMM technologies, an international market review, and a discussion of barriers to market entry and limits to expansion within the United States.

At different points within both reports, the properties of soils treated by DMM were presented, largely to illustrate what can be achieved by each of the different techniques in various soils. In addition, aspects of quality control, quality assurance, and verification were similarly touched upon.

This third report, however, focuses more closely on these issues, and expands upon the illustrative details previously provided. The engineering properties of treated soils, as reported in the international technical press, are reviewed together with the construction parameters and materials used to treat them. Sources of information include data from routine production tests, special laboratory and field tests, and more qualitative statements made in overview by specialists in the DMM technology. Methods of process control and verification of performance are also discussed since these are key issues in the minds of current and potential users.

1.2 Synopsis of Previous Reports

1.2.1 Volume 1: An Introduction to the Deep Mixing Methods as Used in Geotechnical Applications

1.2.1.1 Introduction (Chapter 1)

The Deep Mixing Method (DMM) is defined as an in situ soil treatment technology whereby the soil is blended with cementitious and/or other materials. These materials are widely referred to as "binders" and can be introduced into the ground in dry or slurry form. They are injected through hollow, rotated mixing shafts tipped with some type of cutting tool. The shaft above the tool may be further equipped with discontinuous auger flights and/or mixing blades or paddles. Shafts are mounted vertically on a suitable carrier, usually crawler-mounted, and range in number from one to eight (typically two to four) per carrier, depending on the nature of the project, the particular variant of the method, and the contractor. Column diameters typically range from 0.6 to 1.5 m, and treatment may extend to 40-m depth or more. In some methods, the mixing action is enhanced by simultaneously injecting fluid grout at high pressures through nozzles in the mixing or cutting tools.

1.2.1.2 Historical Development (Chapter 2)

Eighty-two separate entries are recorded in chronological order to illustrate the development of the method in three continents over five decades.

Following an initial development in the United States in 1954, research into "contemporary" DMM began, apparently independently, in Japan and Sweden in 1967. These fundamental studies were largely sponsored and/or conducted by Governmental agencies and featured the injection of dry materials, largely unslaked lime. Research into "wet" methods (i.e., with slurry) began in 1972 and industrial scale projects began in 1975.

Intensive research and development proceeded apace in both countries: the Japanese moving into larger scale equipment and methods more suited to their huge marine and estuarine projects, the Nordic countries focusing more on in situ reinforcement of soft, often organic clays for road and rail projects. A Japanese method was demonstrated in the United States for the first time in 1986, and numerous other foreign contractors (including two from Sweden) followed, as did the development of several U.S. native variants, originally focusing on the environmental market.

European developments (outside of Scandinavia) also began in the mid 1980s, in France, Italy, and the United Kingdom, but despite their technical viability, have not generated major commercial activity application in that continent, or in other parts of the world where the European contractors operate, largely because the markets are less attractive for a number of reasons. It is fair to say that DMM today is a very powerful, flexible, ground engineering tool employed routinely in Japan, China, and other parts of Southeast Asia; Scandinavia and the neighboring Baltic countries; and the United States. There is an increasing number of English language papers and conference proceedings to supplement the traditional volumes in the Japanese, Swedish, and Finnish languages, and this will continue to encourage an increased use in a variety of applications throughout North America and in other parts of the world impacted by American engineers.

1.2.1.3 Applications (Chapter 3)

The various DMM techniques can be used to produce a wide range of treated soil structures (Figure 1), including single columns, rows of overlapping elements (walls or panels), grids or lattices, and blocks. The particular geometry chosen is dictated by the purpose of the application and reflects the mechanical capabilities and characteristics of the particular DM method employed.

The main groups of applications are as follows:

1. Hydraulic cut-offs: For embankment dams, levees, and other water-retaining structures including subsurface reservoirs. Applications have mainly been in Japan, but there have been a few significant projects in the United States and Europe to date.
2. Excavation support walls: Similar to hydraulic cut-offs except that they are often of higher strength (for durability) and are reinforced with vertical steel elements (to withstand lateral loadings). This is a common technique throughout Japan, China, and other parts of Southeast Asia, and in the United States.

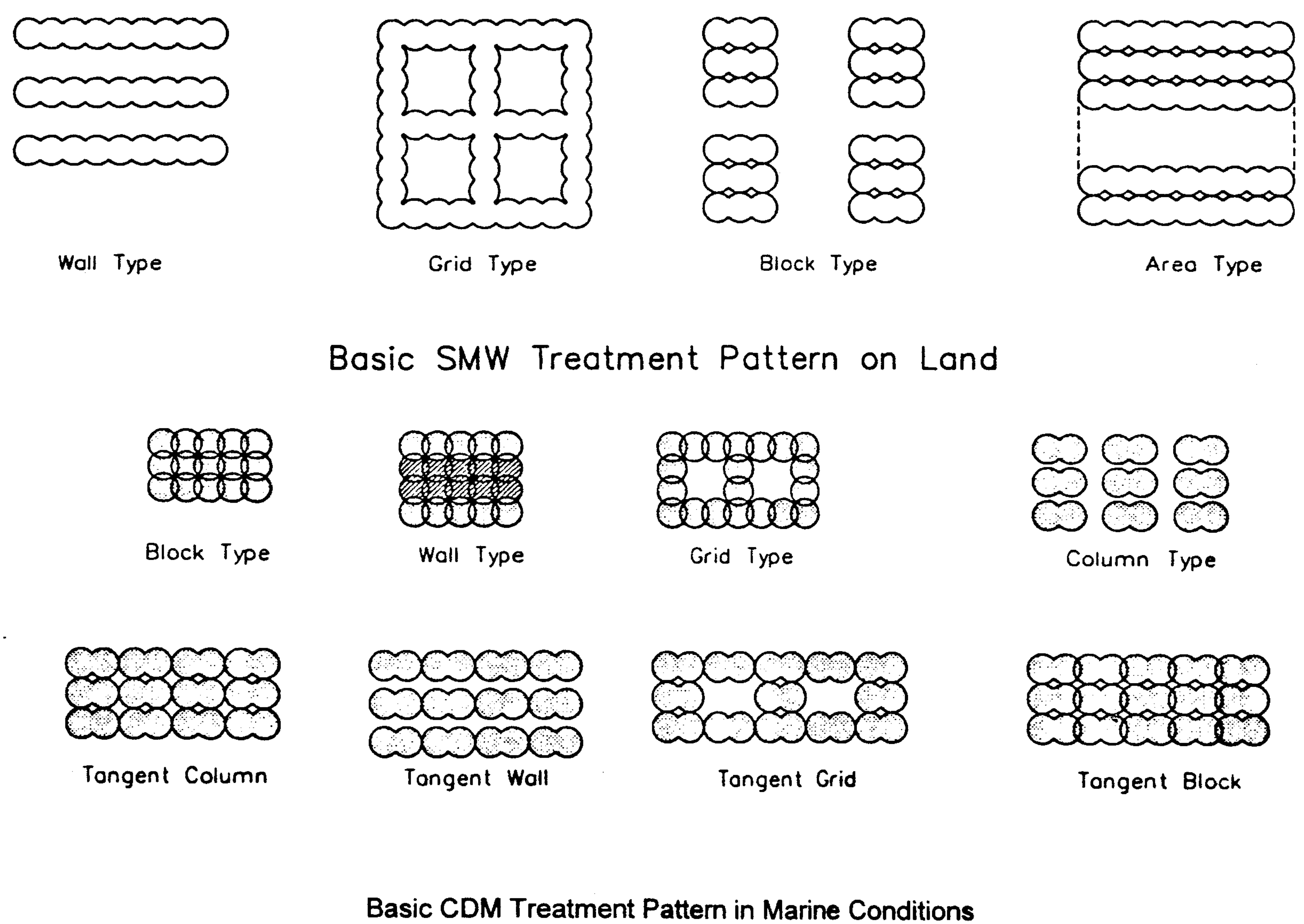


Figure 1. Basic deep mixing treatment patterns (Yang, 1997). (Note: Single columns can also be produced by many of the methods.)

3. Ground treatment: The Japanese developments were largely intended to provide economic treatment for huge volumes of soft soil or fill for marine and terrestrial tunneling and harbor/port developments. This remains a common application in Asia and North America.
4. Liquefaction mitigation: DMM can provide liquefaction prevention, reinforcement of liquefiable soil, or pore pressure reduction via block or lattice treatment patterns. There have been major applications in the coastal areas of Japan in particular. This principle was also used in the first major U.S. application of DMM in 1987, for a dam foundation in Wyoming.
5. In situ reinforcement (or Ground Improvement) and piles: DMM structures, usually in the form of closely spaced single columns, walls, or lattices have been widely used in Scandinavia, and less commonly in Japan, France, and the United States The major applications have been to reduce settlements under embankments, to improve slope stability, and to support light buildings and bridges. Most recently, in the United States, such principles have been used to create self-supporting DMM retaining walls (Nicholson and Jasperse, 1998).
6. Environmental remediation: DMM has been used to solidify, stabilize, and contain contaminated soils and sludges since 1988 in the United States. Increasing scope is afforded in other mature, industrialized countries including France and the United Kingdom.

1.2.1.4 The Application of DMM in Relation to Alternative Competitive Technologies (Chapter 4)

On any given project, the factors leading to the use of DMM are diverse, reflecting both a number of "hard" concerns, including geotechnical, logistical, accessibility, environmental, cost, schedule, and performance factors, as well as numerous less tangible issues including national and historical preferences, and the degree of influence and individual inclinations of the various contractors, consultants, and owners.

DMM is 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 when compared with competitive technologies. In the most general terms, DMM may be most attractive in projects where the ground is neither very stiff nor very dense, does not contain boulders or other obstructions, where treatment depths of less than about 40 m are required, where there is relatively unrestricted overhead clearance, where a constant and good supply of economic binder can be ensured, 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, and/or where treated ground strengths have to be closely engineered (typically in the range of 0.2 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, lightweight fills, compaction grouting, deep dynamic consolidation, bioremediation, vapor extraction, or simply removal and replacement.

The chapter contains tables that detail the relative advantages and disadvantages of DMM for each of the six broad types of applications noted above.

1.2.1.5 Classification and Description of the Various Deep Mixing Methods (Chapter 5)

Research to 1998 had located a total of 24 different variants of DMM at various stages of development or use throughout the world. These variants are listed in accordance with a newly developed classification system (Figure 2), based on the following fundamental operational characteristics:

- The method of introducing the "binder" into the soil: <u>W</u>et (i.e., pumped in slurry or grout form), or blown in pneumatically in <u>D</u>ry form.
- The method used to penetrate the soil and/or mix the agent: purely by <u>R</u>otary methods with the binder at relatively low pressure, or by a rotary method aided by <u>J</u>etting of fluid grout at much higher pressure. (Note: conventional jet grouting, which does not rely on any rotational mechanical mixing to create the treated mass, is beyond the scope of this report.)
- The location, or vertical distance over which mixing occurs in the soil – in some systems, the mixing is conducted only at the distal <u>E</u>nd of the shaft (or within one column diameter from that end), while in the other system mixing occurs along all, or a significant portion, of the drill <u>S</u>haft.

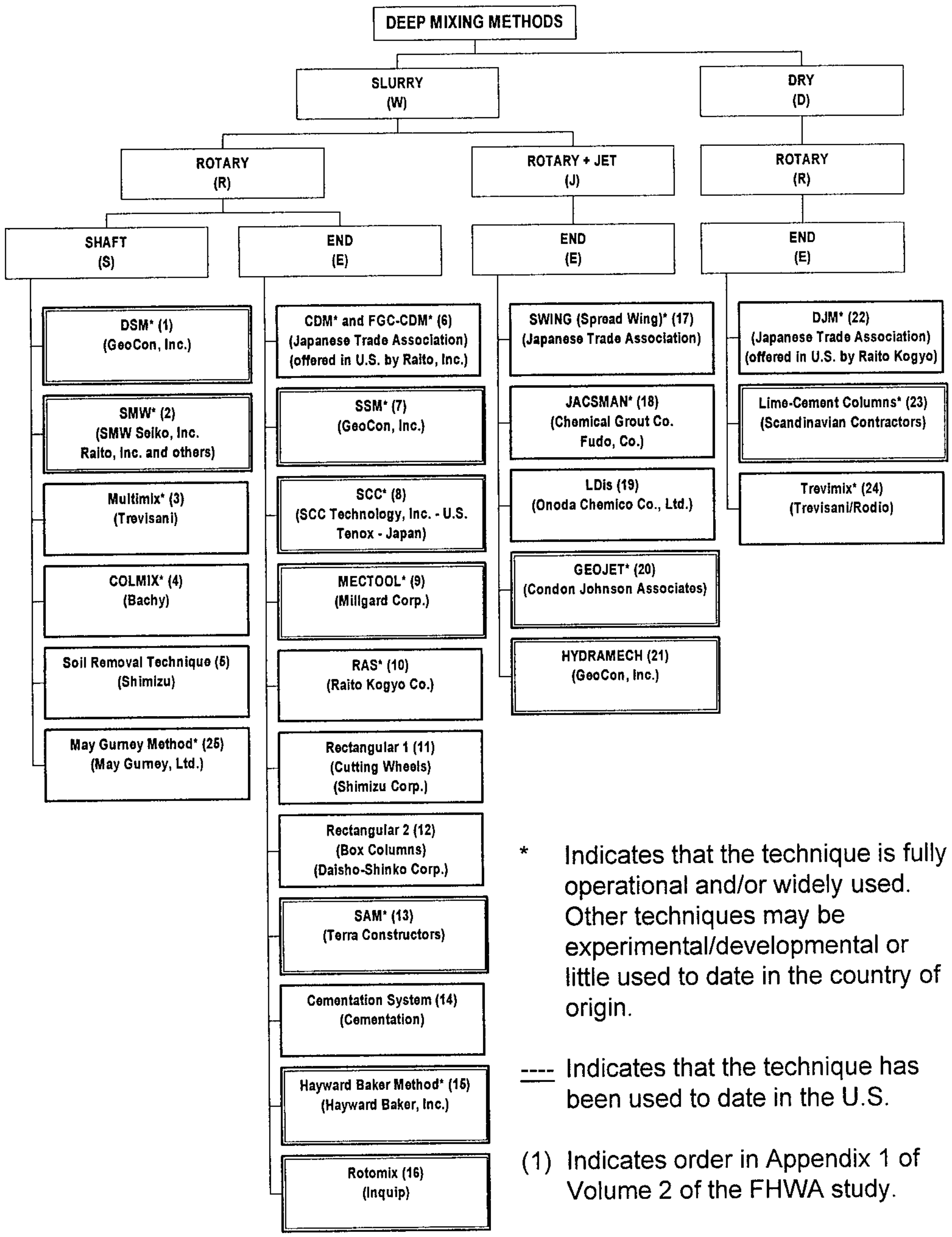


Figure 2. Classification of Deep Mixing Methods based on "binder" (Wet/Dry); penetration/mixing principle (Rotary/Jet); and location of mixing action (Shaft/End).

With three bases for differentiation, each with two options, there are theoretically eight different classification groups. However, in practice, there are only four groups since wet slurry, jetted shaft mixing (WJS) and dry binder, rotary, shaft mixing (DRS) do not exist, and no jetting with dry binder (DJS or DJE) has been developed.

While many of the systems shown in Figure 2 are fully operational, others remain in the experimental or developmental stages. A tabular synopsis of these 24 methods is provided in Table 1. Since the formulation of this table, other systems have become publicized in Europe: Bauer (Germany) operates a WRS system (for walls and cut-offs), and May Guerney (United Kingdom) has developed a WRS/WRE system for environmental applications while Keller Colcrete has applied the Hayward Baker WRE system, again mainly for environmental projects. In the United States, Trevi ICOS has acquired the license for GeoJet (WJE) in the Eastern states and has a cooperation agreement with the Swedish company Hercules for Lime Cement Columns nationwide (DRE). Most recently Schnabel Construction Company has used a WRS system, primarily to construct earth retaining structures.

1.2.1.6 International Market Review (Chapter 6)

A commercial review was conducted of each of the major geographic markets, i.e., United States, Japan, and Scandinavia.

In the United States, 11 different companies were identified as conducting, or having the potential to conduct, some type of DMM. These represented both native developments and holders of foreign licenses. The total annual market volume appears to be growing quickly and is currently estimated to be $50 to $80 million in geotechnical applications.

Japan remains the most active country, with output figures well documented by their powerful trade "associations." These groups each comprise dozens of contractors, suppliers, and manufacturers and help develop and publicize technological developments. It may be estimated that the annual volume is between $250 and $500 million, with a major growth "spurt" in the early and mid 1980s.

Table 1. 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 x 10^{-7} to 1 x 10^{-9} m/s	1 x 10^{-7} to 1 x 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.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		Multimix (Trevimix) 3	Colmix 4
Classification		W-R-S	W-R-S
Company		Trevisani	Bachy
Geography		Italy, U.S.	Europe
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.	Counter-rotating mixing shafts from fixed leads penetrate ground while slurry is injected. Blended soil moves from bottom to top of hole during penetration, and reverses on withdrawal. Restroking of columns in cohesive soils.
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.	6 to 8 auger machines noted in Australian patent (1995). Changing direction of augers during extraction compacts columns. Patented in U.S. 4,662,792 (1987). Automatic drilling parameter recorder synchronizes rate of slurry injection with penetration rate.
Details of Installation	Shafts	1-3, typically 3. Configuration varies with soil.	2, 3, or 4 common (6-8 possible)
	Diameter	0.55-0.8 m at 0.4 - 0.6-m spacings	0.23 to 0.85 m
	Realistic max. depth	25 m	20 m (10 m common)
	rpm	12-30	NA*
	Productivity/ output	0.35-1.1 m/min penetration (typically 0.5) 0.48-2 m/min withdrawal	0.8 m/min penetration; 1.0 m/min withdrawal; 200-300 m/shift
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement grout mainly, plus bentonite in sands; additives common, even in predrilling phase	Cement, lime, flyash, and special grouts to absorb heavy metals and organics
	w/c ratio	Typically low, i.e., 0.6-1.0 (especially in cohesives)	1.0 typical, but wide range
	Cement factor (kg_{cement}/m^3_{soil})	200-250 kg/m^3 typical (80-450 kg/m^3 range)	Up to 320 kg/m^3 (200 kg/m^3 typical)
	Volume ratio ($Vol_{grout}:Vol_{soil}$)	15-40%	30-50%
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	3-4 MPa (clay), higher for sands
	k	$< 1 \times 10^{-8}$ m/s	$< 1 \times 10^{-7}$ m/s
	E	ND*	50 to 100 x U.C.S.
Specific Relative Advantages and Disadvantages		Goals are to minimize spoils (10-20%) and presence of unmixed zones within and between panels	Low spoil claimed. Can be used on slopes and adjacent to structures. Columns have 10-20% larger diameters than shafts due to compaction effect. Flexible equipment and mix design.
Notes		Developed jointly in 1991 by TREVI and Rodio.	Developed in France in late 1980s.
Representative References		Pagliacci and Pagotto (1994)	Harnan and Iagolnitzer, 1992

*ND = No data; NA = Not applicable.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		Soil Removal Technique 5	CDM 6
Classification		W-R-S	W-R-E
Company		Shimizu Corporation	More than 48 members of CDM Association in Japan
Geography		Japan	Japan, China
General Description of Most Typical Method		Upper continuous auger flights on fixed leads extract soil to ground surface during penetration. Lower mixing blades rotate and mix soil with injected slurry during withdrawal.	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		Continuous flight augers from drill tip to the ground surface remove soil to limit ground displacements and lateral stresses during mixing.	Comprises numerous subtly different methods all under CDM Association
Details of Installation	Shafts	2	2-8 (marine): 1-2 (land) (each with 4-6 blades) (12 have been used)
	Diameter	1-1.2 m	1-2 m (marine): 0.7-1.5 m (land)
	Realistic max. depth	40 m	70 m (marine): 40 m (land)
	rpm	ND*	20-30 (penetration); 40-60 (withdrawal)
	Productivity/ output	ND*	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*	Wide range of materials, including portland or slag cement, bentonite, gypsum, flyash, using fresh or seawater; plus various additives.
	w/c ratio	ND*	0.6-1.3, typically 1.0
	Cement factor ($kg_{cement}/m^3{}_{soil}$)	ND*	100-300 kg/m^3, typically 140 to 200 kg/m^3
	Volume ratio (Vol_{grout}:Vol_{soil})	ND*	20-30%
Reported Treated Soil Properties	U.C.S.	0.5 MPa (in soft silt) (70% of conventional DMM)	Strengths can be closely controlled, by varying grout composition, from < 0.5-4 MPa (typically 2-4)
	k	ND*	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		Reduces horizontal displacements and stresses imposed during mixing. Obviates need for pre-augering.	Vast amount of R&D information available. Specifically developed for softer marine deposits and fills, now also used for land-based projects.
Notes		Operational prototype stage. Possibly patented. *Assumed similar to CDM.	Association founded in 1977. Research initiated under Japanese Government (1967). Offered in the U.S. by Raito, Inc.
Representative References		Hirai et al., 1996	CDM (1996); Okumura (1996)

*ND = No data; NA = Not applicable.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		SSM 7	SCC 8
Classification		W-R-E	W-R-E
Company		Geo-Con, Inc.	SCC Technology, Inc.
Geography		U.S.	SCC (U.S.); Tenox (Japan)
General Description of Most Typical Method		Single large-diameter auger on hanging leads or fixed rotary table is rotated by bottom rotary table and slurry or dry binder is injected. Auger rotation and injection continue to bottom of treated zone. Auger rotation during withdrawal usually without injection.	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.
Special Features / Patented Aspects		Single large-diameter auger; cycling up and down is common to improve mixing efficiency.	Very thorough mixing via "share blade" action, which is patented.
Details of Installation	Shafts	1	Single with 3 pairs of rotated mixing blades plus "share blade." Double shafts are possible for ground stabilization; single shaft for piles.
	Diameter	1-4 m	0.6-1.5 m; 1.2 m for double shafts.
	Realistic max. depth	12 m	20 m max
	rpm	15	30-60
	Productivity/ output	500-1500 m^3 per shift	1 m/min penetration and withdrawal 100 m^2 of wall up to 400 m of piles/8-h shift
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement grout, bentonite, flyash, lime, and other additives for contaminant immobilization	Typically cement grout, but others, e.g., ash, bentonite, possible.
	w/c ratio	1-1.75	0.6-0.8 (clays) to 1.0-1.2 (sands)
	Cement factor (kg_{cement}/m^3_{soil})	200-400 kg/m^3	150-400 kg/m^3 cement
	Volume ratio (Vol_{grout}:Vol_{soil})	12-20%	25-35%
Reported Treated Soil Properties	U.C.S.	3.5-10 MPa in granular soils. 0.6-1.2 MPa common in high-water-content sludges.	3.5-7 MPa (sands) 1.3-7 MPa (cohesives)
	k	1 x 10^{-10} m/s possible.	1 x 10^{-8} m/s
	E	100 to 300 x U.C.S.	180 x U.C.S.
Specific Relative Advantages and Disadvantages		Can treat wide variety of contaminants, including creosote, tar, organics, petroleum, etc.	Low spoil with minimal grout loss claimed, due to low w/c and minimized injected volume. Very efficient mixing.
Notes		Mainly used for environmental applications to date, but increasing use in geotechnical field	Used since 1979 in Japan and 1993 in U.S.
Representative References		Walker, 1992; Day and Ryan, 1995; Nicholson et al., 1997	Taki and Bell (1997)

*ND = No data; NA = Not applicable.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		MecTool® 9	RAS Column Method 10
Classification		W-R-E	W-R-E
Company		Millgard Corporation	Raito Kogyo, Co.
Geography		U.S. and U.K.	Japan
General Description of Most Typical Method		For cohesive soils, grout is placed in pre-drilled hole in center of each element, and soil in the annulus of the tool is then blended with mixing tool. End mixing with grout injected through hollow kelly bar.	Large diameter, single-shaft, concentric double-rod system on fixed lead is rotated at high rpm into ground, and grout injected over zone to be treated. Unit cycled up and down through zone with or without additional grout injection.
Special Features / Patented Aspects		MecTool (U.S. Patent #5,135,058). Also Aqua MecTool (U.S. Patent #5,127,765), describes an isolation mechanism that encloses submerged mixing tool in remediation zone providing protection against secondary contamination.	Cutting blade on inner rod rotates in opposite direction from two mixing blades on outer rod. Slurry injection ports located at base of inner rod.
Details of Installation	Shafts	1	1
	Diameter	1.2-4.2 m max.	1.4 and 2.0 m (larger than typical CDM)
	Realistic max. depth	25 m max (typically less than 6 m)	24 m typical; 28 m possible.
	rpm	ND*	Up to 40 (in each direction)
	Productivity/ output	0.6 m/min	0.5 m/min penetration 1 m/min withdrawal
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement grouts including PFA and other materials ± proprietary additive to breakdown "plastic seals thereby enabling through-the-tool delivery"	Cement grout
	w/c ratio	ND*	0.8 (field trial)
	Cement factor ($kg_{cement}/m^3{}_{soil}$)	ND*	300 kg/m^3 (field trial)
	Volume ratio ($Vol_{grout}:Vol_{soil}$)	20-35% estimated range	33% (field trial)
Reported Treated Soil Properties	U.C.S.	0.8-2.5 MPa	1-6 MPa
	k	1 x 10^{-8} to 1 x 10^{-9} m/s	ND*
	E	ND*	ND*
Specific Relative Advantages and Disadvantages		"Excellent control of grout and spoil quantity"	Large-diameter auger speeds production, computer control and monitoring, uniform mixing. Specially useful in dense soils.
Notes		Mainly environmental applications to date.*Probably similar to SSM	*Assumed similar to CDM
Representative References		Millgard Corporation, 1998	Isobe et al., 1996

*ND = No data; NA = Not applicable.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		Rectangular 1 (Cutting Wheels) 11	Rectangular 2 (Box Columns) 12
Classification		W-R-E	W-R-E
Company		Shimizu	Daisho Shinko Corp.
Geography		Japan	Japan
General Description of Most Typical Method		A pair of laterally connected shafts with horizontal mixing blades and vertical vanes are rotated during penetration. Grout injection during penetration and/or withdrawal. Vertical vanes create rectangular elements.	Mixing shaft rotated, "box casing" conveyed (without rotation), and grout injected during penetration. Shaft is counter-rotated during withdrawal.
Special Features / Patented Aspects		Use of claw-like vanes to create rectangular columns; vanes may be patented. Inclinometer fixed to mixing unit to monitor verticality.	Use of box casing, which surrounds mixing tools and contains treated soil to create square or rectangular columns.
Details of Installation	Shafts	2	1 with 4 horizontal mixing blades
	Diameter	1-m x 1.8-m columns	1-m square box
	Realistic max. depth	15 m	ND*
	rpm	ND*	30 (shaft only)
	Productivity/ output	1 m/min penetration/withdrawal	0.5 m/min penetration 1 m/min withdrawal
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement grout	Cement grout
	w/c ratio	ND*	1.0-1.2
	Cement factor ($kg_{cement}/m^3{}_{soil}$)	ND*	150-400 kg/m^3
	Volume ratio ($Vol_{grout}:Vol_{soil}$)	ND*	ND*
Reported Treated Soil Properties	U.C.S.	ND*	1.2-4.2 MPa
	k	ND*	ND*
	E	ND*	ND*
Specific Relative Advantages and Disadvantages		Rectangular columns require less overlap than circular. Vertical flow during mixing, larger cross-sectional column area per stroke.	Square/rectangular columns require less overlapping than circular columns. Uniform mixing promoted.
Notes		Operational prototype stage. *Assumed similar to CDM	Operational prototype stage.*Assumed similar to CDM.
Representative References		Watanabe et al., 1996	Mizutani et al., 1996

*ND = No data; NA = Not applicable.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		Single Auger Mixing (SAM) 13	Cementation 14
Classification		W-R-E	W-R-E
Company		Terra Constructors	Kvaerner Cementation
Geography		U.S.	U.K.
General Description of Most Typical Method		Large-diameter mixing tool on hanging leads rotated, with slurry injection during penetration.	Single auger on fixed leads rotated during penetration. Auger cycled up and down through 1-m length five times, then raised to next 1-m increment. Repeat to surface. Injection upon penetration, cycling, and/or withdrawal
Special Features / Patented Aspects		Multiple-auger mixing capability (MAM) foreseen for deeper applications.	Combination of a short interrupted length of auger with smaller diameter continuous flights.
Details of Installation	Shafts	1	1
	Diameter	1-3.6 m	0.75 m (1 m also possible)
	Realistic max. depth	13 m max.	10+ m
	rpm	8-16	ND*
	Productivity/ output	380 m^3/8-h shift	0.5-0.67 m/min penetration/mixing
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement grout mainly, and other additives for oxidation/stabilization of contaminants.	Cement grout with or without flyash
	w/c ratio	0.75-1.0	0.4
	Cement factor (kg_{cement}/m^3_{soil})	ND*	60-130 kg/m^3
	Volume ratio ($Vol_{grout}:Vol_{soil}$)	10-20% by weight	Unknown
Reported Treated Soil Properties	U.C.S.	Varies dependent upon soil type; up to 3.5 MPa	5-10 MPa
	k	Similar to in situ soil	ND*
	E	ND*	ND*
Specific Relative Advantages and Disadvantages		Applicable in soils below water table. Environmental applications.	Low spoil, low heave potential, specific horizons can be treated, good in saturated ground where dewatering cannot be used.
Notes		Developed since 1995.	Not now apparently used in U.K. due to market conditions.
Representative References		Terra Constructors, 1998	Greenwood, 1987

*ND = No data; NA = Not applicable.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		HBM (Single Axis Tooling) 15	Rotomix 16
Classification		W-R-E	W-R-E
Company		Hayward Baker Inc., a Keller Co.	INQUIP Associates
Geography		U.S. (but with opportunities for sister companies worldwide)	U.S. and Canada
General Description of Most Typical Method		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).	Single rotating shaft and bit; grout injection
Special Features / Patented Aspects		Method proprietary to Keller.	Proprietary to INQUIP
Details of Installation	Shafts	Single with 2 or 3 pairs of mixing paddles above drill bit.	Single, rotating bit with paddles
	Diameter	0.5-3.5 m, typically 2.1 and 2.4 m	1.2 to 4.8 m
	Realistic max. depth	20 m max.	3-30 m (depends on auger diameter)
	rpm	20-25 (penetration); higher upon withdrawal	5-45
	Productivity/ output	0.3-0.5 m/min (penetration); faster upon withdrawal. In excess of 500 m^3/shift	ND*
Mix Design *(depends on soil type and strength requirements)*	Materials	Varied in response to soil type and needs	Cement
	w/c ratio	1-2 (typically at lower end)	0.8-2 typical
	Cement factor ($kg_{cement}/m^3{}_{soil}$)	150 kg/m^3	>100 kg/m^3
	Volume ratio ($Vol_{grout}:Vol_{soil}$)	15-30%	>15%
Reported Treated Soil Properties	U.C.S.	3.5-10 MPa (sands) 0.2-1.4 MPa (clays)	>0.1 MPa
	k	1×10^{-10} m/s possible	$< 1 \times 10^{-8}$ m/s typical
	E	ND*	ND*
Specific Relative Advantages and Disadvantages		Good mixing; moderate penetration capability; low spoils volume. Dry binder method also available.	Good penetration/mixing. Dry binder available for use in treating sludges.
Notes		In development since 1990. Commercially viable since 1997.	Developed in 1990, mainly used for environmental applications. Limited data.
Representative References		Burke et al., 1998	INQUIP Associates, 1998.

*ND = No data; NA = Not applicable.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		Spread Wing (SWING) 17	JACSMAN 18
Classification		W-J-E	W-J-E
Company		Taisei Corporation/Raito Kogyo Co., and others	Chemical Grout Co., Fudo Co., and others
Geography		Japan, U.S.	Japan
General Description of Most Typical Method		With blade retracted, 0.6-m diameter pilot hole is rotary drilled to bottom of zone to be treated. Blade expanded and zone is treated with rotary mixing to 2-m diameter and air jetting to 3.6 m diameter.	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
Special Features / Patented Aspects		Retractable mixing blade allows treatment of specific depths to large diameter. Concentric mechanically mixed and jet mixed zones are produced. Patented. Trade association.	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.
Details of Installation	Shafts	1	2 shafts at 0.8-m spacing each with 3 blades.
	Diameter	0.6-m pilot hole, 2.0-m (mechanical) to 3.6-m (jetted) column	1 m (blades at 0.8-m spacing along shaft)
	Realistic max. depth	40 m	20 m
	rpm	ND*	20
	Productivity/ output	0.03-0.1 m/min penetration	1 m/min penetration 0.5-1 m/min withdrawal
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement grout	Cement grout
	w/c ratio	ND*	1.0
	Cement factor ($kg_{cement}/m^3{}_{soil}$)	450 kg/m^3	200 kg/m^3 (jetted); 320 kg/m^3 (DMM). Air also used to enhance jetting
	Volume ratio ($Vol_{grout}:Vol_{soil}$)	ND*	200 L/min per shaft during DM penetration; 300 L/min per shaft during withdrawal (jetting); i.e., 20-30%
Reported Treated Soil Properties	U.C.S.	0.4-4.4 MPa (mechanically mixed zone); 1.5 MPa (sandy), 1.2 MPa (cohesive) (jet-mixed zone)	2-5.8 MPa (silty sand and clay) 1.2-3 MPa (silty sand)
	k	1 x 10^{-8} m/s	ND*
	E	150 x U.C.S. (mechanically mixed zone); 100 x U.C.S. (jet-mixed zone)	ND*
Specific Relative Advantages and Disadvantages		Variable column size generated by varying pressures; retractable/expandable blade, jet mixing allows good contact with adjacent underground structures in difficult access areas.	New system combining DMM and jet-grouting principles to enhance volume and quality of treatment; jetting provides good overlap between columns.
Notes		SWING Association with 17 members established in late 1980s in Japan.	Name is an acronym for Jet and Churning System Management.
Representative References		Kawasaki et al., 1996; Yang et al., 1998	Miyoshi and Hirayama (1996)

*ND = No data; NA = Not applicable.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		LDis 19	GeoJet 20
Classification		W-J-E	W-J-E
Company		Onoda Chemical Co., Ltd.	Condon Johnson and Associates
Geography		Japan	Western U.S.
General Description of Most Typical Method		The mixing tool is rotated to full depth. Tool is withdrawn (rotating) to break up and remove the soil, followed by re-penetration to full depth. Grout is injected during second withdrawal via jets, at high pressure.	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		Conventional jet grout equipment with addition of single-blade auger to reduce volume of material displaced by jet and, therefore, limit ground movement (i.e., make volume injected equal to volume removed).	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	1	1 shaft with pair of wings or similar "processor"
	Diameter	About 1.0 m (jetted)	0.6-1.2 m
	Realistic max. depth	20 m	45 m max (25 m typical)
	rpm	3-40	150-200 (recent developments focusing on 80-90 rpm)
	Productivity/ output	0.33 m/min penetration. Overall, about 65% that of jet grouting.	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	ND*	0.5-1.5 (typically 0.8-1.0)
	Cement factor (kg_{cement}/m^3_{soil})	ND*	150-300 kg/m^3
	Volume ratio (Vol_{grout}:Vol_{soil})	About 40%	20-40%
Reported Treated Soil Properties	U.C.S.	2 MPa	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		Re-penetration causes production to be low. Spoil volume approximately equal to injected volume. Minimal ground heave.	Computer control of penetration parameters excellent. High strength. Low spoil volumes. High repeatability. Excellent mixing. High productivity.
Notes		Operational prototype stage. *Assumed similar to conventional jet grouting.	Developed since early 1990s. Fully operational in Bay Area. Five patents on "processor," system, and computer control; three patents pending.
Representative References		Ueki et al., 1996	Reavis and Freyaldenhoven (1994)

*ND = No data; NA = Not applicable.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		Hydramech 21	Dry Jet Mixing 22
Classification		W-J-E	D-R-E
Company		Geo-Con, Inc.	DJM Association (64 companies)
Geography		U.S.	Japan
General Description of Most Typical Method		Drill with water/bentonite or other drill fluid to bottom of hole. No compressed air used. At bottom, start low-pressure mechanical mixing through shaft. Cycle three times through bottom zone. Multiple high-pressure jets started at same time (350-450 MPa).	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.
Special Features / Patented Aspects		2-mm-diameter "hydra" nozzles on outer edges of mixing tool. Mechanical mixing occurs in center of columns, chunks of soil forced to perimeter where disaggregation occurs by jets.	System is patented and protected by DJM Association. Two basic patents (blade design and electronic control system). Many supplementary patents.
Details of Installation	Shafts	1	1-2 shafts adjustably spaced at 0.8 to ~1.5 m, each with 2-3 pairs of blades
	Diameter	1.2-m paddles on 0.9-m auger; column up to 2-m diameter, depending on jet effectiveness.	1 m
	Realistic max. depth	20+ m	33 m max.
	rpm	10-20	24-32 during penetration. Twice as high during withdrawal.
	Productivity/ output	Up to 500 m^3/shift	0.5 m/min penetration; 3 m/min withdrawal.35-45% lower in low-headroom conditions
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement	Usually cement, but quicklime is used in clays of very high moisture content
	w/c ratio	1.0-1.5	NA*
	Cement factor ($kg_{cement}/m^3{}_{soil}$)	100-250 kg/m^3	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)
	Volume ratio (Vol_{grout}:Vol_{soil})	10-15% by weight of soil.	NA*
Reported Treated Soil Properties	U.C.S.	Up to 10 MPa	Greatly varies depending on soil and binder, 1-10 MPa
	k	Up to 1×10^{-9} m/s	"Higher than CDM permeabilities"
	E	100 to 300 x U.C.S.	E_{50} = 50 to 200 x U.C.S.
Specific Relative Advantages and Disadvantages		No air used. Very uniform mixing. Control over diameters provided at any depth. Several times cheaper than jet grouting. Mixing can be performed within specific horizons, i.e., plugs can be formed instead of full columns.	Heavy rotary heads remain at bottom of leads, improving mechanical stability of rigs, especially in soft conditions. Very little spoils; efficient mixing. Extensive RandD experience. Fast production on large jobs.
Notes		Field-tested at Texas A&M. Fully operational from 1998.	Sponsored by Japanese Government and fully operational in 1980. (First application in 1981.) Offered in the U.S. by Raito, Inc. since 1998.
Representative References		Geo-Con, Inc., 1998	DJM Brochure (1996); Fujita (1996); Yang et al., 1998

*ND = No data; NA = Not applicable.

Table 1. Summary of mixing equipment and pertinent information for each technique (continued).

Name		Lime Cement Columns 23	Trevimix 24
Classification		D-R-E	D-R-E
Company		Various (in Scandinavia/Far East). Stabilator alone in U.S.	TREVI, Italy
Geography		Scandinavia, Far East, U.S.	Italy, Eastern U.S., Far East
General Description of Most Typical Method		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%.	Soil structure disintegrated during penetration with air. Augers are then counter-rotated on withdrawal and dry materials are injected via compressed air through nozzles on shaft below mixing paddles. Binder can also be added during penetration.
Special Features / Patented Aspects		Very low spoil. High productivity. Efficient mixing. No patents believed current. Strong reliance on computer control. Close involvement by SGI.	Use of "protection bell" at surface to minimize loss of vented dry binder. System is patented by Trevi and also used under license by Rodio. Needs soil with moisture content of 60-145+%, given relatively high cement factor and diameter.
Details of Installa-tion	Shafts	Single shaft, various types of cutting/mixing blades.	1-2 (more common). Separated by fixed (but variable) distance of 1.5-3.5 m.
	Diameter	0.5-1.2 m, typically 0.6 or 0.8 m	0.8-1.0 m (most common)
	Realistic max. depth	30 m max. (20 m typical)	30 m
	rpm	100-200, usually 130-170	10-40
	Productivity/ output	2-3 m/min (penetration) 0.6-0.9 m/min (withdrawal) 400-1000 lin m/shift (0.6 m diameter)	0.4 m/min penetration 0.6 m/min withdrawal 39 m/8-h shift
Mix Design *(depends on soil type and strength requirements)*	Materials	Cement and lime in various percentages (typically 50:50 or 75:25)	Dry cement (most common), lime, max. grain size 5 mm
	w/c ratio	NA*	NA*
	Cement factor ($kg_{cement}/m^3{}_{soil}$)	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	150-300 kg/m^3
	Volume ratio (Vol_{grout}:Vol_{soil})	NA*	NA*
Reported Treated Soil Properties	U.C.S.	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)	1.8-4.2 MPa (avg. 2.5 MPa)
	k	For lime columns, k = 1000 times higher than the k of the clay; for lime-cement columns, the factor is 400 to 500.	ND*
	E	50 to 200 x U.C.S.	1 to 2.66 x 10^3 MPa (clays) 3.125 x 10^3 MPa (sandy soils)
Specific Relative Advantages and Disadvantages		Same as for DJM. Excellent Swedish/Finnish research continues.	No spoil, uniform mixing, automatic control of binder quantity. System allows for "possibility of injecting water during penetration."
Notes		Developed by Swedish industry and Government, with first commercial applications in mid 1970s, and first U.S. application in 1996.	Developed by TREVI in Italy in late 1980s. Trevi-ICOS, U.S. licensee, in Boston, MA
Representative References		Holm (1994); Rathmeyer (1996)	Pavianni and Pagotto, 1991; Pagliacci and Pagotto, 1994

*ND = No data; NA = Not applicable

The Scandinavian market is estimated at $30 to $40 million per year, and is served by about 10 specialty contractors offering the Lime Cement Column method, historically developed for the local fine-grained saturated, organic soils. As in Japan, research and development continues apace, sponsored by industrial consortia headed by Governmental agencies. Strong market growth occurred from the late 1980s, although in recent years, growth rates have been more erratic due to national economic pressures.

1.2.1.7 Barriers to Market Entry and Limits to Expansion within the United States (Chapter 7)

The rate of growth in DMM use in the United States was relatively slow and irregular between 1986 and 1992, but has increased substantially since then. It is possible to identify several, often interrelated, factors that have conspired to act as barriers to market entry for prospective contractors, and as potential limits to market growth in the United States:

- Demand for DMM: the changing nature of the U.S. market, emphasizing urban and infrastructure redevelopment, usually in coastal areas with soft soils, high water table and/or seismic risk, favors and encourages the use of technologies like DMM.

- Awareness of DMM by Specifiers and Other Potential Clients: a wide range of specialty contractors and consultants are promoting DMM through a variety of media and so the level of awareness among the engineering public is increasing.

- Bidding Methods/Responsibility for Performance: performance specifications, design-build approaches, and other innovative procurement vehicles facilitate the introduction of "new technologies" such as DMM.

- Geotechnical Limitations: sites with boulders, very dense or stiff soils, fills with previously installed structures (including piles), and other obstructions are not best suited technically or economically. Treatment depths should be less than 40 m for most systems, and as shallow as 20 m for some.

- Technology Protection: most U.S. systems appear to be protected by patent or are operated under foreign license. This, plus the following factor, will act naturally to restrict the number of contractors who can conduct DMM.

- Capital Cost of Startup: whether the DMM system has been "home grown" or imported under license, the financial investment required to acquire, develop, promote, and run a DMM system tends to be relatively high. This automatically also restricts the number of contractors who can offer the technique.

As a final point, the chapter notes that fluctuations in the volume of DMM conducted in certain countries may be correlated to the incidence of natural disasters (e.g., the Kobe earthquake in 1995). On the other hand, it may be expected that a major technical reverse on a high profile project would have a correspondingly negative impact on market acceptance and level of DMM usage.

1.2.1.8 Final Remarks (Chapter 8)

This chapter observes that DMM is a well researched, well documented, vibrant technology with international reputation and application. However, it is essential for its continued growth in the United States that it is applied correctly, designed properly, constructed efficiently, and restricted sensibly to the natural restraints of soil conditions and equipment capability. If these conditions are observed, then DMM can become a commodity product in the geotechnical market, albeit a product provided by a relatively small number of contractors.

1.2.2 Volume 2: An Introduction to the Deep Mixing Method as Used in Geotechnical Applications (Appendices)

These appendices provide details on each DMM variant; observations on Scandinavian practice based on a visit to Finland and Sweden in May, 1997; a new joint venture for Lime Cement Columns work in the United Kingdom; an introduction to the Swedish Deep Stabilization Research

Center; an abstract from the new Finnish specifications for Lime Cement Columns; and details of the "Mass Stabilization" method used by the YIT Company, Finland.

2. BACKGROUND TO THE SOIL-BINDER CHEMICAL REACTIONS INDUCED BY DMM

2.1 Some Basic Definitions

DMM technology has been developed to remedy soils generally defined as "soft." Porbaha (1998) refers to "soft ground" as comprising cohesive soils with high moisture contents, and saturated fine granular deposits in a loose state. More specifically, soft ground has high compressibility, low strength, or a loose structure rendering it potentially liquefiable. As such, soft ground may be found to be: a fat clay with high plasticity, a silt with uniformly graded fines, a peat with several hundred percent water content, or a loose fine to medium sand of low density.

Figure 2 and Table 1 introduce a large number of terms, most of which – like penetration rate, rpm, and so on – are self-explanatory. However, the following terms merit clear definition at this point since they are key controls over treated soil properties:

Cement Factor (also known as the α factor): defined as the weight of dry binder introduced into the ground to be treated, divided by the volume of ground to be treated. The weight can refer to the weight of binder used in dry methods, or the weight of binder used in the slurry in wet methods. Expressed in units of kg/m^3. Alternatively, and often confusingly, the term a_w is also used, and this is the ratio of weight of dry binder to dry weight of soil (expressed as a percentage). The two definitions of cement factor are related but vary from each other, depending on the natural moisture content as shown in Figure 3.

Volume Ratio (abbreviated to V.R.): defined as the ratio of the volume of slurry injected (in wet method systems) to the volume of ground to be treated. Expressed as a percentage.

Binder: usually portland cement and (in the Scandinavian methods) unslaked (quick) lime. However significant research has been conducted on the use, in substitution or addition, of gypsum, flyash, coal slag, and "a variety of waste materials that have the ability to produce or

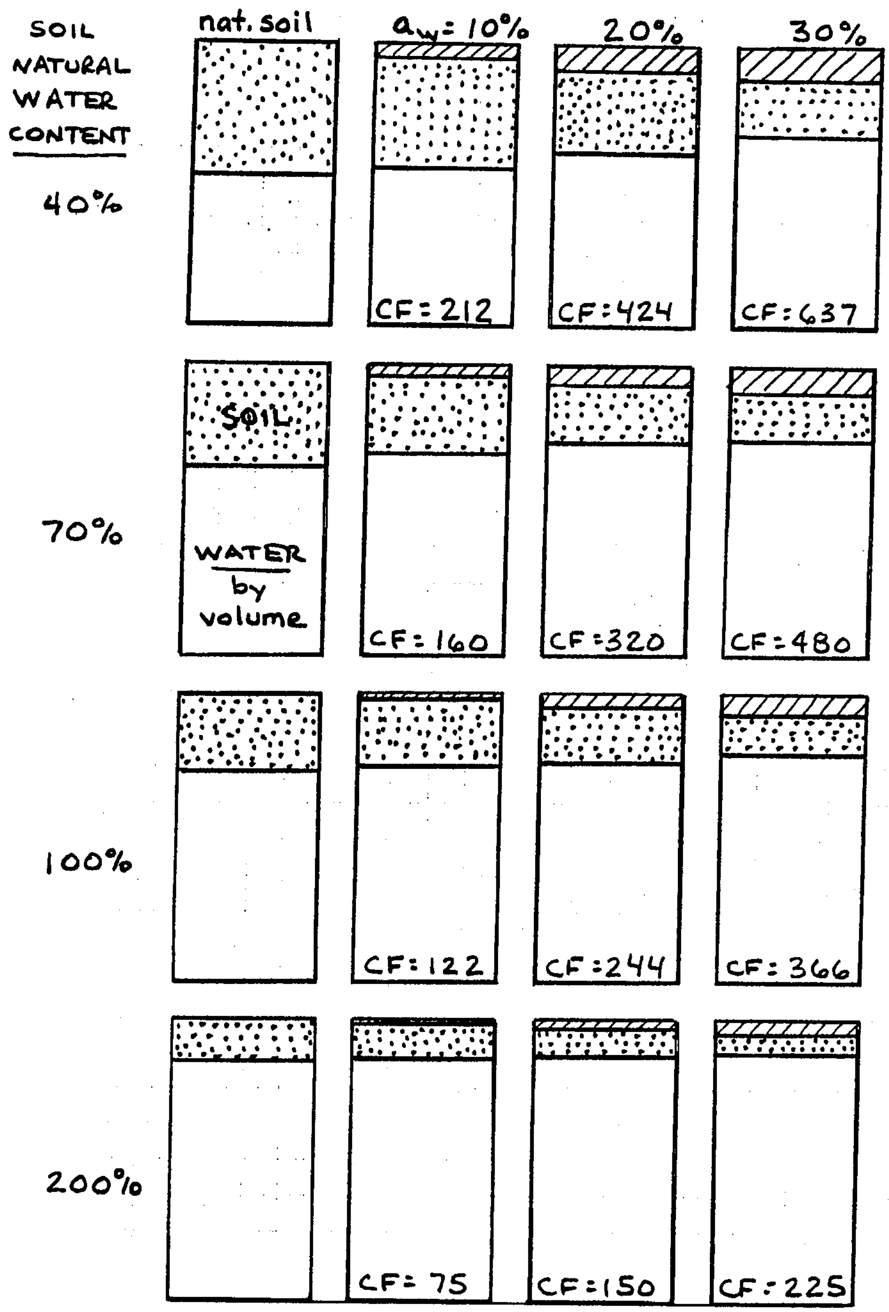


Figure 3. Phase diagrams showing the influence of moisture content on cement factor.

enhance pozzolanic reactions when mixed with soil and water, and also to reduce costs" (Esrig, 1997).

Pozzolan: can be defined as a finely divided silicaceous or aluminous material which, in the presence of water and calcium hydroxide, will form cemented products such as calcium or aluminate-silicate-hydrates. Clays can therefore contribute to the pozzolanic reactions believed to occur during cement and/or lime stabilization.

2.2 Overview of Soil-Cement Chemistry

The soil-cement material created by DMM is usually a mixture of soil, cement, and water. Although other binders, fillers, and admixtures can be substituted or added (Chapter 4), for simplicity this brief overview focuses only on the fundamentals of the soil, cement, and water system.

The soil composition, which can vary widely even within the boundaries of one site, has a major influence on the resultant product, as is clearly illustrated in this and subsequent chapters. This degree of control and influence of the inherent variability of soils on the composition of soil-cement and the degree of process control needed to produce the required materials properties is critical to bear in mind.

Whereas coarser grained soils and fills, comprising sands, are typically of simple and durable composition (e.g., quartz) and are therefore chemically inert during the mixing process, soils comprising cohesive materials are altogether more complex chemically. Their natural behavior results from their chemical composition, and their subsequent reactivity can be strongly affected by changes to their chemical environment. Because the chemical interactions between the clay particles themselves strongly influence the nature of the resulting soil-cement, it is appropriate to review some details of the composition of clays, the hydration of cement, and the pozzolanic and hardening reactions that occur in the formation of soil-cement in clay soils.

However, almost irrespective of whether clay or sand is being mixed, a key factor in determining the makeup of the resulting soil-cement is the efficiency of mixing that is imparted to the combined soil and cement (Chapter 5). The subsequent behavior of the soil-cement depends on the ability to distribute the cement throughout and within the soil mass, and the degree to which cement grains are distributed throughout the array of soil particles. The following sections dealing with soil and cement interactions assume that there is thorough distribution of cement throughout the soil, notwithstanding that this may not always be the case in practice.

2.3 Chemistry of Clay Soils

There are three principal compositions of elements that make up the majority of clay minerals, namely sheets of silica oxides, aluminum hydroxides, and magnesium hydroxides. The clay mineral particles are then formed as stacks of alternating sheets. The arrangement of sheets for the three predominant clay minerals, kaolinite, illite, and montmorillonite, is shown in Figure 4.

A large number of sheets stack up to form kaolinite, and resulting particles typically are 100 nm thick and 1000 nm in diameter, for a 1:10 thickness to width ratio. At the other extreme is montmorillonite, which is usually about 1 nm thick and 100 nm in diameter, for a 1:100 thickness to width ratio. As an example of the relative difference in particles, if the kaolinite were represented by a 4-ft by 8-ft sheet of ¼-in-thick plywood, then the montmorillonite would be a 3 by 5 in index card. The important point in this difference is the relative surface areas that the same weight of kaolinite and montmorillonite particles would possess, because it is on the surfaces of the clay minerals that cement reactions occur: montmorillonite is therefore far more reactive. It often also has Na^+ ions on the surface that exchange with Ca^{++} ions from the lime during the mixing process. Hence lime is effective in controlling montmorillonite behavior. Illite, by comparison, is rather inert, except when deposited in a marine environment, where Na^+ ions are incorporated.

Surrounding each clay particle is a layer of water molecules that is tightly held by clay surface attractive molecular forces. The chemical make-up of the surface layers of the clay minerals also has significant effect on the clay's reactivity with cement. Isomorphous substitution occurs in the

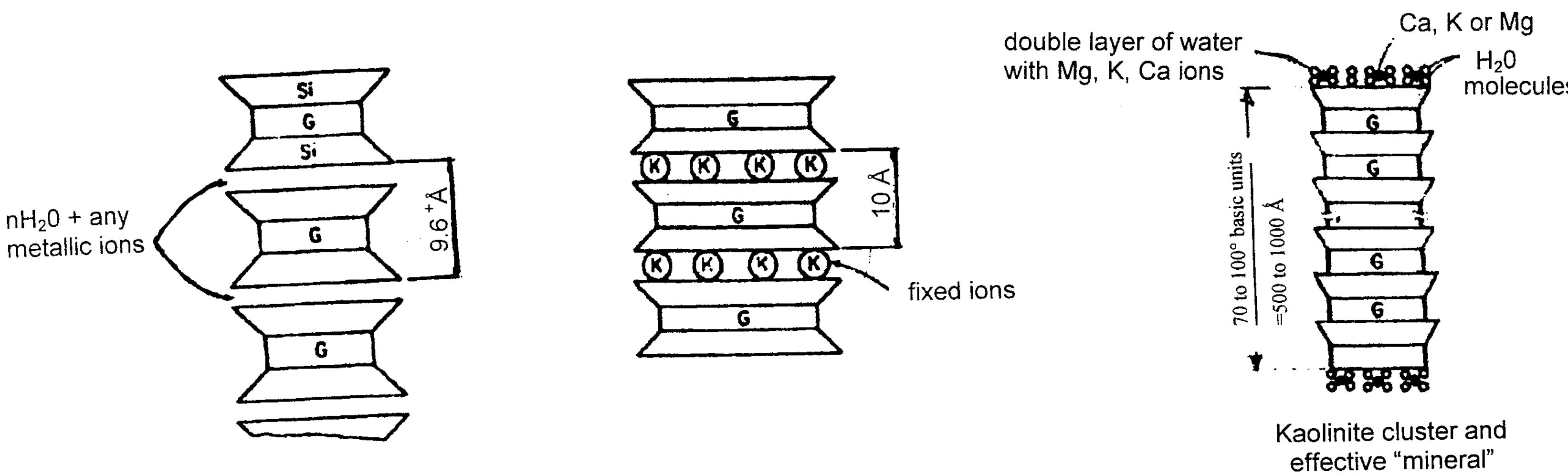
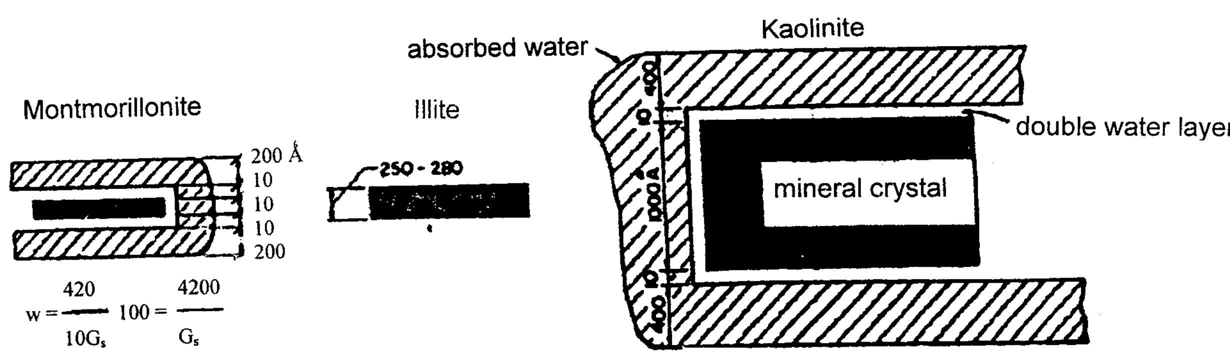


Plate Plan	0.1 to 1 μm	10 μm	0.3 to 4 μm
Specific Surface	800 m^2/g	80 m^2/g	15 m^2/g (statistical average)



$$w = \frac{420}{10G_s}\, 100 = \frac{4200}{G_s}$$

$$w = \frac{820}{1000G_s} = 100 = \frac{82}{G_s}\ \%$$

Figure 4. Relative sizes, absorption potential, relative range in water content, grain size, and specific surface area for all clay minerals (Das, 1983).

layers that result in substitution of some of silicon, aluminum, or magnesium atoms by other clay mineral surface atoms of lower valence (commonly by aluminum, magnesium, and iron, respectively). This substitution causes the flat surfaces to have negative charge, which attracts "exchangeable" cations in the pore fluid and results in the tightly bonded "double layer" of water that forms around the surface of each clay mineral particle. The chemical reactions of cement, water, and clay particles then directly interact with the clay surface elements and the particle's double layer water.

Within this framework, the reactions which lead to development of clay particle interaction with cement occur. For further information on clay mineralogy, the reference by Mitchell (1993) is recommended.

2.4 Chemistry of Cement Stabilized Clay Soils

Portland cement clinker is produced by burning a mix of calcium carbonate (limestone or chalk) and an aluminosilicate (clay or shale) and then grinding the product with approximately 5% gypsum to produce cement (Table 2). The significant levels of iron oxide in a gray cement are derived from the clay, as are the much lower levels of alkalis. The ASTM classification of ASTM Type 1 cement corresponds to the United Kingdom terminology "Ordinary Portland Cement" (OPC), CEMI (British Standard 12: 1996) and the European pre-standard ENV197.

As a simplification, Portland cement comprises primarily four compounds: Tricalcium Silicate, Bicalcium Silicate, Tricalcium Aluminate, and Tetracalcium Aluminoferrite. Bogue (1955) proposed the following chemical formulae representing the mass percentage of each oxide:

Tricalcium Silicate: $4.07CaO - 7.60SiO_2 - 1.43Fe_2O_3 - 6.72Al_2O_3$

Bicalcium Silicate: $8.60SiO_2 + 1.08Fe_2O_3 + 5.07Al_2O_3 - 3.07CaO$

Tricalcium Aluminate: $2.65Al_2O_3 - 1.69Fe_2O_3$

Tetracalcium Aluminoferrite: $3.04Fe_2O_3$

Table 2. Composition of portland cement (Bye, 1999).

	Cement*	Clinker*		Cement		Clinker	
	Grey: %	Black: %	White: %	Grey		Black	White
SiO_2	19 – 23	21.7	23.8	LSF%[‡]	90–98	98.4	97.2
Al_2O_3	3 – 7	5.3	5.0	LCF%[‡]	–	96.2	93.8
Fe_2O_3	1.5 – 4.5	2.6	0.2	S/R	2–4	2.7	4.6
CaO	63 – 67	67.7	70.8	A/F	1–4	2.0	25
MgO	0.5 – 2.5	1.3	0.08	C_3S%	–	65.4	59.4
K_2O	0.1 – 1.2	0.5	0.03	C_2S%	–	12.9	23.5
Na_2O	0.07 – 0.4	0.2	0.03	C_3A%	–	9.6	12.9
SO_3	2.5 – 3.5[†]	0.7	0.06	C_4AF%	–	7.9	0.6
LOI	1 – 3.0[†]	–	–				
IR	0.3 – 1.5[†]	–	–				
Free lime[§]	0.5 – 1.5	1.5[§]	2.5[§]				

* cement — usual range; clinkers — examples used in text
† upper limits in BS 12: 1996
‡ lime saturation and combination factors (Section 2.2)
§ also included in total CaO
LOI loss on ignition (CO_2 + H_2O) typically 0.8–1.8%
IR insoluble residue — usually siliceous and typically <1%
S/R silica ratio % SiO_2/(%Al_2O_3 + %Fe_2O_3)
A/F alumina ratio %Al_2O_3/%Fe_2O_3

The first two of these comprise about 75% of Type I portland cement. The reactions of these compounds begin with the addition of water, initiating hydration. Subsequent interactions of the resulting chemicals with the clay cause other reactions. Hardening occurs in two forms – the byproducts of cement hydration, and pozzolanic chemical reactions with clay. The various stages of the soil-cement reactions follow, and are illustrated in Figure 5.

2.4.1 Initial Hydration of Cement

When cement is mixed with water, hydration occurs in which the four primary constituents of portland cement combine with water (and with the free lime and gypsum that are present in cement) to form other compounds that will eventually also cause hardening of the soil-cement. These reactions are shown in Equations 1 through 5 (Bergado et al., 1996), which illustrate that considerable amounts of water are consumed. Esrig (1999) reports that cement needs 25% of its own weight of water, while unslaked lime needs 32%.

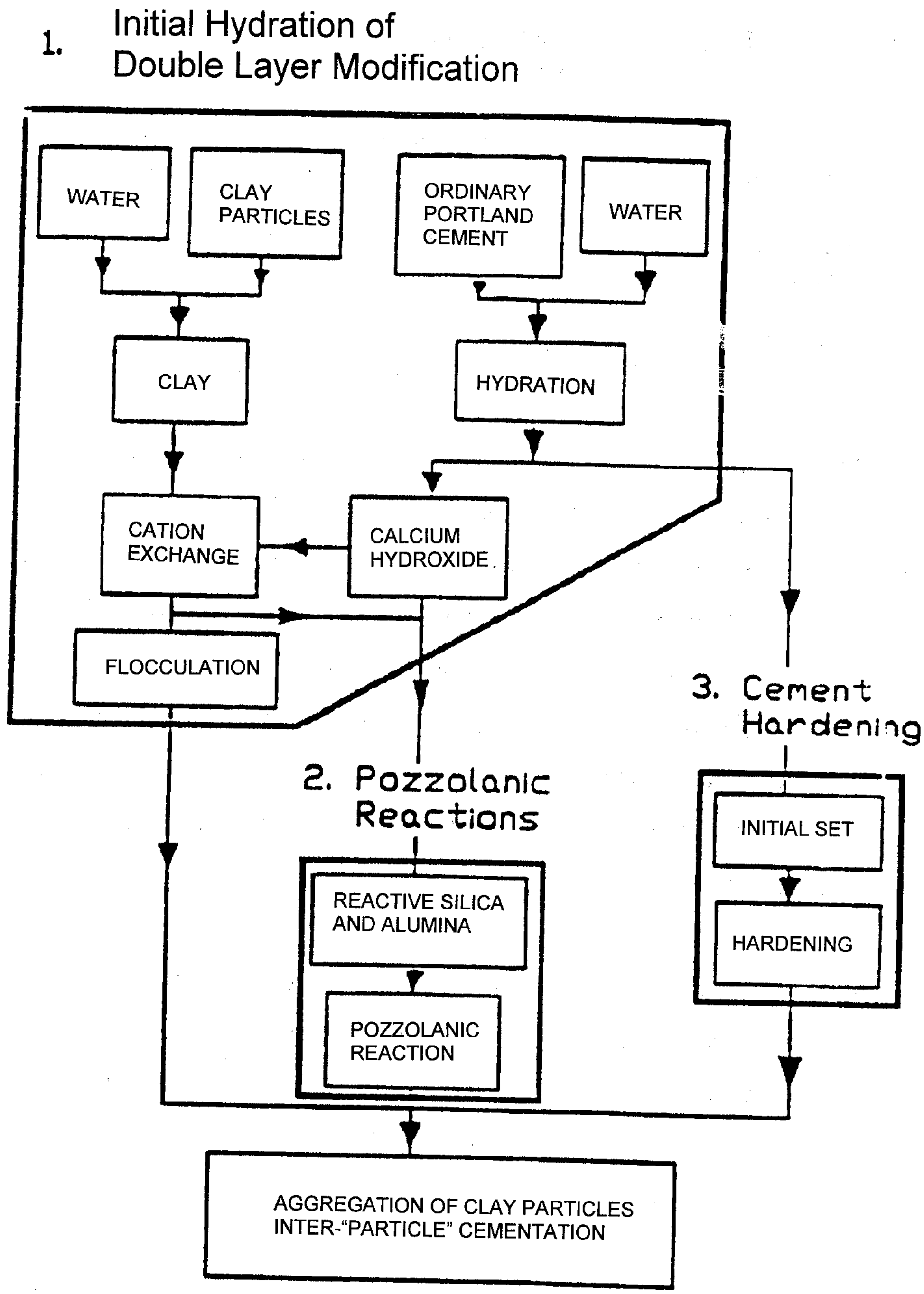


Figure 5. Process diagram of soil-cement reaction.

$$2(3CaO.SiO_2) + 6H_2O = 3CaO.SiO_2.3H_2O + 3Ca(OH)_2 \quad [1]$$
(tricalcium silicate) (water) (tobermorite gel) (calcium hydroxide)

$$2(2CaO.SiO_2) + 4H_2O = 3CaO.2SiO_2.3H_2O + Ca(OH)_2 \quad [2]$$
(bicalcium silicate) (water) (tobermorite gel) (calcium hydroxide)

$$4CaO.Al_2O_3.Fe_2O_3 + 10H_2O + 2Ca(OH)_2 = 6CaO.Al_2O_3.Fe_2O_3.12H_2O \quad [3]$$
(tetracalciumaluminoferrite) (calcium aluminoferrite hydrate)

$$3CaO.Al_2O_3 + 12H_2O + Ca(OH)_2 = 3CaO.Al_2O_3.Ca(OH)_2.12H_2O \quad [4]$$
(tricalcium aluminate) (tetracalcium aluminate hydrate)

$$3CaO.Al_2O_3 + 10H_2O + CaSO_4.2H_2O = 3CaO.Al_2O_3.Ca(OH)_2.12H_2O \quad [5]$$
(tricalcium aluminate) (gypsum) (calcium monosultoaluminate)

The tobermorite gel (the primary hydration product) forms on the surface of the cement grains and because it is adhesive, it stiffens the cement mixture and gives the "initial set" to cement based mixtures (i.e., concrete, soil-cement, or grout). With time, the gel will harden, and it is responsible for a large part of the mixture's strength. The tobermorite gel is usually identified as $C_3S_2H_x$ and is often referred to, simply, as CSH gel. However, for the purpose of this report, the terms tobermorite gel and $C_3S_2H_x$ are used to identify these primary cementitious products, and to avoid confusion with the secondary cementitious products discussed below. When sufficient quantity of cement is present, tobermorite gel that forms on the surface of cement particles will surround nearby clay particles (Figure 6).

The higher the cement content, the greater will be the adhesive effect of the CSH gel and the more marked the stiffening of the mixture. (This can impede machine shaft rotation, particularly when the mixing augers or paddles are in the soil-cement for a considerable time. In the case of mixing to great depth, a cement set retarder may be necessary.) The initial setting phenomenon is of course dependent on the amount of cement and clay minerals, and the water content.

Also released in the initial hydration reaction is a considerable amount of calcium hydroxide, $Ca(OH)_2$ (Equations 1 and 2), which chemically alters the porewater. Calcium ions are released by dissociation, as shown in Equation 6 below, and they replace cations in the double layer of water absorbed onto the clay particles, which leads in turn to reduced repulsive forces and easier

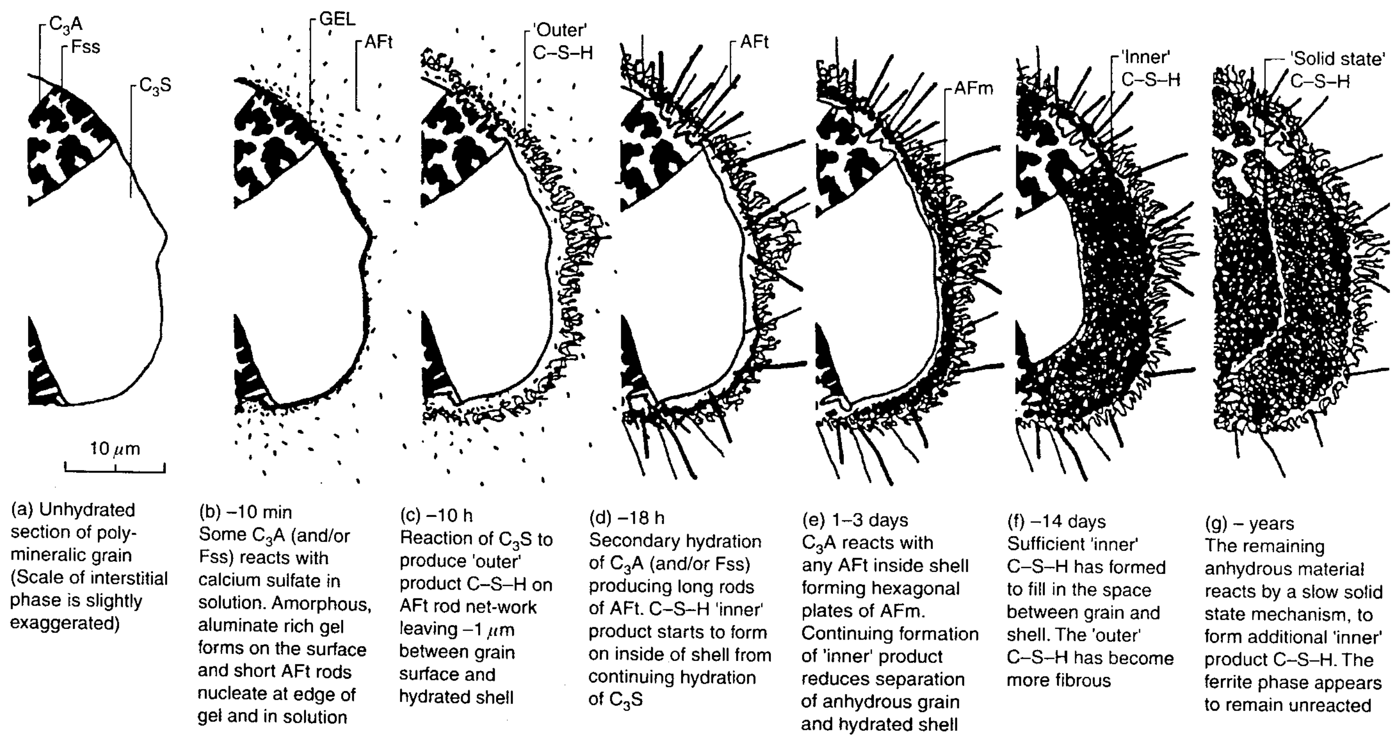


Figure 6. Hydration of a polymineralic grain of portland cement. Note the persistence of high iron content regions (Bye, 1999).

formation of flocculated structures. However, this is not thought to lead to increase in strength since the mixing process typically remolds the original soil particle structure.

$$Ca(OH)_2 \rightarrow Ca^{++} + 2(OH)^- \qquad [6]$$

2.4.2 Cement Hardening Reactions

The $C_3S_2H_x$ gel developed on the surfaces of cement particles forms hydrate crystals as it hardens, as simply illustrated in Figure 7. As the crystals grow outward from the cement grains, they will encounter other crystals growing out from other nearby cement grains, and link together to form a framework of hydrate crystals. This crystal framework will attach to soil particles, thus binding cement grains and soil particles together in a rigid structure. This is the fundamental cement paste hardening process.

However, not all of the tobermorite gel will have time to form hydrate crystals. A parallel reaction occurs on the surface of the clay minerals that will eventually lower the pH of the porewater environment. When the pH drops below 12.6, the $C_3S_2H_x$ undergoes hydrolysis (Equation 7 below) and then cannot hydrate into the insoluble salt. The remaining gel is then lost from the normal cement hardening process and forms a secondary cementitious product (much like that which occurs in the pozzolanic reaction described in Section 2.4.3 below). These secondary cementitious products provide much less strength than the hydrated CSH crystals (Equation 7).

$$C_3S_2H_x \rightarrow \underset{\text{(secondary cementitious product)}}{CSH + Ca(OH)_2} \qquad [7]$$

2.4.3 Pozzolanic Reactions

The calcium hydroxide ($Ca(OH)_2$) released by the cement hydration dissociates, as shown in Equation 8 below, and the hydroxyl ions cause the porewater pH to rise considerably.

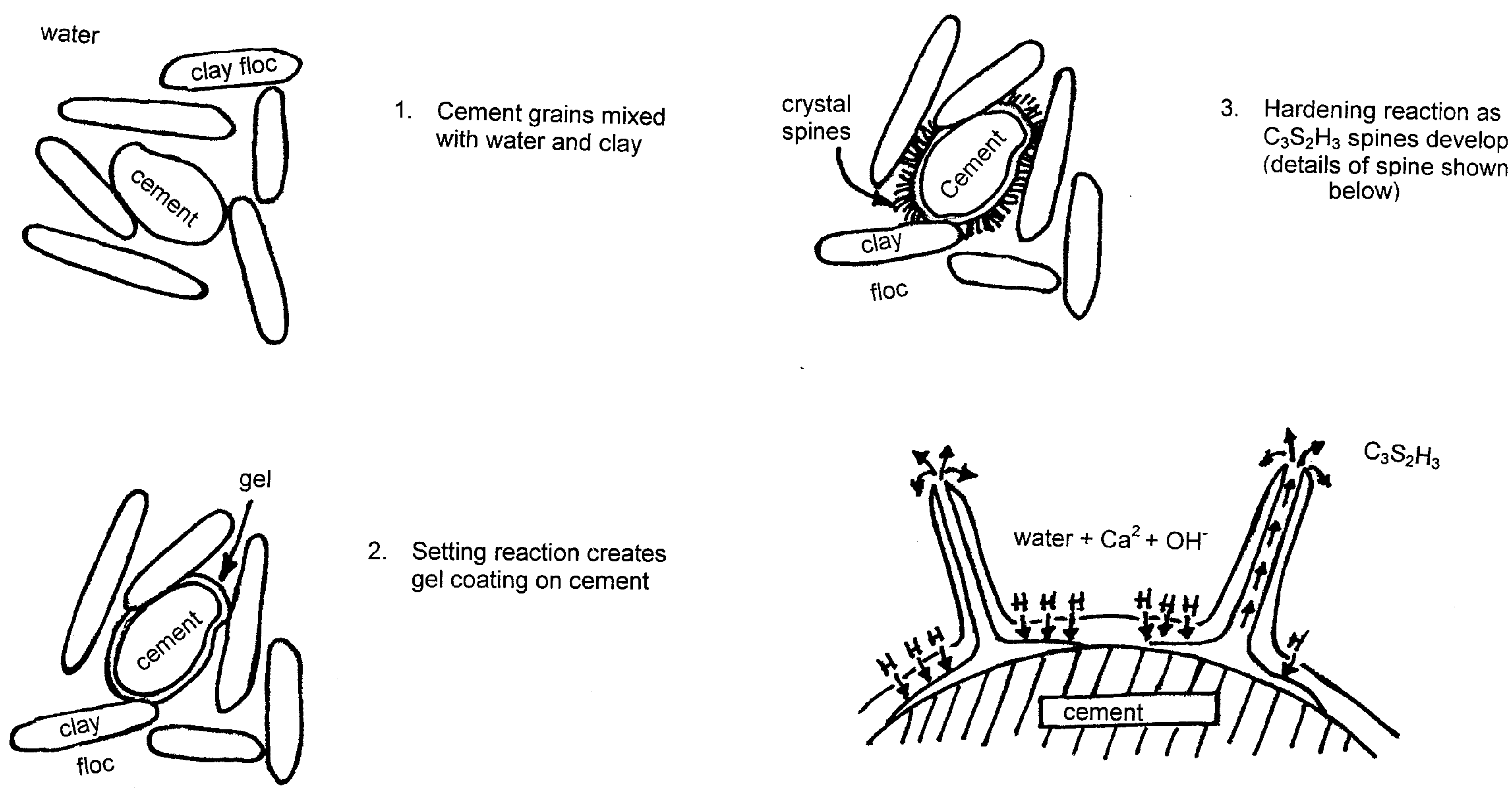


Figure 7. Setting and hardening of ordinary portland cement (after Ashby and Jones, 1986).

$$Ca(OH)_2 \rightarrow Ca^{++} + 2(OH)^{-1} \qquad [8]$$

This usually results in a strongly alkaline environment (pH greater than 12), and the high alkalinity causes partial dissolving of the clay minerals, releasing silica and alumina from clay particle surfaces. The elements released from the clay are acidic, and they combine in a pozzolanic reaction with the free calcium and hydroxyls (the alkali) to form insoluble silica and alumina salts (referred to as CSH and CAH) (Equations 9 and 10).

$$Ca^{++} + 2(OH)^{-} + \underset{\text{(soil silica)}}{SiO_2} \rightarrow \underset{\text{(secondary cementitious product)}}{CSH} \qquad [9]$$

$$Ca^{++} + 2(OH)^{-} + \underset{\text{(soil alumina)}}{Al_2O_3} \rightarrow \underset{\text{(secondary cementitious product)}}{CAH} \qquad [10]$$

This pozzolanic reaction can continue for quite a while, essentially until the components that caused the alkaline solution are consumed. This may be many months or years depending on the amount of chemicals available. The large surface area of the clay particles provides more than enough available silica and alumina to neutralize the basic solution. In this pozzolanic reaction, the pH of porewater is lowered and, when it falls below 12.6, the primary cement gel hardening reaction is stopped and the secondary product is formed. It is important to appreciate that these insoluble salts, which precipitate from the pozzolanic reactions, are not nearly as strong as the primary cementation resulting from the initial formation of tobermorite gel.

The degree to which pozzolanic reactions will occur is dependent on the amount of available $Ca(OH)_2$, the chemistry of the pore fluid, and the solubility of the silica and alumina on the surfaces of the particular clay mineral particles present. If the porewater is quite acidic to begin with, then much of the alkalinity induced by the cement would be used in raising pH to even a neutral state. Clay mineral solubility is affected by any impurities that may be present, clay particle grain size, degree of crystallinity, and other factors.

Conversely, if only quicklime (CaO) is injected into the soil, the following reaction occurs:

$$CaO + \text{a lot of } H_2O \rightarrow Ca(OH)_2 + \text{a lot of heat} \qquad [11]$$

The calcium hydroxide so formed disassociates in the porewater (into Ca^{++} and OH^- ions), raising the pH and dissolving the SiO_2 and Al_2O_3 from the clay particles, in the same fashion described above (i.e., ion exchange, flocculation, and pozzolanic reactions initiated). The consumption of soil porewater may itself be a substantial benefit in stabilizing the clay soil, by reducing its Plasticity Index to far below the "A" line of Casagrande (i.e., properties of a silt). Esrig (1999) reported on tests on Boston Blue Clay of 35 to 40% natural moisture content where cement and lime addition reduced the water content by 2.5 percentage points, equivalent to an approximate increase in consolidation stress of about 50%. The reduction in moisture content is much less significant in montmorillonitic soils with natural moisture contents near 250%.

In contrast, adding gypsum with lime or cement leads to the production of ettringite needle crystals, which can improve final strength. However, ettringite can cause swelling and is unstable at temperatures over 40°C, leading to a decline in the use of gypsum in recent years.

2.4.4 Comments on Soil-Cement Strength Gain

The two principal reactions, namely the cement hardening reaction and the pozzolanic reaction, occur simultaneously in the formation of soil-cement. The duration and extent (or speed) of each reaction depends on the amount of cement introduced into the soil. At lower cement contents, the tobermorite gel from the cement grain hydration may only extend a small distance from its surface, and the pH reduction from the pozzolanic reaction may quickly overtake the primary cementitious crystal growth, thus providing only a weak soil-cement material.

When the cement content is higher, the tobermorite gel will be more abundant, and the resulting crystals can grow large enough to overlap into an interlocking structure. Within this structure, the secondary cementitious products from the pozzolanic reactions will reside, giving a hardened, dense matrix around stiff particles.

A certain threshold cement content would appear to be necessary to achieve a minimum spread of cement throughout the remolded clay. This would impart a minimum stiffening throughout the particle matrix by pozzolanic reaction and provide a weak, widely spread degree of primary cementation. As cement content increases above the minimum, then the products of the primary cementation would converge and become stronger as the time to pH depression below 12.6 increases, and pozzolanic reaction potential would be greater and continue longer. However, there would also appear to be an upper boundary to the cement content beyond which further improvement in soil-cement strength would be minimal.

Illustrative data are provided in Chapters 4 and 5.

2.5 Soil Chemistry and Other Factors Affecting Soil-Cement Formation

The clay-cement reactions show strong dependence on pH, clay composition, and crystalline structure, and related factors that would interfere with the chemical reactions needed for primary cementation and pozzolanic reactions. However, the water content and the cement factor have been shown to be the dominant controls over the properties of the treated soil. Although this is explored in detail in later chapters, the following introductory statements can be made at this point.

Generally, when water contents exceed 200 to 250%, soil-cement formation is greatly inhibited. Cement factor must be above 5% before there is significant cementing action. It has been demonstrated by Thompson (1966) that organic content and pH of a soil have significant influence on reactivity with lime. Takenaka and Takenaka (1995) show the same dependence for cement stabilization. The reasons for organic content to negatively affect the ability of soil to react with lime or cement are not completely clear, but are related to the propensity for acid formation during the chemical reactions of the stabilization process. Data from the summary presented by Takenaka and Takenaka are shown in Figure 8, which clearly shows very low strengths achieved for the five clay soils tested that had high organic components and low pH values.

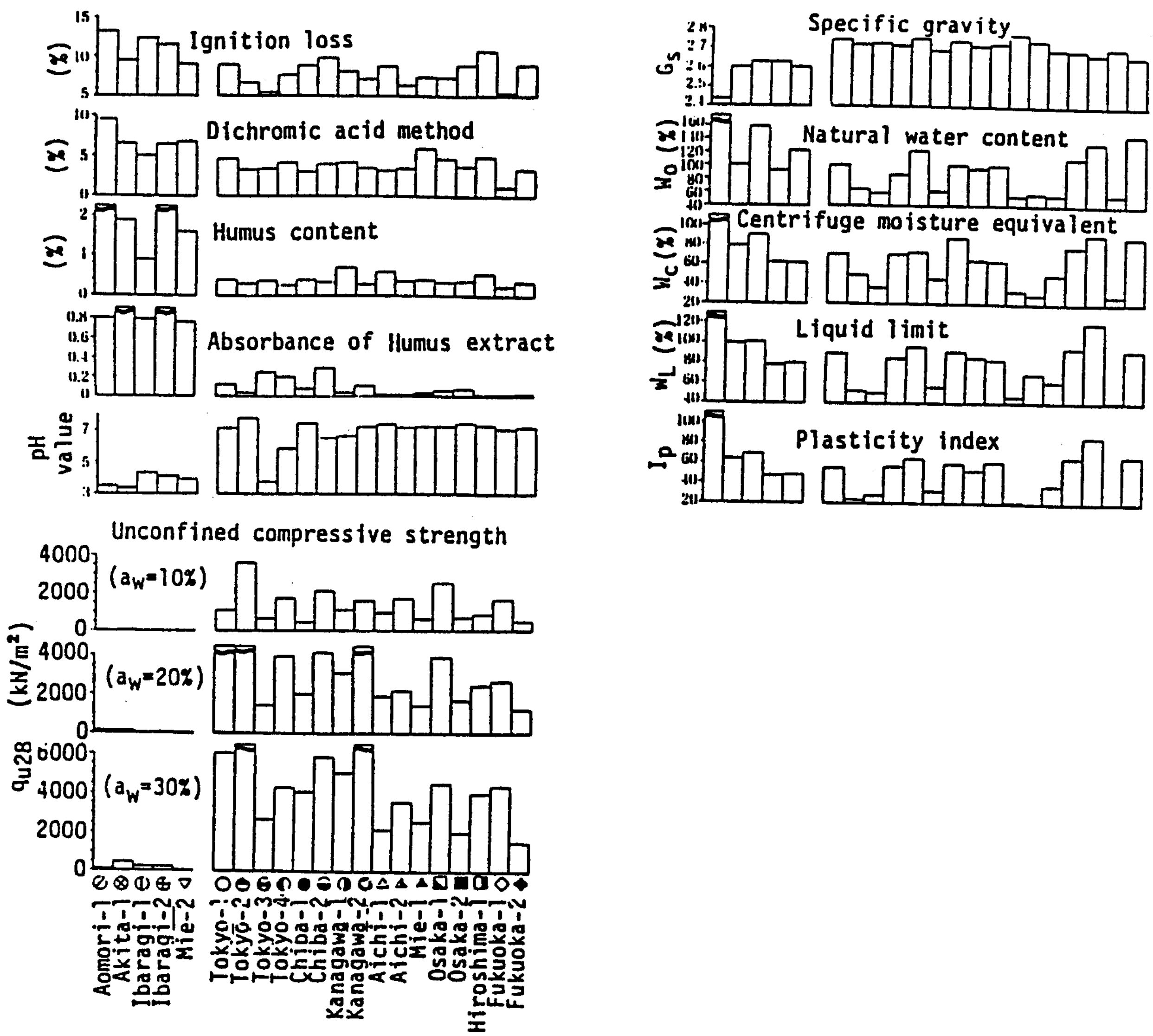


Figure 8. Relationship between physical and chemical properties and unconfined compressive strength of improved soils (Takenaka and Takenaka, 1995).

The influence of soil type, especially as related to organic soils, is also clearly illustrated in data presented by Åhnberg (1996) in Figure 9. The clayey silt and quick clay achieved significantly higher strengths. Interestingly, the peat, when mixed with an extremely high cement factor of 500 kg/cm^3, was made to achieve strengths of the same magnitude as the other soils, but at such high cement content, there must have been considerable replacement of peat by the added cement. The quick clay had a high sodium content making it relatively reactive and so capable of higher strength due to increased secondary CSH-CAH gel production. The gyttja (mud or muck) had a very high water content, liquid limit, and organic content and therefore very low strength. (Acidic conditions inhibit solubility because of low pH, and contribute to the weakness of organic matter once it becomes cemented.)

The amount of reactive silica and alumina in clay soil and fines content have been found to affect lime reactivity (Queiroz de Carvalho, 1981), but may have little effect on cement stabilization, which is only partly dependent on the pozzolanic reaction for soil stabilization. Clearly, for the stabilization of cohesionless soils, the strength gain will result from the primary cementation reaction.

Water content has been shown to have substantial affect on development of soil strength, with higher water contents decreasing the achievable strength of soil-cement. Just as in concrete and grout mix proportioning, water in excess of an optimum range will cause lower strength (and reduced durability) in stabilizing both sand soils and clays. Data presented by Babasaki et al. (1996) are shown in Figure 10 and demonstrate that about 225% water content is an upper threshold for soils that can be strengthened.

2.6 The Action of Additives to Cementitious Systems

Figure 11 shows how the permeability of a concrete is controlled primarily by the capillary porosity of the cement-water paste. The paste comprises in addition to the $C_3S_2H_x$ gel, CSH gel,

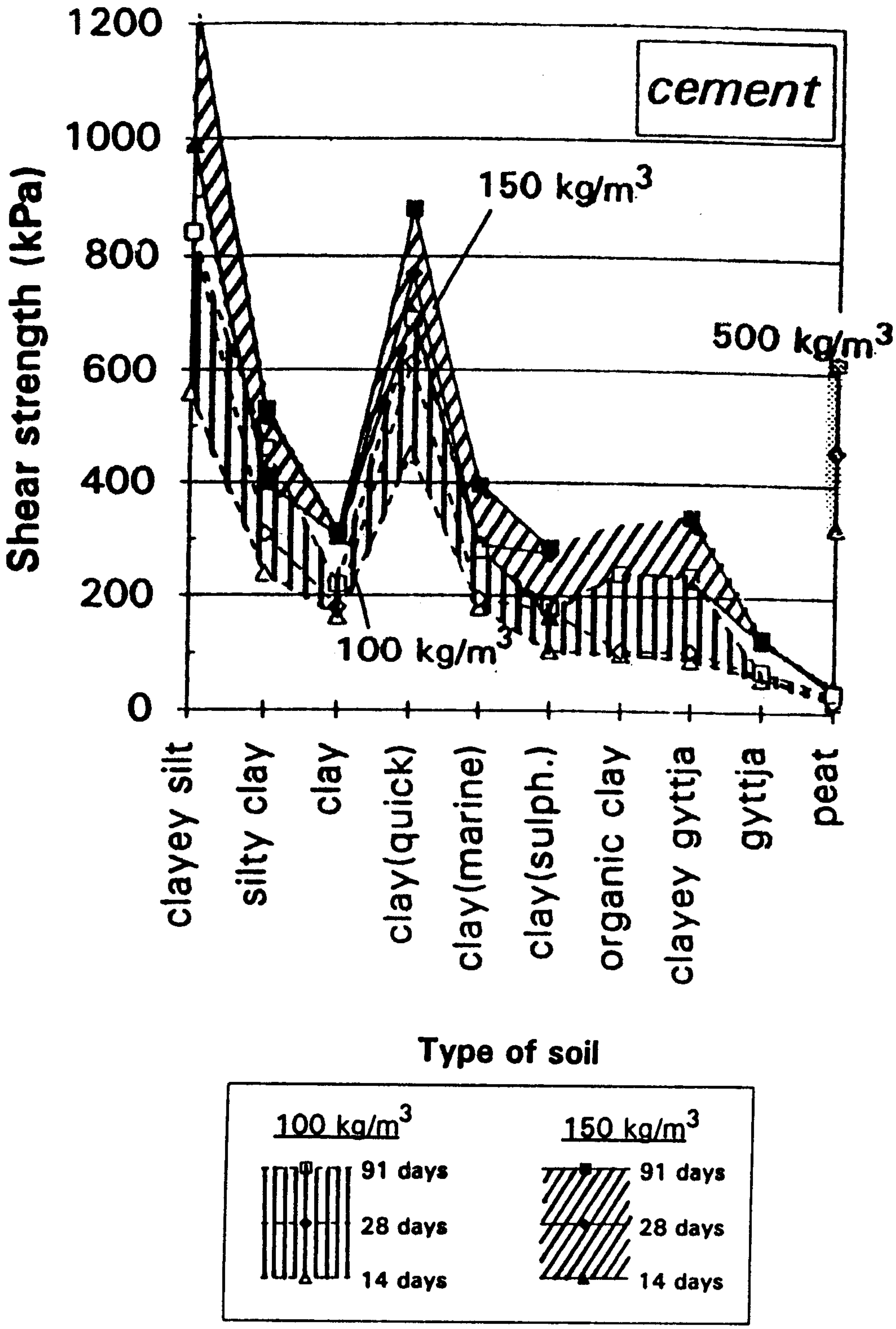


Figure 9. Shear strength estimated from unconfined compression tests on 50-mm diameter x 100-mm samples, 14 to 91 days after stabilizing with different amounts of cement (Åhnberg, 1996).

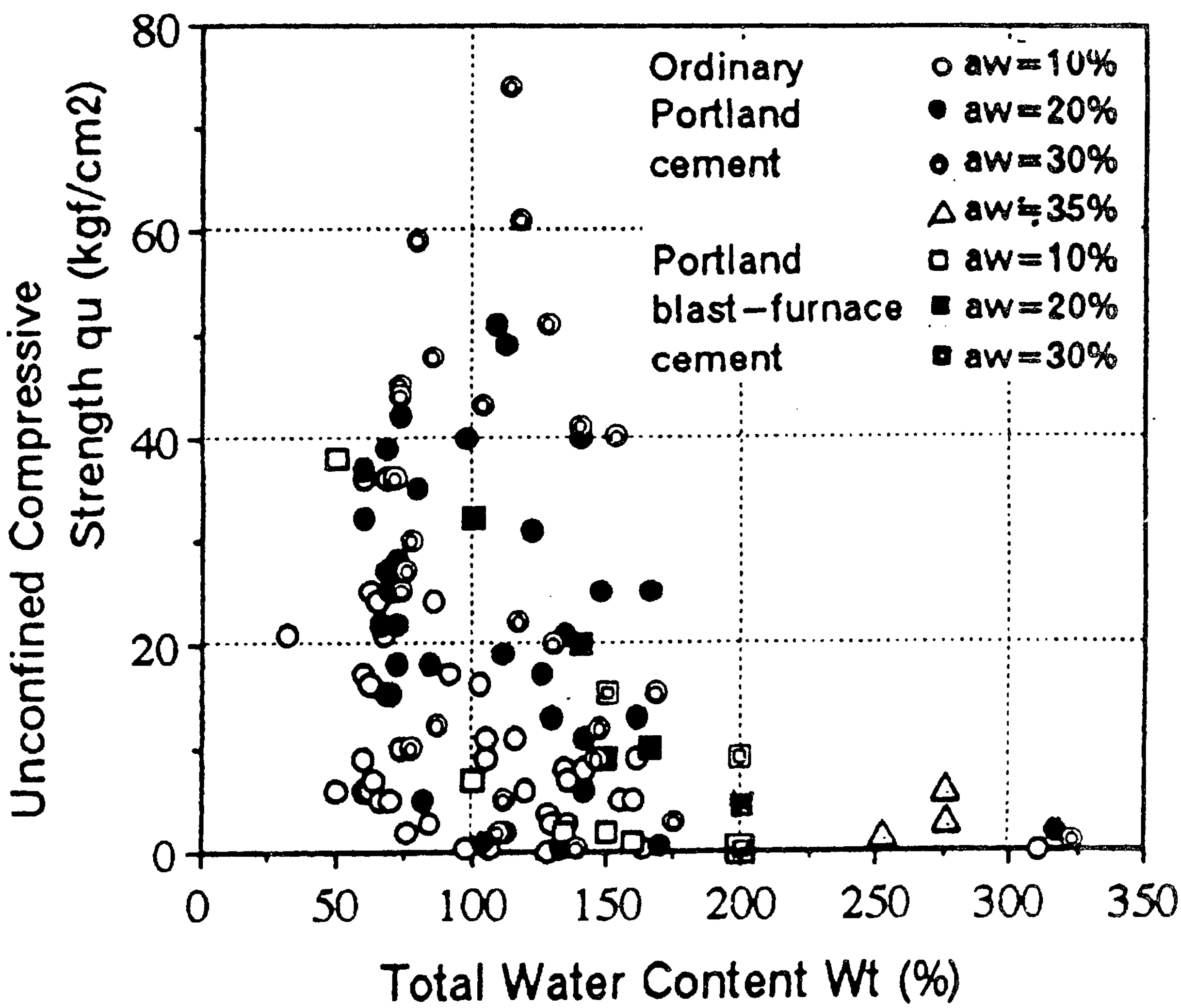


Figure 10. Relation between unconfined compressive strength q_u and water content (by weight) (Babasaki et al., 1996).

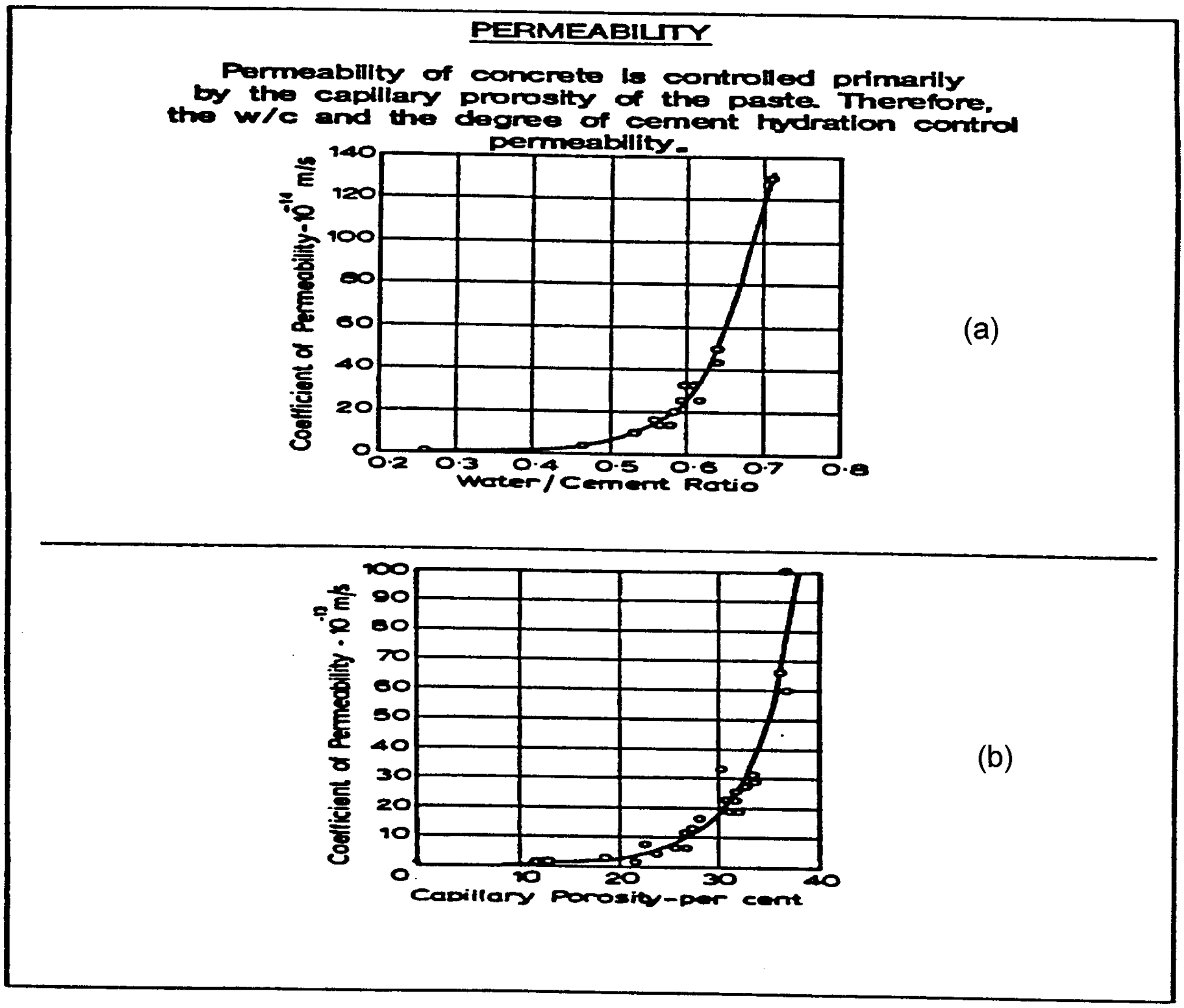


Figure 11. a) Relationship between permeability and water/cement ratio for mature cement pastes (93% of cement hydrated) and b) Relationship between permeability and capillary porosity of cement paste (Gause, 1998).

Calcium Hydroxide (20 to 25% of paste volume), capillary pores (dependent on water/cement ratio and degree of hydration), unhydrated cement (dependent on water/cement ratio and efficiency of cement dispersion), porewater, and entrained or entrapped air. Thus the permeability (and other related factors such as strength, durability, freeze-thaw resistance, and so on) are directly related to the water/cement ratio and the degree of cement hydration.

Chemical and mineral admixtures can be used to decrease porosity by

- Reducing capillary porosity;
- Reducing unhydrated cement;
- Increasing hydration product;
- Reducing porewater; and
- Filling voids and pores.

Surfactants (dispersants) are materials with two structurally dissimilar groups within a single molecule that provide unique surface behavior or surface activity by reducing interfacial tensions between two liquids, or between a liquid and a solid. These additives aid rheological properties of cement-based systems by

- Dispersing individual particles, promoting uniform and maximum separation (lowers the yield stress); and
- Deflocculating groups of particles (lowers the plastic viscosity).

Retarders act in their normal range on the C_3S and gypsum components, but not on the C_3A. At high doses they retard C_3S and gypsum but accelerate C_3A, leading to rapid stiffening or set.

Conversely, hydration inhibitors, such as MBT's Delvo system (Figure 12) control the hydration of all the cement minerals (C_3S, C_3A, C_2S, C_4AF, and gypsum). Hydration is allowed to recommence only when the second chemical component (the Activator in Figure 12) is added to the cement and water grout.

3.1 Portland cement concrete acquires setting time and strength characteristics by a chemical reaction between cement compounds and water to form a rigid material called calcium silicate hydrate gel (CSH gel). This process is called hydration and produces a rapid release of calcium ions into solution and forms a CSH gel rind around the cement particles. As concrete sets, hydrates formed by cement hydration flocculate (clump up) as shown in Figure 1. It is this process which turns workable concrete into a stiff mass

3.2 Master Builders has developed the DELVO System, a two-component non-chloride chemical system to control the dynamics of cement hydration. The first component of the DELVO System, DELVO Stabilizer, when dispensed and thoroughly mixed into returned plastic concrete, controls the rate of hydrate formation by tying up (complexing calcium ions on the surface of cement particles. Figure 2 shows that the DELVO Stabilizer performs a dual purpose by stopping cement hydration by forming a protective barrier around cementitious particles, and acts as a dispersant preventing hydrates from flocculating (clumping up) and setting. The protective barrier around cementitious particles prevents portland cement, fly ash, and granulated slag from achieving initial set.

3.3 The stability of the protective barrier around cement particles is so great that returned plastic concrete can be stabilized and kept plastic for a few minutes, a few hours, overnight or over a weekend. The DELVO Stabilizer is different from conventional retarding admixtures because it (DELVO Stabilizer) is a surface active material having a greater affinity for calcium ions on cement hydrate surfaces. The DELVO Stabilizer controls (stops) cement hydration by acting on all phases of cement hydration. Conventional retarding admixtures at normal dosage rates do not act on C_3A, a primary cement mineral which contributes to setting time and early age strength characteristics of concrete. The use of retarding admixtures at high dosage rates may cause severe concrete stiffening, flash set and low strength performance.

3.4 The second component of the DELVO System, DELVO Activator, when dispensed and thoroughly mixed into stabilized concrete either the same day, the following day or after a weekend, breaks down the protective barrier around cementitious particles as shown in Figure 3. As soon as this is completed and the activated concrete is combined with freshly manufactured concrete, normal cement hydration (flocculation), setting time and strength performance takes place (see Figure 4).

Figure 1 – As concrete sets, hydrates formed by cement hydration flocculate (clump up).

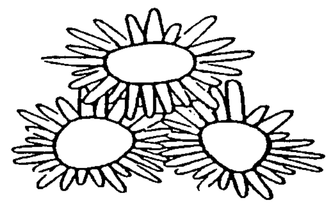


Figure 2 – The DELVO Stabilizer, when dispensed and thoroughly mixed into returned plastic concrete, stops cement hydration by forming a protective barrier around cementitious particles.

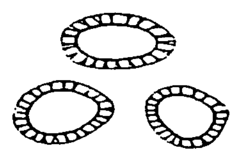


Figure 3 – The DELVO Activator, when dispensed and thoroughly mixed into stabilized concrete, breaks down the protective barrier around cementitious particles

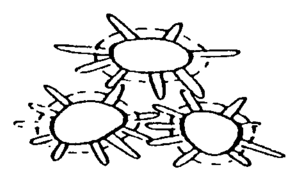


Figure 4 – When activated concrete is combined with freshly manufactured concrete, normal cement hydration (flocculation), setting time and strength performance takes place.

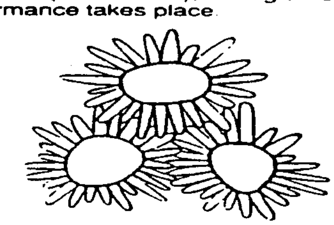


Figure 12. Chemical system control of cement hydration (Gause, 1998).

Dispersants can also be used that act on the individual clay particles themselves. The benefits of these chemicals include providing increased rates of penetration, improved grout/soil mixing efficiency, and reduced amounts of binder (for a given strength criterion).

2.7 Summary

Deep mixing introduces cement into the soil, which then reacts with available water to hydrate and form various cementitious compounds. When mixing is in sandy soils, the cement alone is the cementitious medium. In clay soils, an important secondary pozzolanic reaction occurs that adds to the strength of the soil-cement system: the initial hydration of the cement releases calcium hydroxide that increases porewater alkalinity and reacts with available silica and alumina on the clay particles. Although the resulting crystals are not as strong as those of the cementation reaction, there can be substantial long-term strength gain from pozzolanic activity. However, since clays may be highly and often subtly different, thorough site-specific study and laboratory testing are still necessary to confirm proper mixing conditions and expected results for any given application.

It has been clearly demonstrated that organic soils and soils of low pH greatly impede stabilization with cement. Although it may be possible to achieve some benefit from cement stabilization, specific study is imperative at sites having either property. Also, soils with moisture contents in excess of 225% are likely not readily amenable to economic improvement.

A most important factor in achieving efficient deep mixing remains the degree of uniformity in mixing the cement with the soil. For the cement to react properly with and cover soil particles, it must be initially well dispersed and the soil well disturbed, so as to allow the cement to be uniformly distributed. In this regard, the use of chemical admixtures may offer considerable advantages.

3. QUALITY ASSURANCE/QUALITY CONTROL AND PARAMETER VERIFICATION

3.1 General Philosophy of Test Programs

Regardless of the level of expertise of the contractor, and/or the level of understanding of the particular site conditions, some type of pre-production test program is highly advisable, if not essential. Such a program affords the opportunity for the contractor to demonstrate that the specified performance criteria, tolerances, and engineering properties can be met, even if two or more iterations have to be made. Once these criteria have been achieved, then the production parameters can be selected logically and only modified if there are obvious changes in the production area ground, or project scope. These parameters can be as straightforward as water/cement ratio and cement factor. It is also essential to explore the differences between data obtained from laboratory-generated samples and those obtained under actual field conditions.

Such pre-production test programs require the scope of the testing and the acceptance criteria for every aspect to be clearly defined. Testing and sampling is usually more rigorous than in the subsequent production phase. Test programs should also be a demonstration of the efficiency of the quality assurance/quality control and verification processes themselves.

During production, it is normal to find the intensity of material sampling and testing markedly reduced from that of the pre-production phase, and a proportionally larger emphasis placed on construction parameter records as the prime level of comfort of acceptable in situ performance. This is then backed up by post-construction verification testing of the treated soil, for example by coring or any one of a number of geophysical tests.

This chapter focuses on the different levels of process quality control exercised by the contractors and on the various methods used to verify the properties of treated soil. Subsequent chapters illustrate the mass of experimental data available on the properties of treated ground. Reinforced by such data, performance prediction models can be established and optimized. Typical of the systematic approach adopted in Japanese practice is the work of Saitoh et al. (1996), colleagues in the Takenaka group of companies. Figure 13 is a standard conceptual flow chart for determining and achieving target strengths, while Figure 14 summarizes the planning of a test program based on lab testing of soil from the target area. This permits key parameters to be selected for field use, with 7-day testing allowing a 28-day result to be projected (and changes made, if necessary), and 28-day core testing of in situ material providing confirmation of performance.

Before mobilizing the DMM equipment to the site, the contractor must have a sense of both the requirements for the project and the behavior of the soils when mixed with the binder. Although there is a major difference in the mixing processes used in the laboratory and in the field, the initial judgments for actual in situ mixing will be guided by the preliminary laboratory study.

With this index of soil treatment properties that can be obtained in a well-planned laboratory study, and experience with similar soils and knowledge of the operation and mixing efficiency of the equipment, the contractor can proceed to design the mixing operation. In bidding the project, the contractor will be aware of the degree of verification required by the contract documents.

As an elegant summary of the various elements of a Quality Control and Verification Program, Takenaka (1995) provided the brochure page shown in Figure 15. In their terminology, the successive steps shown in this figure are

1. Pre Investigation – "confirmation of soil profile and soil constant."
2. Indoor Mix Test – laboratory testing.
3. Test Application – field test of treated soil.
4. Application Control – determination of production parameters.
5. Application Procedure – recording of production parameters.
6. Post Investigation – verification of treated soil.

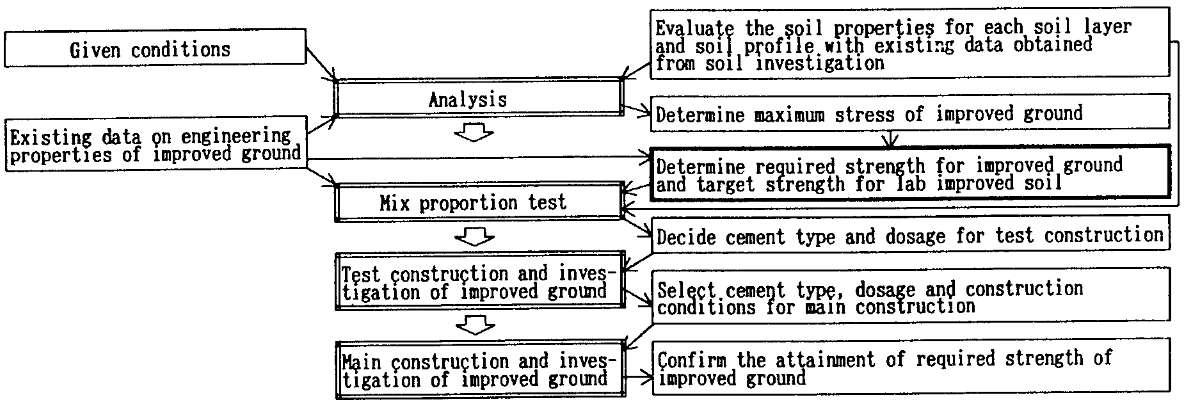


Figure 13. Flow chart of work involved to determine and achieve required strength of improved ground (Saitoh et al, 1996).

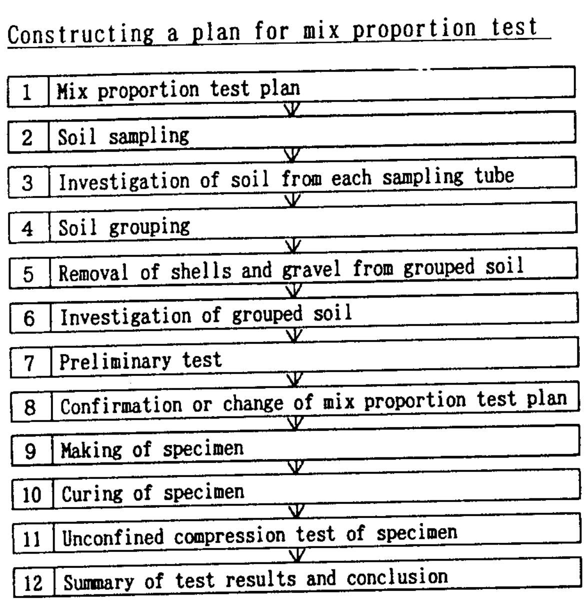


Figure 14. Actual work flow for mix proportion test (Saitoh et al., 1996).

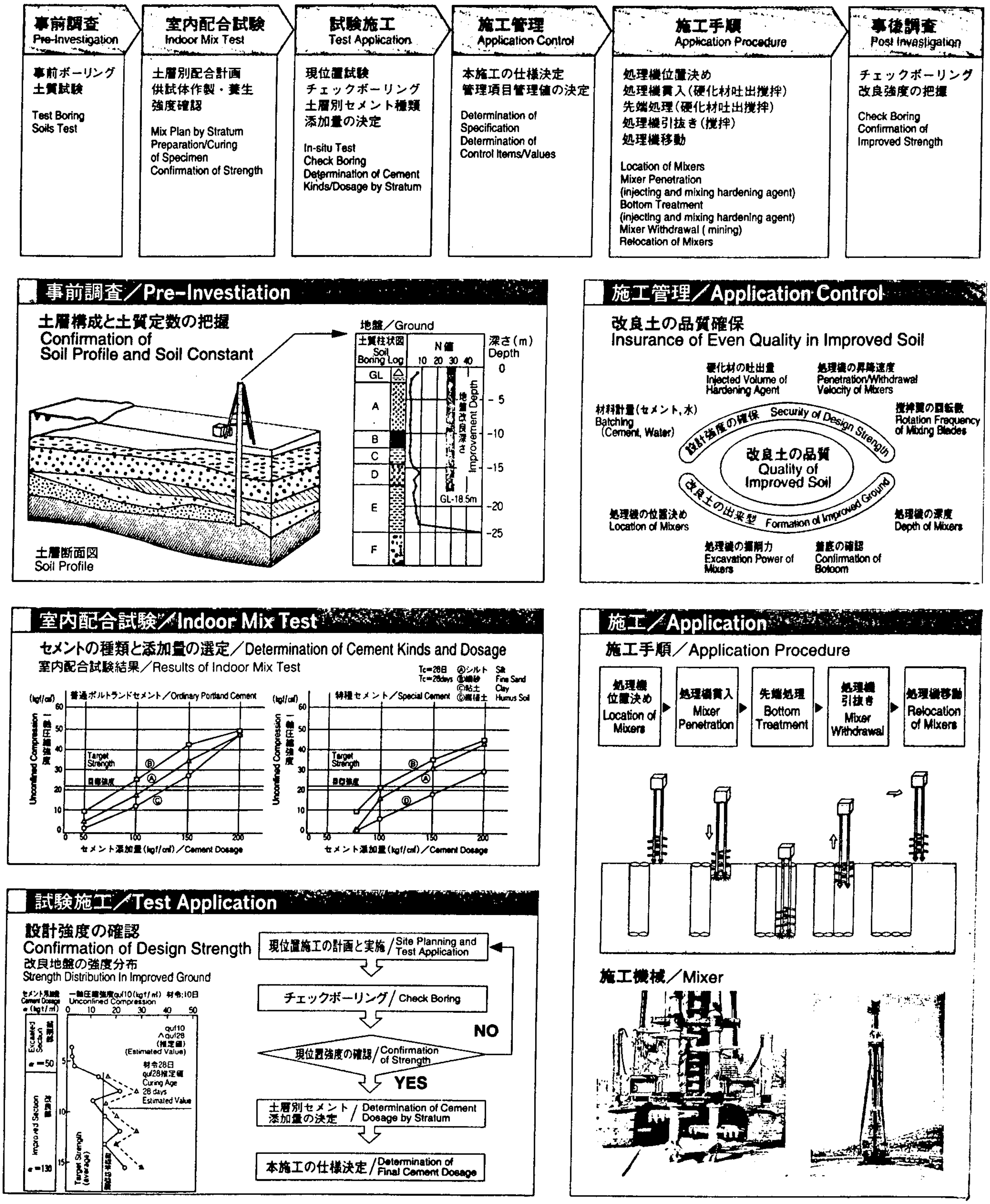


Figure 15. Quality control and application procedure, from pre-investigation to post-investigation (Takenaka, 1995).

3.2 Process Control

3.2.1 Background

During the formation of DMM elements, the key mechanical and other data can be transmitted to the DMM rig operator, and/or the binder plant operator, in real time, so that they can monitor consistency of performance, and if necessary make changes in response to binder and/or soil characteristics. Conversely, certain DMM techniques involve such rapid penetration rates that the desired binder injection parameters, especially rate of slurry injection, can only be controlled by the use of a computer management system. Computerized systems also provide a historical record of the operation, which constitutes for both the contractor and the client an accurate basis for payment, as well as confirmation that the process design parameters have in fact been observed during construction.

When regarding the different levels of parameter recording, it should be recalled that the requirements placed on the contractor for quality control may vary greatly. Thus, for example, strength targets are not usually set for mixed soil in Scandinavia and so the contractor is responsible only for ensuring the quality and the amount of the binder (Halkola, 1999) whereas the owner dictates the rates of rotation (e.g., 150 to 200 rpm) and withdrawal (e.g., 10 to 25 mm/rev) and the cement factor (actual within 10% of design). In Japan and North America, it is common to find performance goals set for the contractor in terms of strength and/or permeability and so different levels of recording and reporting are often required.

3.2.2 Levels of Process Control

Table 3 summarizes published information from the various DMM techniques listed in Table 1. It is possible to determine three broad levels of process control, based on the degree of sophistication: Level 1: Batching and injection parameters for the slurry (or dry binder) are monitored by simple instrumentation and are displayed on digital or analog gauges for field personnel to view. Spot

Table 3. Information on process controls and testing methods, based on published information.

Method Name (Company)	Process Control	Type of Field Verification Reported						Treated Ground Parameters Actually Measured					
		Lab Tests	Wet Grab	Spoils Composition	Core	Expose and sample	Geophysical and others	U.C.S.	Shear	E	Tensile	k	Others
1 DSM (Geo-Con)	"The configuration of auger shafts mechanically created the required dimensions and overlaps in the ground and the arrangement of the flights and beater bars assured thorough mixing. A tachometer was attached to a roller on the electric motor guide and ran down the leads giving a digital readout on depth. An inclinometer was used to align the leads to within ±0.1° in the x and y axis. With the leads vertical and the auger shafts being very rigid, the columns were drilled vertically. The grout flow was controlled by electronic flow metering devices. A control box received depth readings from the tachometer and converted the data into a drill rate in ft/min. The flow devices were installed on the grout circulation lines. Depending on drill rates, the devices caused a predetermined amount of grout per foot of drilling to be injected into the column. Excess grout was recirculated to the holding tank. The control box totaled the grout injected per column and digital readouts displayed grout pressure, depth of column, verticality, drill rate, grout flow rates, and total grout flow" (Ryan, and Jasperse, 1989).	✓	✓	✓	✓			✓	✓			✓	
2 SMW (SMW Seiko, Raito, et al.)	Computer controlled batching plant and pumping rates. Close verticality control. Display of penetration and withdrawl rates, rpm, torque.	✓	✓	✓	✓			✓	✓	✓	✓	✓	Poissons ratio, unit weight, drained triaxial, durability

Table 3. Information on process controls and testing methods, based on published information (continued).

Method Name (Company)	Process Control	Type of Field Verification Reported						Treated Ground Parameters Actually Measured					
		Lab Tests	Wet Grab	Spoils Composition	Core	Expose and sample	Geophysical and others	U.C.S.	Shear	E	Tensile	k	Others
3 Multimix (TrevilCOS)	Computer recording of major parameters for construction control and records, principally drilling parameters and slurry parameters. Slurry viscosity measured.	✓	✓	✓	✓	✓		✓		✓		✓	Plate load tests, core recovery
4 Colmix (Bachy)	The correct dosage of slurry was achieved by a Lutz system using a flowmeter on the outflow of the agitator linked by cable via the controls to a Tarracord computer in the rig. The operator enters the required dosage, and the computer switches the pumps on and off, as required, to ensure that the correct dosage of slurry is evenly distributed to each column. Relevant information is displayed on a screen in the cab, and printouts are subsequently obtained for each column. The printouts include volume injected, torque, time, and drilling speed.	✓	✓		✓	✓	Sonic velocity in 3D, uniform response to phenolphthalein	✓	✓	✓		✓	
5 Soil Removal Technique (Shimizu)	No data, but assumed comparable to CDM.				✓			✓					Lateral movements
6 CDM (Numerous)	Very high emphasis on control and recording of drilling and injection parameters. "Execution Management Unit" claimed to offer real time understanding of actual field conditions, reliable execution and excellent quality assurance and control.	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	✓	Durability

Table 3. Information on process controls and testing methods, based on published information (continued).

Method Name (Company)	Process Control	Type of Field Verification Reported						Treated Ground Parameters Actually Measured					
		Lab Tests	Wet Grab	Spoils Composition	Core	Expose and sample	Geophysical and others	U.C.S.	Shear	E	Tensile	k	Others
7 SSM (Geo-Con)	Assumed as for DSM.	✓				✓		✓				✓	Chemical analyses
8 SCC (SCC Technology)	Slurry is weigh batched and rate of injection shown on flowmeter.	✓			✓	✓		✓				✓	
9 MecTool (Millgard)	Computer control over "major system components," which also provides full documentation of each column's parameters.				✓			✓				✓	
10 RAS (Raito Kogyo)	"Construction Management System" exerts control over depth, penetration/withdrawal rates, rpm, and injection rate. Like CDM.				✓			✓					
11 Rectangular 1 (Shimizu)	Shape of treated volume closely defined.					✓	Inclinometer						
12 Rectangular 2 (Daisho Shinko)	Shape of treated volume closely defined.	✓			✓	✓		✓					Core recovery

Table 3. Information on process controls and testing methods, based on published information (continued).

Method Name (Company)	Process Control	Type of Field Verification Reported						Treated Ground Parameters Actually Measured					
		Lab Tests	Wet Grab	Spoils Composition	Core	Expose and sample	Geophysical and others	U.C.S.	Shear	E	Tensile	k	Others
13 Single Auger Mixing (Terra Constructors)	No data: assumed as for DSM.												
14 Cementation (Cementation)	No data.												
15 Single Axis Tooling (Hayward Baker)	Various degrees of computer display and control. Only instrumentation provided on volume and pressure at the pumps. Drill penetration data displayed and recorded. Slurry unit weights (density) measured every 15 minutes.	✓	✓	✓	✓	✓	CPTs before and after SPTs possible in softer treatments	✓					Chemical analyses
16 Rotomix (Inquip)	No data. Assumed as for DSM.												
17 SWING (Numerous)	Computer control of mixing, drilling, and injection parameters.			✓	✓			✓					

Table 3. Information on process controls and testing methods, based on published information (continued).

Method Name (Company)	Process Control	Type of Field Verification Reported						Treated Ground Parameters Actually Measured					
		Lab Tests	Wet Grab	Spoils Composition	Core	Expose and sample	Geophysical and others	U.C.S.	Shear	E	Tensile	k	Others
18 JACSMAN (Fudo / Chemical Grouting Co.)	Shape of treated volume closely defined. Strong control over drilling and injection parameters (like CDM).			✓	✓			✓					
19 LDis (Onoda)	Assumed equivalent to jet grouting.				✓		Vertical displacements	✓					
20 GeoJet (Condon Johnson Associates)	During the very rapid installation, the 486k microprocessor analyzes rotation rate, penetration rate, slurry pressure, torque, crow force and treated soil volume and density as a function of depth during the construction of each element. The computer reacts to changing ground parameters and adjusts injection parameters to maintain specific treated soil properties in different strata. Rotation is stopped automatically if treated soil parameters exceed preset limits. The system is equipped with a microprocessor to monitor and control the entire process given the very fast penetration rates achievable. An automated spotter/positioning system is also provided. The operator control panel has a touch screen.	✓			✓	✓	Pile load testing	✓					
21 Hydramech (Geo-Con)	Computer controlled grout delivery system.				✓			✓					

Table 3. Information on process controls and testing methods, based on published information (continued).

Method Name (Company)	Process Control	Type of Field Verification Reported						Treated Ground Parameters Actually Measured					
		Lab Tests	Wet Grab	Spoils Composition	Core	Expose and sample	Geophysical and others	U.C.S.	Shear	E	Tensile	k	Others
22 DJM (Numerous)	"Advanced automatic monitoring gives continuous and accurate records of the construction depth, rpm, penetration/withdrawal rates, and volume of binder."	✓	✓	✓	✓	✓		✓	✓	✓	✓	✓	
23 Lime Cement Columns (Numerous)	Strong reliance is placed on computer control and real time data display and recording. Operator has touch screen demonstrating all parameters. Automatic verticality control.	✓	✓	✓	✓	✓	Pressuremeter testing	✓	✓	✓		✓	Compressibility important
24 Trevimix (TreviICOS)	Computer control and recording of injection and drilling parameters.	✓			✓	✓		✓		✓			

checks are made manually on slurry fluid properties, e.g., density (by Baroid Mud Balance), fluidity (by Marsh Cone), and so on. Walker (1994) provided a case history example where an intensive program of materials testing was conducted (Table 4), providing the range of data shown in Table 5.

Data on shaft penetration and withdrawal rates are displayed in the drill rig cabin. Typically, the operator is in telephonic contact with the batch plant, and/or the batch plant data may be electronically relayed to the cabin. The operator determines if changes are to be made to drilling and injection parameters based on these inputs and upon general progress observations.

Typical examples of this level of process control would appear to be Methods 8 and 15 as they are currently configured.

Level 2: Batching and injection parameters are largely controlled by computer, having been preset to provide the design volume ratio and cement factor, which is closely related to shaft penetration rate. In turn, these data are automatically recorded and displayed, with visual confirmation to the rig operator that they are within the pre-selected parametric range. If not, manual corrections may have to be made. Full electronic records are made for each column of all salient drilling and injection parameters although operators typically maintain manual logs as well. Spot checks are made of fluid slurry properties.

As illustration, Yano et al. (1996) described details of the "centralized control system of CDM Method," where a computer system is employed in order to guarantee high quality production of columns, and also to control all stages from execution to daily report generation. This centralized control system is composed of a sensor section and an execution management section. The sensor section includes sensor detectors for depth, penetration and withdrawal velocity, volume of slurry discharge, shaft rotation speed, and shaft rotation motor current, while the execution management section consists of processing, monitoring, and recording devices. During execution, the centralized control system checks that the real time data measured each second satisfy the quality standard values registered in advance for each depth and that acceptable quality columns are produced. "If the standard values are not satisfied, an alarm is

Table 4. Materials quality control program (Walker, 1994).

	Subject	Standard	Type of Test	Minimum Frequency	Specified Values
Material	Water	EPA Standard Methods	• pH • Total hardness	Per water source or as changes occur	As required to properly hydrate bentonite with approved additives
	Thinners	-	Manufacturer certification	One per truckload	As approved by engineer
	Bentonite	API STD 13A	Manufacturer certificate of compliance	Truck load	Premium grade sodium cation montmorillonite
	Cement	ASTM C 150	Manufacturer certificate of compliance	One per lot	Portland, Type 1
Bentonite Slurry	Prior to addition of cement	API STD 138	• Viscosity • Density	4 per shift 4 per shift	MFV ≥ 35 s Density ≥ 64 pcf
C-S Slurry	At mixer or surface in the trench	API STD 138	• C/W ratio • Density	• Continuously • 4 per shift	c/w ≥ 0.19 70 ≥ density ≥ 90 pcf
Soil-Cement Bentonite	Grab sample	ASTM D 1633 ASTM D 5084	• UCS • Permeability testing with hydraulic gradient of 15	1 per shift 1 per shift	$k \geq 1 \times 10^4$ cm/s when fully cured
Materials	Water		• pH • Total hardness	One per source One per source	As required to properly hydrate bentonite with approved additives
	Cement	ASTM C150	Manufacturer certificate of compliance	One per truckload	Portland, Type I
	Bentonite	API Std 13A	Manufacturer certificate of compliance	One per truckload	Premium grade sodium montmorillonite clay
	Attapulgite	API Std 13A	Manufacturer certificate of compliance	One per truckload	Premium grade sodium-free attapulgite clay
Slurry	Clay Slurry	API Std 13B	• Viscosity • Unit weight • pH • Filtrate	Two per shift Two per shift One per shift One per shift	TDB
	Grout	API Std 13B	• Cement content • Clay content • Water content • Unit weight	One per batch One per batch One per batch Two per shift	
Backfill Mix	Soil-Grout	EM-1110-2-1906 App VII App XI	• Triaxial permeability • Unconfined strength	One per 100 lin. ft One per 100 lin. ft	TDB

Table 5. Wall permeability and strength results (Walker, 1994).

SAMPLE NO.	SAMPLE DEPTH (m)	γ_d (t/m^3)	PERMEABILITY (cm/sec)	UNCONFINED COMPRESSIVE STRENGTH (KN/m^2)
D1	2.1	1.41	9.31×10^{-7}	73 (7 day)
D2	2.1	1.22	1.63×10^{-6}	229 (7days)
D3	2.1	1.30	1.29×10^{-6}	124 (7 days)
D4	4.6	1.39	8.48×10^{-7}	136 (7 days) 334 (36 days)
D5	2.1	1.26	3.48×10^{-6} 3.09×10^{-6} (Duplicate)	170 (7 days) 423 (35 days)
D6	5.2	1.47	1.63×10^{-6} 9.89×10^{-7} (Duplicate)	157 (7 days) 354 (34 days)
D7	5.2	1.43	1.82×10^{-6}	114 (7 days) 294 (33 days)
D8	2.7	1.40	1.12×10^{-7}	170 (9 days)
D9	1.5	1.03	5.28×10^{-7}	210 (7 days)
D10	1.5	1.51	5.4×10^{-8}	194 (7 days)
D11	1.8	1.40	1.1×10^{-7}	226 (7 days)
D12	1.5	1.55	1.05×10^{-7}	38 (14 days) 28 (28 days)
D13	1.5	1.44	1.07×10^{-7}	75 (7 days) 113 (28 days)
D14	1.5	1.51	1.83×10^{-7}	332 (7 days)
D15	1.5	1.42	8.96×10^{-8}	264 (7 days)
D16	3.7	1.73	1.48×10^{-7}	85 (7 days) 127 (28 days)
D17	3.7	1.50	5.32×10^{-7}	170 (7 days) 233 (28 days)
D18	5.5	1.63	1.27×10^{-6}	122 (7 days) 189 (28 days)
D19	3.7	1.49	3.7×10^{-7}	43 (7 days) 65 (28 days)
D20	3.7	1.48	1.36×10^{-6}	69 (7 days)
D21	3.1	1.48	2.04×10^{-6}	157 (7 days)
D22	2.4	1.59	2.01×10^{-6}	360 (7 days)
D23	1.8	1.31	1.35×10^{-7}	332 (7 days)

issued." After completion of the work, the records from each column can be printed out in tabular form, comparing them with the quality standard values. At the end of each working day, the system can also print out a daily report and production total report that include the numbers of columns, their length, and the amount of cement used. "This centralized control system thus helps produce high quality piles, simplifies the work of preparing various reports and saves time and labor."

Yano et al. further report that when CDM was first commercially used in 1975, quality control was performed by using a four pen recorder (depth, rpm, and the amount of slurry pumped through each of the two mixing shafts). In 1987, a six-pen recorder was introduced, to record in addition, the penetration and withdrawal velocities and the electric current drawn by the rotary head. In 1989, control systems were computerized, enabling all data to be monitored during construction and daily work reports to be produced. This basic approach is still used to confirm the two key quality control items, namely:

- That the amount of slurry pumped is uniform in relation to the volume of soil penetrated, and in accordance with the specifications.
- That the rpm and penetration/withdrawal velocities are sufficient to blend the soil at a "minimum of 350 blade cuttings/meter."

All the data are transferred to the execution management unit, every second, and are totaled as an integrated value per 1 m of depth for comparison with previously input standard values. The operators can therefore easily and quickly adjust the penetration/withdrawal speeds and rate of slurry injection, if necessary. The system also informs the operator when target depth is reached.

The authors listed this control method as having been used on 44 projects (988,797 m^3 treated soil) in 1993 and 61 projects (1,371,348 m^3) in 1994. Typical examples would appear to be Methods 1, 2, 3, 4, 6, 7, 9, 21, 22, and 24, as they are currently configured.

Automatic verticality control over the drill shaft(s) is common at this level and above (e.g., Methods 2, 20, and 23) and, for example, Markteknik (1999) describes the use of a special "direction indication device." In certain cases (e.g., Walker, 1994) such control may be required on certain

Level 1 projects. Verticality and alignment were achieved by two controls: a laser provided a line onto a target on the shaft, for horizontal alignment, while verticality was monitored by two measurements made on the mast (for pitch and roll). Two servo accelerometers mounted on the leads continuously displayed the angle to a display in the operator's cab.

<u>Level 3</u>: The highest level of computer control and display is provided. Method 20 (GeoJet) features a microprocessor that senses, every 6 seconds, rpm, penetration rate, torque, thrust, slurry density, pressure, and rate. The computer reacts to changing ground conditions and automatically adjusts injection parameters to maintain specific treated soil parameters for each stratum. Rotation is stopped automatically if these projected treated soil parameters are unlikely to meet preset limits. The drill operator has a touch screen control system. Level 3 is also characterized by full, continuous records of each column installed.

Tateyama et al. (1996) described how fuzzy logic was used in the process of evaluating real time construction data in order to rationalize construction using the DJM system and to automatically control the mixing rig. In detail, the SPT values of the ground were continuously evaluated based on data on rig torque, penetration resistance on the mixing blades, and the penetration speed of the blades (Figure 16). Predicted and actual SPT values (Figures 17 and 18) showed good correspondence. Using this method, the authors claimed that:

- The depth of penetration can be logically selected.
- The amount of binder (i.e., cement factor) can be varied depending on the variations in the soil.
- The operating parameters of the drilling rig can be optimized.

Level 2 process control concepts are illustrated for Methods 1, 3, 6, 22, and 23 in Figures 19 through 24, respectively. Typical production records for Level 2 controls are shown in

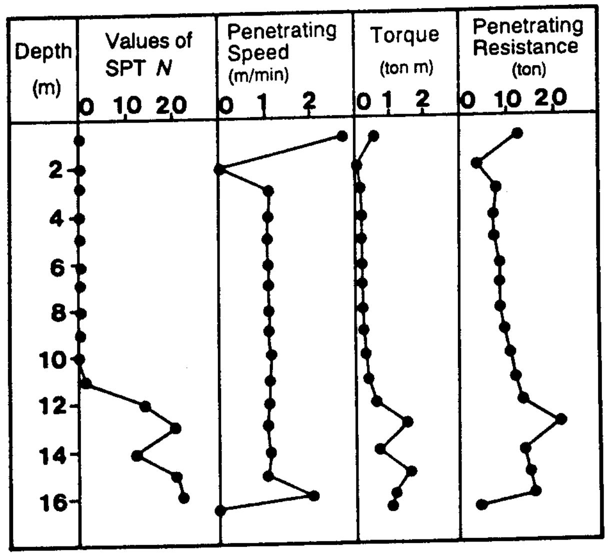


Figure 16. An example of construction data obtained using fuzzy logic (Tateyama et al., 1996).

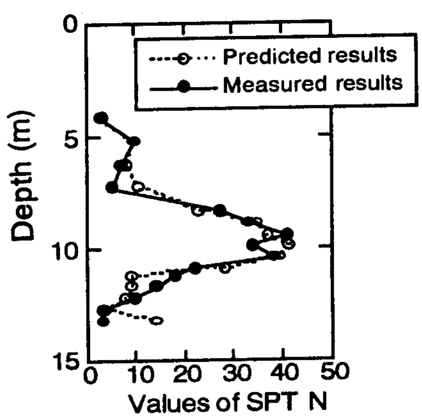


Figure 17. An example of a comparison between predicted and measured values of SPT (Tateyama et al., 1996).

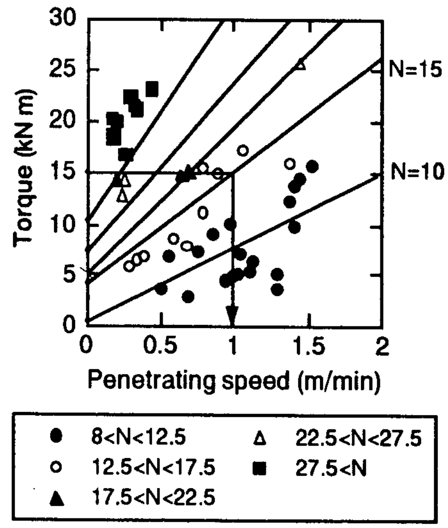


Figure 18. A comparison of predicted and measured values of SPT N (Tateyama et al., 1996).

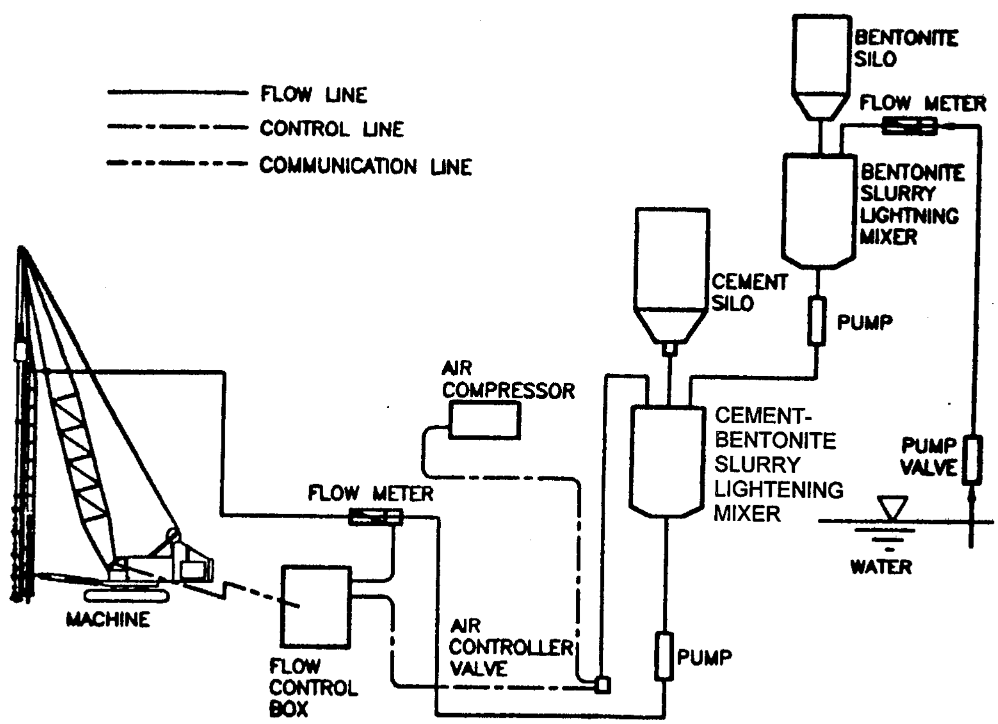


Figure 19. Schematic of DSM batch plant (Bahner and Naguib, 1998).

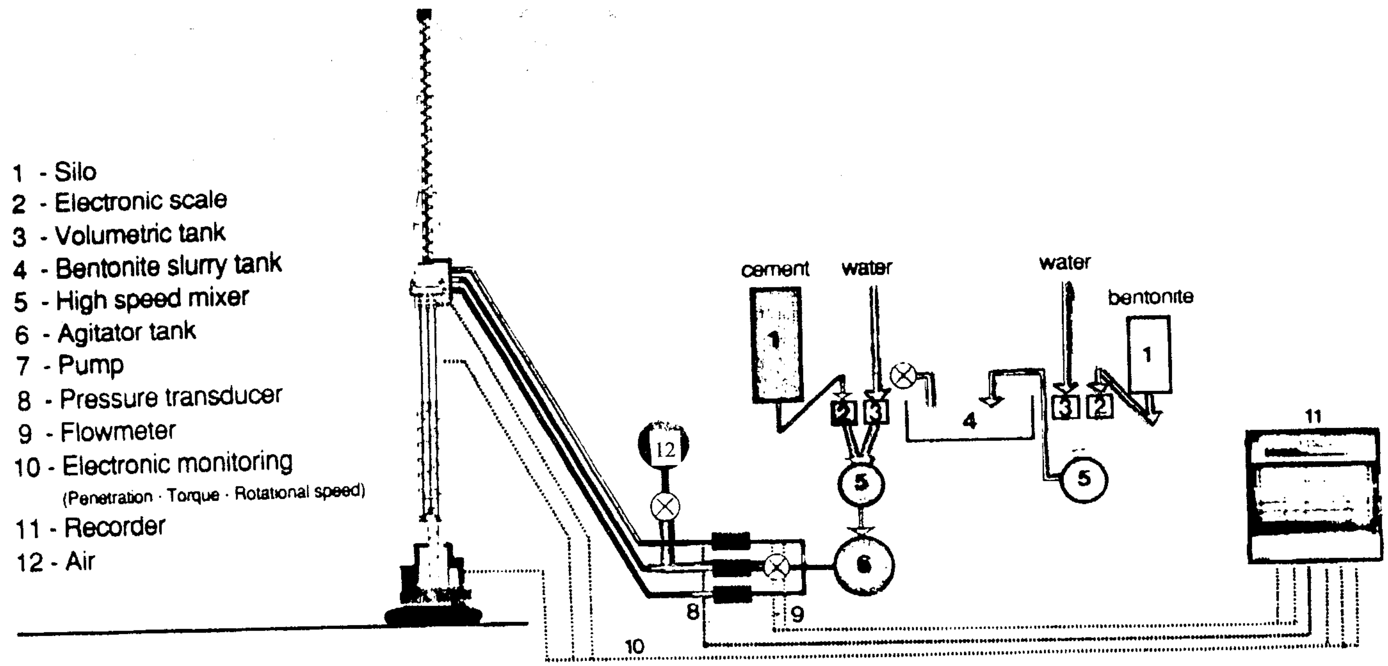


Figure 20. Schematic of Multimix equipment (Rodio, Inc., 1990).

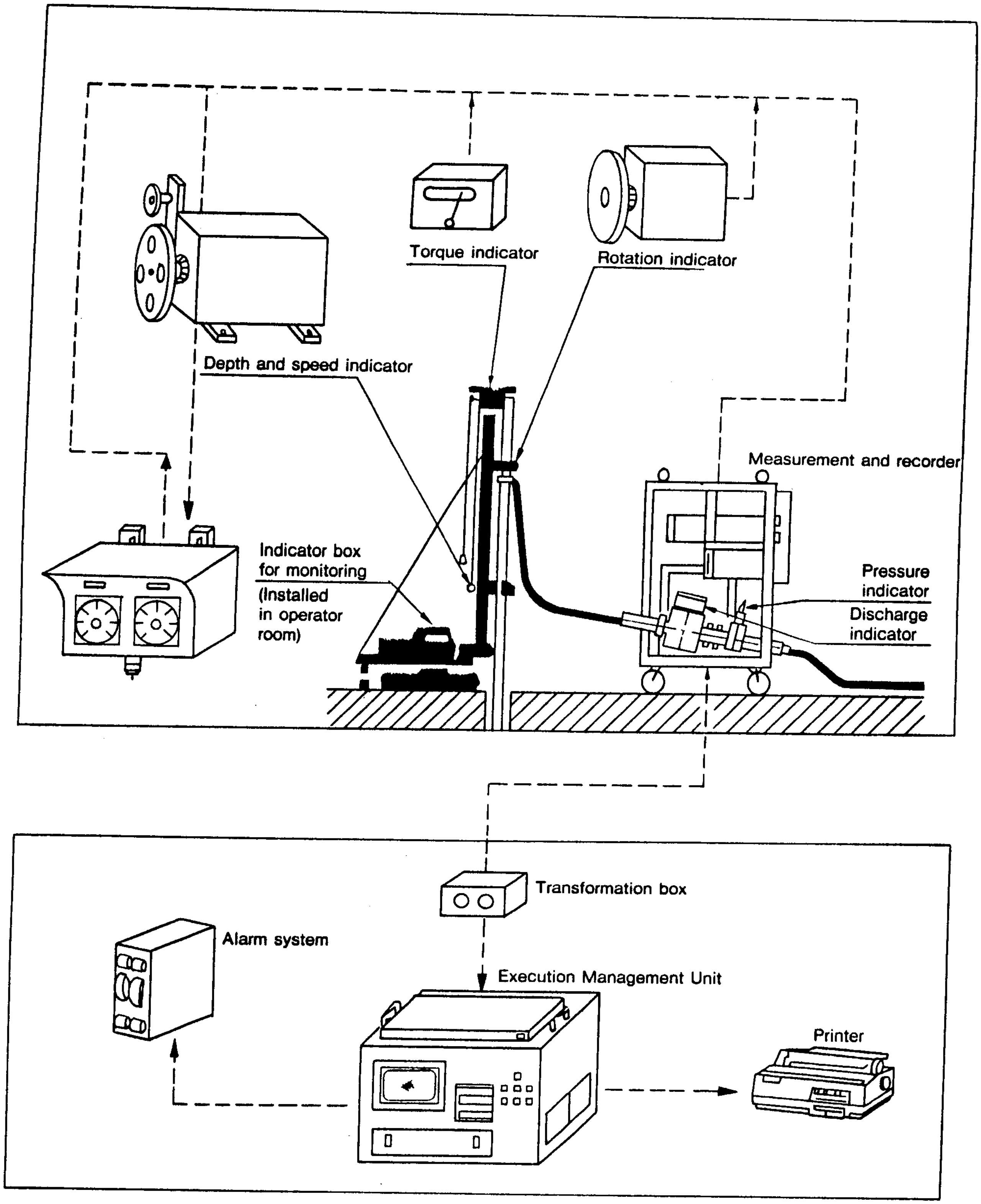


Figure 21. CDM computer control and management systems (Yano et al., 1996).

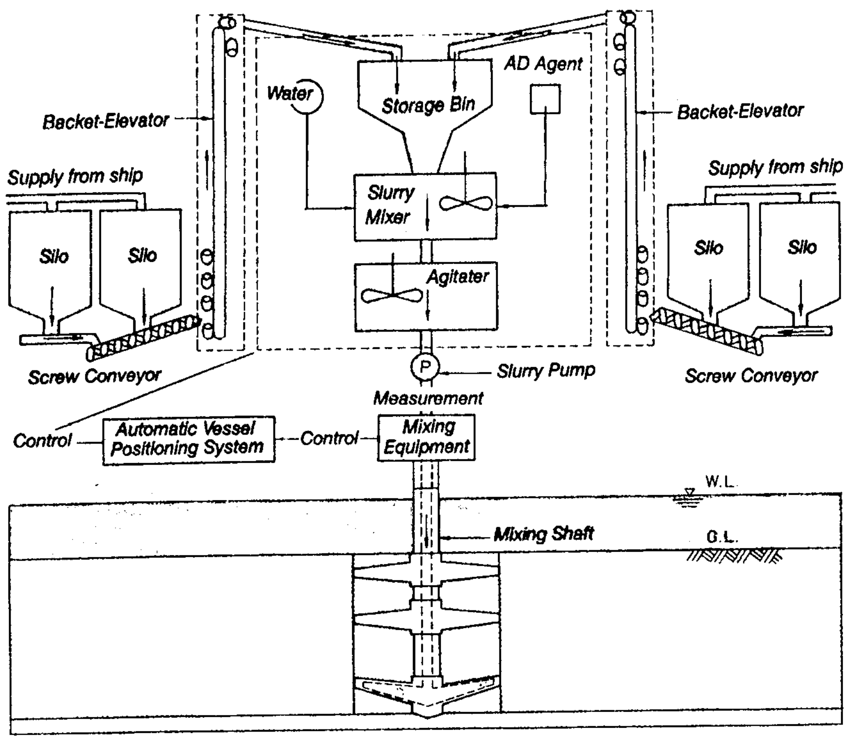


Figure 22. Example of process control for DECOM system (2- to 8-auger marine CDM variant) (Toa Corp., undated, possibly late 1980s).

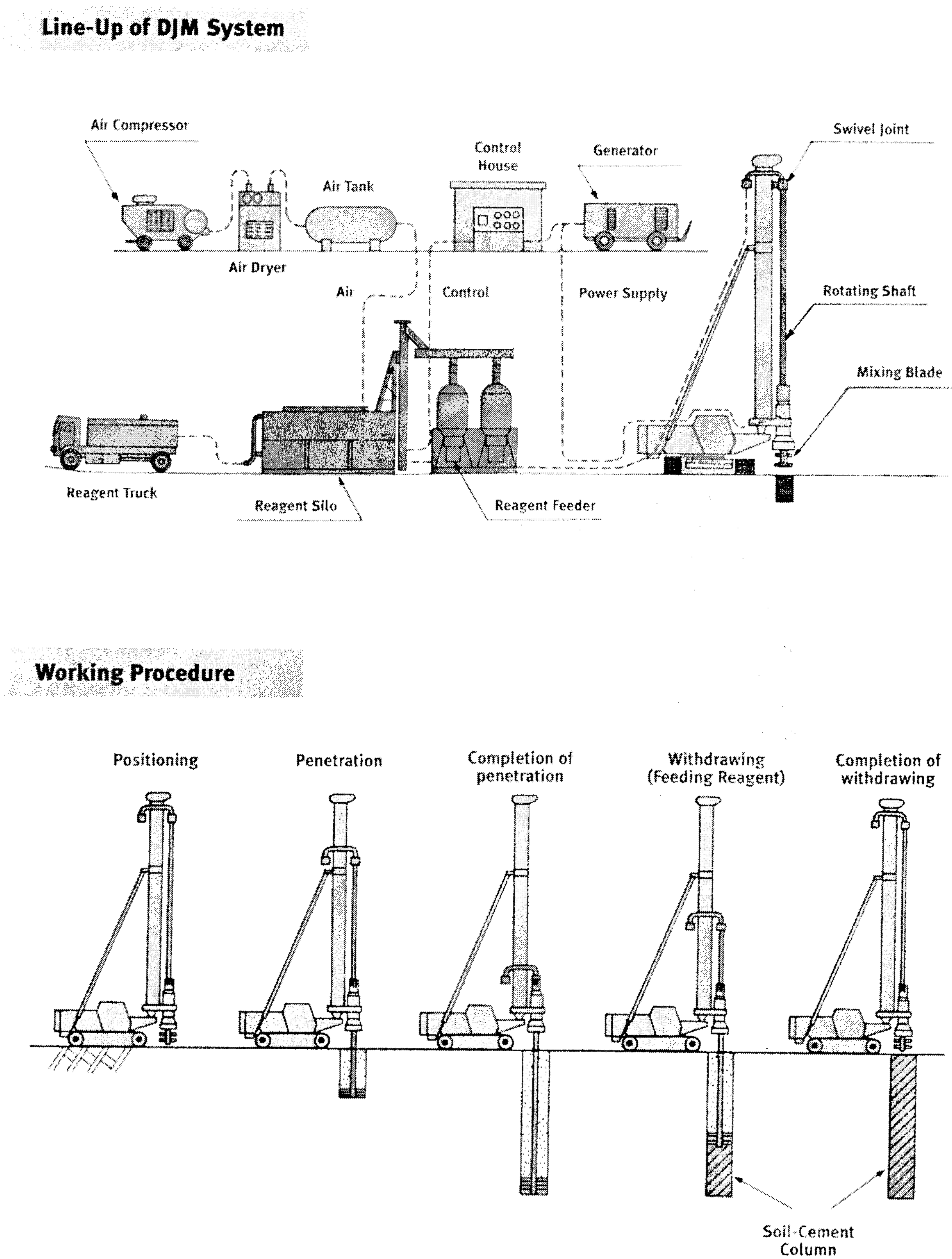


Figure 23. Level 2 process control for DJM systems (DJM Brochure, 1994).

Quality Control

A computer in the cab of the Stabilator LC machine gives the operator precise control of the injection process. The computer also maintains complete records of:

- The quantity of material injected into each column.
- The rate of injection.
- Speed of tool rotation and rise.

The records can be displayed or printed out for real-time inspection and for quality assurance records.

The Stabilator machine mixes the lime and cement in a closed system, eliminating the risk of dust leakage during the installation process.

Under ISO 9000, the Stabilator Lime and Cement Column process has been certified as incorporating QA/QC procedures to assure quality of work.

Figure 24. Quality control processes for lime cement columns (Stabilator, 1996).

Figure 25 (Drilling), and Figures 26 through 28. Level 3 data from Method 20 are shown in Figures 29 through 31.

One example of a commercially available drilling instrumentation package is the one provided by Jean Lutz, of France, and in fact many of the drill parameter recorders, such as Enpasol, used in certain DMM techniques are directly related to the original Lutz developments. Figure 32 shows the system used, the parameters measured, and typical display and printer units. The LT3 unit digitally records the parameters on a high performance memory card without connectors. Data are then downloaded onto a PC by a specific software program. A graphical real time display can be provided through a choice of printers.

Another example of an automatic recording system is that provided by Pile Dynamics, Inc., which is basically an instrument developed for the control of auger cast piling. Information is provided in Figure 33.

Regarding future developments, Yano et al. (1996) reviewed innovative trends in computer controlled CDM work and summarized these as follows:

"The central control system permits high quality uniform pile placement and simplifies daily report preparation and other administrative tasks. The following studies and development activities are now underway order to make the control systems easier to use and deliver even better quality control.

1. Automatic Control of Slurry Discharge. The system will automatically control the amount of slurry discharged by controlling the rotation of the inverter motor on the slurry pump based on comparisons of the actual measured amount with the design values. For every 25 to 50 centimeters of depth, the set values sent to the pump will be recomputed from the integrated data and controlled so that the design values for every meter of depth are satisfied. The pump discharge volume will also be adjusted in stages in response to changes in the penetration and withdrawal velocity.

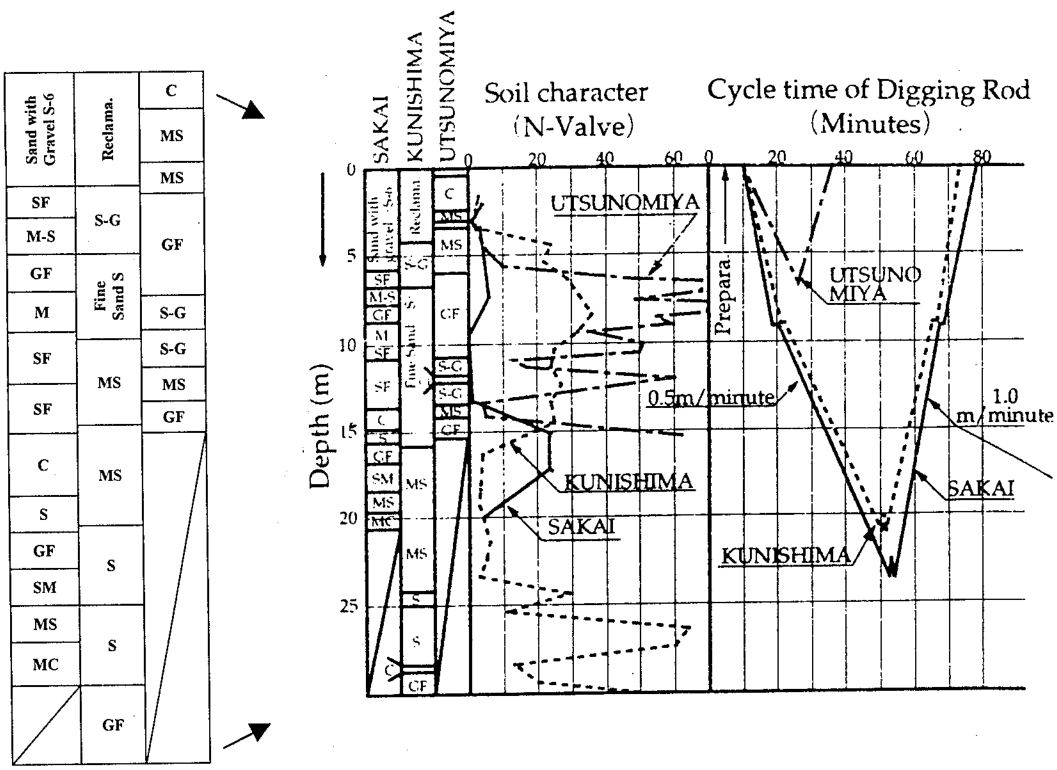


Figure 25. Soil property and cycletime of RAS construction (Isobe et al., 1996).

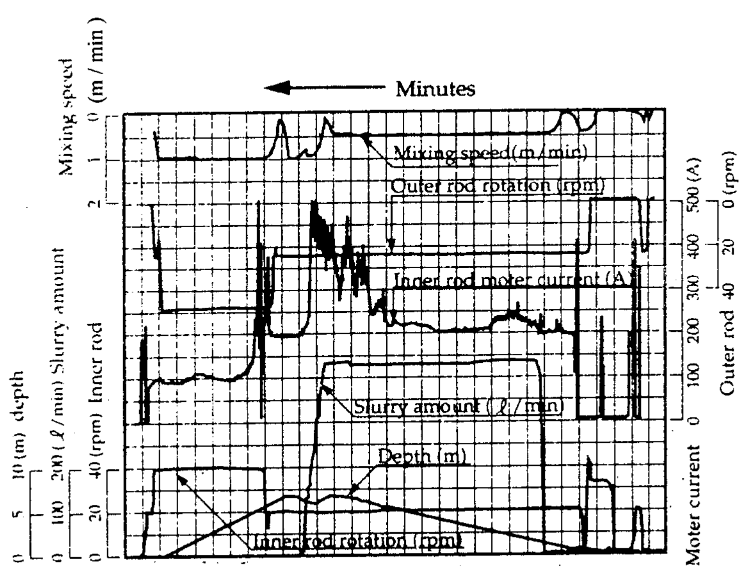


Figure 26. Oscillograph showing RAS construction variables (Isobe et al., 1996).

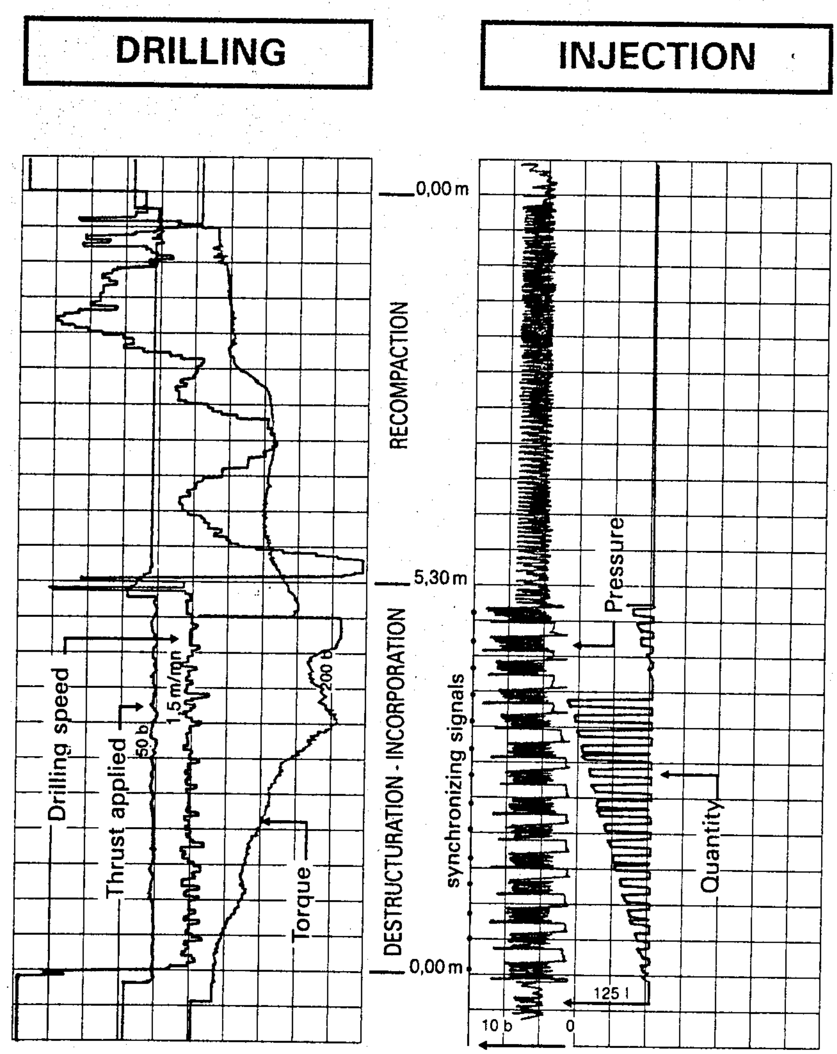


Figure 27. Records of Colmix drilling and injection parameters (Bachy, 1996).

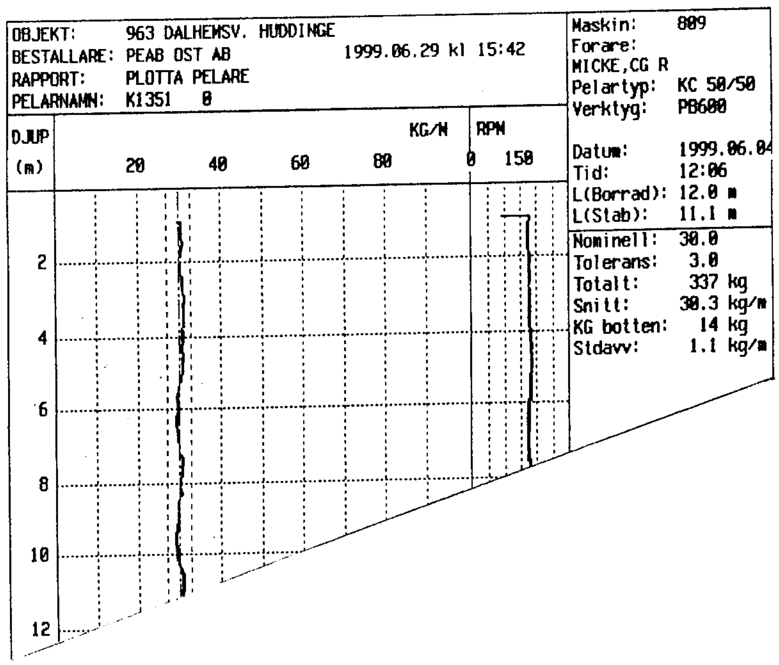


Figure 28. Output graph according to LC-REGREP (recording system of Markteknik's Lime Cement Column method), 1999.

AA-1 Started: Tue Mar 11 08:30-02 1997

Time (min)	Depth (ft)	RotaryFlow (gpm)	RotaryPres (psi)	CrowdPres (psi)	Slurry Density (ppg)	SlurryFlow (gpm)	SlurryPres (psi)
0.1	0.2	135	1222	2041	12.24	48	179
0.2	1.2	135	1211	2021	12.1	47	211
0.3	2.1	134	1354	2029	12.1	47	230
0.4	2.7	134	1427	2047	12.19	47	62
0.5	3.7	133	1445	2075	13.45	51	388
0.6	4.5	129	1112	2102	13.95	159	1939
0.7	5.3	129	1339	2049	13.77	156	2045
0.8	6.3	128	1350	2098	13.2	157	1869
0.9	7.2	128	1233	2092	13.13	160	2258
1	8	129	1279	2074	13.1	160	2005
1.1	9	127	1286	2045	13.09	159	2092
1.2	9.7	127	1522	2076	13.01	160	1965
1.3	10.7	128	1387	2056	12.98	160	2113
1.4	11.4	128	1396	2078	13.12	159	1736
1.5	12.4	128	1326	2099	13.19	159	2145
1.6	13.2	128	1431	2094	13.31	160	2240
1.7	14.2	128	1255	2118	13.43	158	2130
1.8	15.1	128	1295	2126	13.65	158	2009
1.9	15.9	129	1240	2120	13.79	158	2005
2	16.9	128	1235	2112	13.76	158	1834
2.1	17.4	128	1348	2109	13.79	158	1700
2.2	18.4	127	1456	2093	13.77	158	1947
2.3	19.3	127	1476	2091	13.71	156	1889
2.4	20	128	1442	2082	13.7	157	2249
2.5	20.9	128	1326	2064	13.56	157	2162
2.6	22.2	127	1191	2100	13.53	157	2229
2.7	22.3	129	1189	2094	13.48	156	2043
2.8	23	128	1357	2121	13.33	155	2183
2.9	24.1	127	1464	2135	12.73	155	2174
3	24.9	128	1370	2120	12.49	155	2170

Figure 29. Verification documentation for GeoJet (Condon Johnson Associates, 1998).

CJA GEO-JET Production

Inputs Column		Calculated Column	
		Tooling Performance	
Augar diameter (in)	36	Treatment volume per lineal ft. (cf)	7.07
		Pilot Performance	
Diameter of jets pilot (in)	0.125	Pilot unit jet flow (gpm)	16.80
Num of jets on pilot	3	Pilot total jet flow (gpm)	50.40
		Cement solids (ppg)	5.36
		Triplex power required for pilot (hP)	88.21
		Main Processor Performance	
Diameter of jets main processor (in)	0.14	Main processor unit jet flow (gpm)	21.07
Num of jets on main prosessor	2	Main processor total jet flow (gpm)	42.15
		Cement solids (ppg)	5.36
		Triplex power required for main processor (hP)	73.77
		Gel Breaker Performance	
Diameter of jets gel breaker (in)	0.125	Gel breaker unit jet flow (gpm)	16.80
Num of jets on gel breaker	6	Gel breaker total jet flow (gpm)	100.80
		Cement solids (ppg)	5.36
		Triplex power required for gel breaker (hP)	176.43
		Tool Totals	
		Total slurry flow (gpm)	193.34
		Total triplex power required (hP)	338.41
		Triplex output (% of max rpm)	0.83
		Total engine power required (hP)	389.17
Slurry Spec.		**Feed Rate**	
Slurry injection pressure (psi)	3000	Required wt. cement solids per. ft. (ppf)	199.33
Slurry density (ppg)	12.00	Required volulme slurry per ft. (gpf)	37.18
Dry cement solids per soil vol (%)	30%	Total slurry req. for hole (gal)	2974.17
		Feed Rate (fpm)	5.20
Inputs-Feed		**Calculated Feed**	
Number of cutter faces	2	Chip thickness (in/rev/face)	0.34
Tool rotation (rpm)	91		
Job Info.		Drilling time per hole (min)	15.38
		Total retract time (min)	16.00
Hole depth (ft)	80	Total cycle time (min)	46.38
Retract rate (fpm)	5		
Avg. set-up time on next hole (min)	15	Total pay length per rig shift (lf)	620.9
Shift length (min)	360	Total columns per rig shift	7.8
		Quantities	
		Pounds cement per hole	15,947
		Sacks per hole	169.6
		Sacks per rig shift	1,317
		Tons per rig shift	61.9

Figure 30. Production documentation for GeoJet (Condon Johnson Associates, 1998).

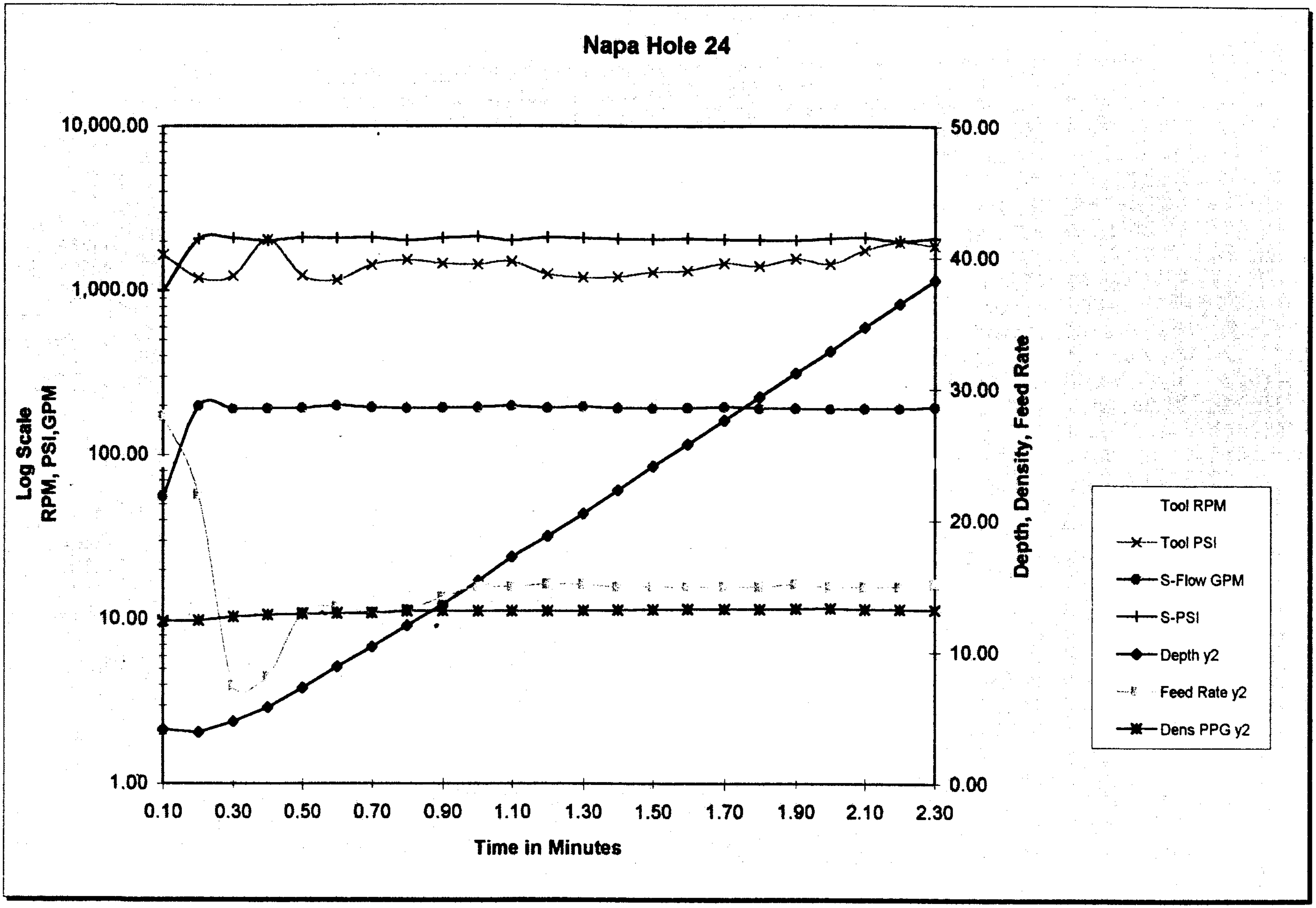
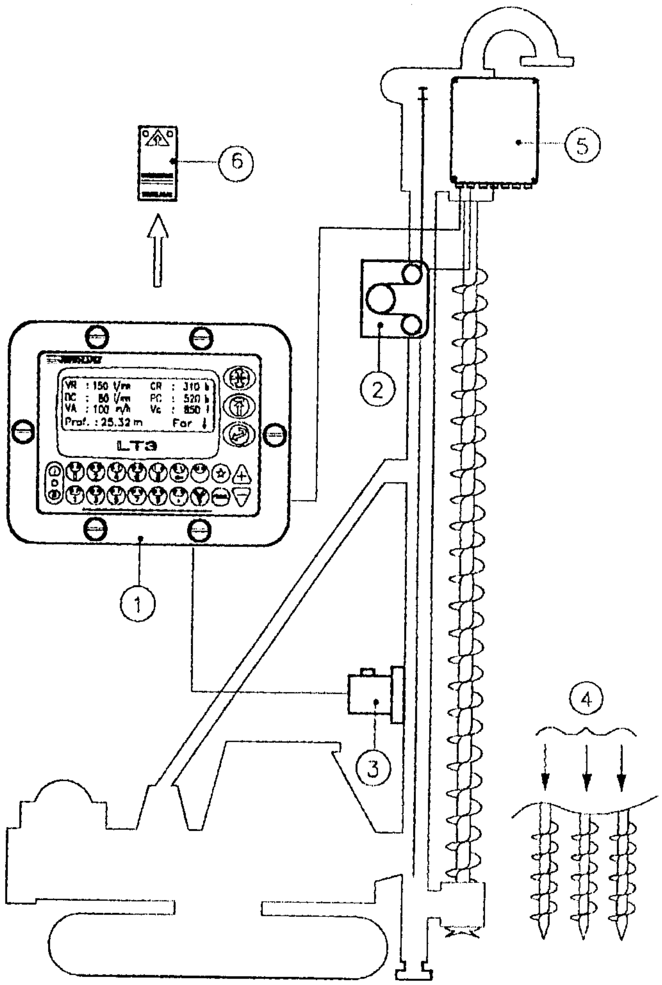


Figure 31. Graphed production data for GeoJet (Condon Johnson Associates, 1998).



① LT3/SMX
Prof ② Profondeur
VA Vitesse d'avancement
XY ③ Inclinaison
CR, VR ④ Couple et vitesse de rotation
PC Pression du coulis
DC, Vc Débit, Volume du coulis
⑤ Boîtier de connexion
⑥ Mémoire numérique

① LT3/SMX
Prof ② Depth and
VA Penetration rate
XY ③ Inclination
CR, VR ④ Torque and rotation speed
PC Grout pressure
DC, Vc Grout flow and volume
⑤ Junction box
⑥ Memory card

① LT3/SMX
Prof ② Tiefe und
VA Bohrfortschritt
XY ③ Neigungsmesser
CR, VR ④ Drehmoment und Umdrehungsge-schwindigkeit
PC Suspensionsdruck
DC, Vc Suspensionsdurchfluß-und-Volumen
⑤ Verbindungsgehäuse
⑥ Speicherkarte

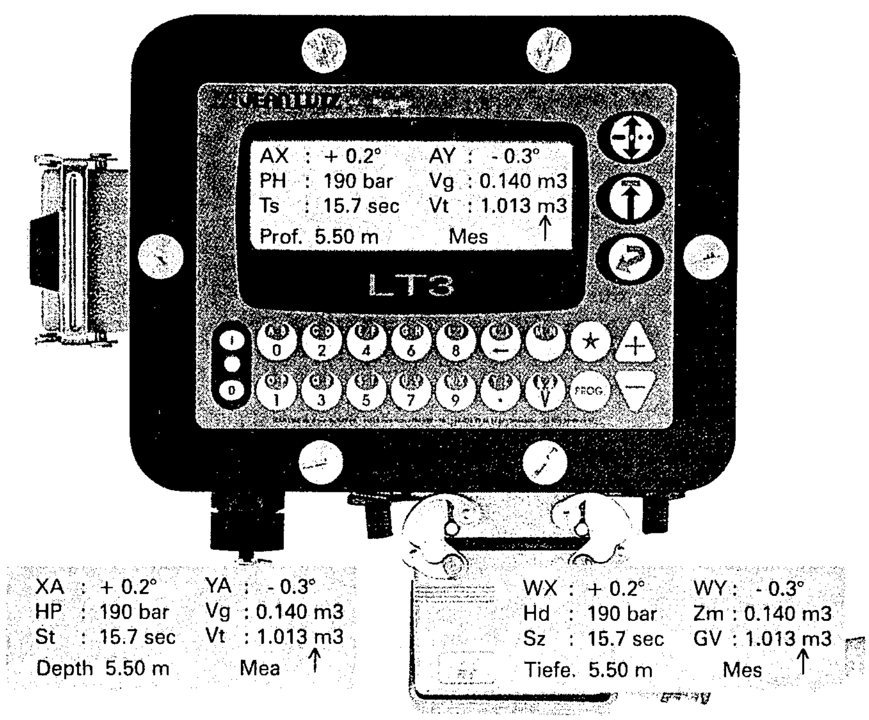
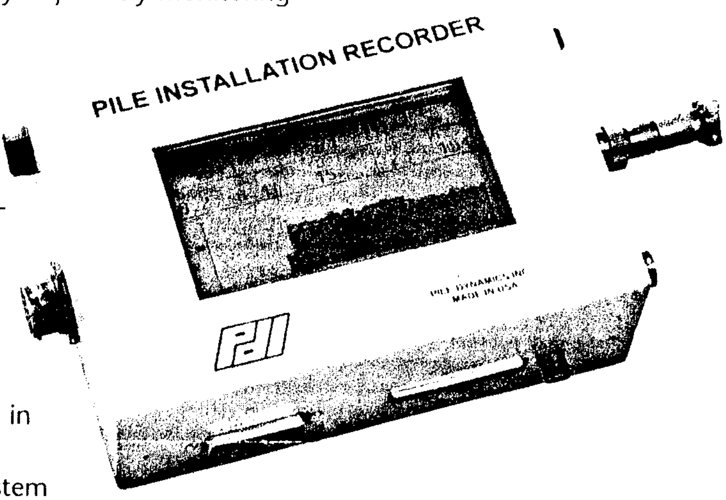


Figure 32. Details of parameter recording and display for Jean Lutz equipment (Jean Lutz, 1998).

Pile Installation Recorder - PIR-A

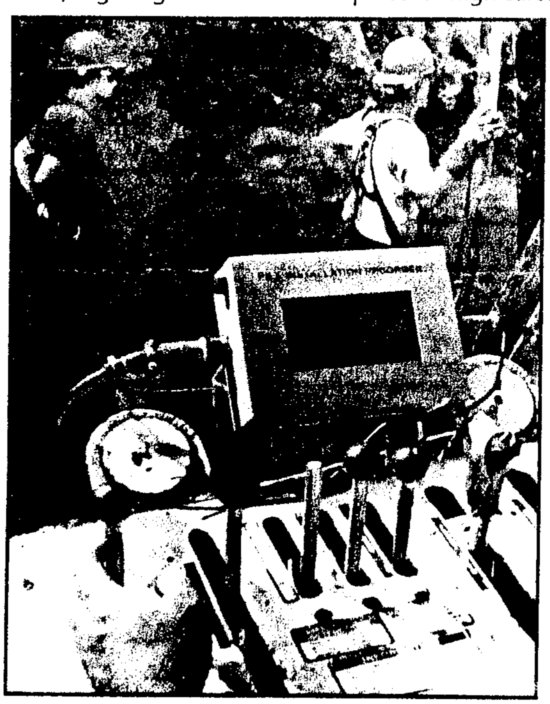
For Improved Quality Assurance of Augercast Piles

- Reduces construction time and cost by objectively monitoring pile installation.
- Adapts to any type of rig configuration (swinging or fixed leads).
- Magnetic flow meter assures accuracy of grout volume measurement.
- Provides the rig operator with installation guidance by displaying accurate grout volume pumped versus depth information.
- Allows immediate correction while shaft is still fluid.
- Reduces risk and improves confidence in Augercast piles.
- Designed with an affordable price; system pays for itself in a very short time.



- Precise measurements of time, volume, pressure, and depth to optimize the grout volume required.
- Simplicity and ease-of-use assures success.
- Results in one page standard summary sheet for each pile.
- Summary of all piles for productivity analysis.

There is a growing need for improved quality assurance in deep foundation installations due to increased design loads. For Augercast piles, increased loads raise the risk due to cross section uncertainties. Augercast pile quality is very dependent on the skill of the crew. If the auger is withdrawn too rapidly, the concrete volume is reduced and the shaft's structural strength may be insufficient. In many cases, higher grout ratios are required or high safety factors are assigned to reduce this risk, increasing Augercast pile costs. Accurately and thoroughly monitoring every pile during installation improves the Augercast pile uniformity and allows the engineer to specify Augercast piles with greater confidence. Augercast piles become cost effective and are accepted in more project sites.



The PIR-A is simple to use. After entering a pile name or number, the piling rig operator presses a single key. Operation and data collection are automatic. Since the volume pumped is graphically displayed and referenced to the minimum acceptable volume per unit depth, the operator withdraws the auger at a rate that assures quality while optimizing efficiency and economy. This increases profits. If the grout volume is low, corrective action can be taken immediately while the grout is still fluid. Anomalous piles are virtually eliminated. Results may be printed in the rig cabin or the office, in detail or as a summary for each pile.

Figure 33. Details of parameter recording and display for Pile Dynamics equipment (Pile Dynamics, Inc., 1999).

The most serious problem to be resolved in developing this automatic slurry pump control system is to find a way to control the pump's own time lag between the time the discharge amount instruction is sent to the slurry pump and the time the pump actually begins to deliver the required amount.

2. Automatic Confirmation of Bearing Layer. The system will automatically confirm the bearing layer by judging the arrival at the bedrock based on electric current measured as the load on the auger and the penetration and withdrawal speed. Data are now being analyzed to determine what type of operation expression should be incorporated to evaluate the current.

3. Execution Management of the Machine's Operation. With the existing centralized control system, a single operator controls and monitors the execution management unit and the flow volume, but if the amount of slurry discharged can be automatically controlled, this person will no longer be needed, and the execution equipment can be managed from a central operator room for the deep mixing machine. However, this means that the execution equipment operator will have to spend more time operating the control equipment. To minimize the workload on the operators, studies are now in progress to develop easy-to-use system based on the operator's sense of sight and hearing by using one-touch panels, voice synthesizers and so on. Centralized control systems will thus be far easier to operate than those in use today.

4. Pile Track Measurements. To ensure that the strength of the overall improved soil satisfies the design values, the work must be executed in accordance with the pile arrangement and lap width stipulated by the design. To achieve this, it is necessary to confirm the manner in which the piles have been formed under the ground. This requires a technique of monitoring and controlling the track of the mixing bit underground to be developed. (Authors' note: it may be assumed that this topic relates to column verticality and straightness.)

These technique problems will be resolved and even more accurate execution work will then be possible."

3.3 Verification Methods for Treated Ground

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

- Laboratory testing of laboratory samples (before construction).
- Wet grab sampling of fluid in situ material (during construction).
- Coring of hardened in situ material (after construction).
- Excavation and exposure, allowing cutting of block samples, if required (after construction).
- Miscellaneous, including geophysical testing (during and after construction).
- Various penetrometer and cone tests as developed specially for Lime Cement Columns in Scandinavia.

It is reiterated that these properties are influenced in detail by many interactive factors, including soil type, amount and type of binder, water/cement ratio, degree of mixing, curing conditions, environment, and age, although the soil characteristics themselves seem to be the most sensitive determinant of variations in strength.

Table 3 also shows the types of tests that various DMM contractors have reported as being used on their projects.

3.3.1 Laboratory Testing

Such testing is conducted to confirm basic mix design assumptions and to demonstrate the effect and impact of the various materials involved (both artificial and natural). Laboratory testing is also clearly useful in establishing baseline parameters, and for investigating in a controlled fashion the nature of the relationships between the various treated soil parameters (e.g., unconfined compressive strength (U.C.S.) and E; U.C.S. and tensile strength; rate of gain of strength, etc.), and the construction variables (Table 6). With respect to temperature, this is related to the size of the treated soil mass, as well as the quantity of binder introduced. In laboratory testing, there is no way to reliably vary and simulate factors III and IV from Table 6, except for the amount of binder and the curing time. Laboratory testing therefore standardizes these factors, with the result that the strength data obtained during such tests are "not a precise prediction" but only an "index" of the actual strength. Likely field strengths can then be estimated by using empirical relationships from previous projects, and by exercising engineering judgment.

Kamon (1996) summarized that U.C.S. data from field samples (q_{uf}) are 20 to 50% those prepared in the laboratory (Figure 34). These data were determined from land projects whereas on the frequently massive marine CDM projects, larger values of q_{uf} than q_{ul} are often obtained due to the in situ "adiabatic temperature rise."

Kawasaki et al. (1996), when investigating work done with the SWING method (Method 17), concluded that laboratory strengths typically averaged two times the field core strengths.

Taki and Bell (1998) also found a reduction in apparent strengths from laboratory to field, with a wider data scatter in the field data (Figure 35), based on cores of treated soil.

3.3.2 Wet Grab Sampling

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

Table 6. Factors affecting the strength increase of treated soil (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.

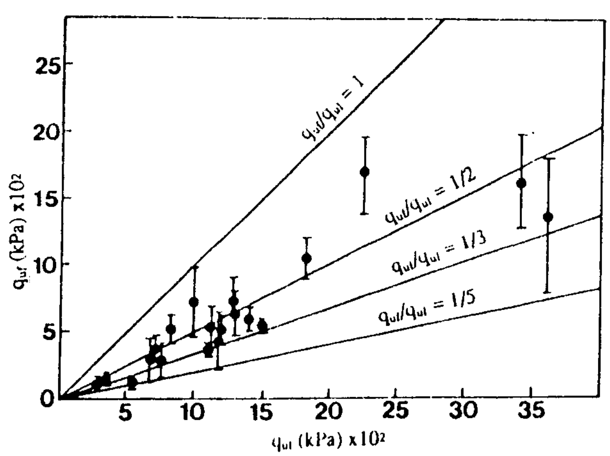


Figure 34. Field strength, q_{uf}, vs. laboratory strength, q_{ul} (Kamon, 1996).

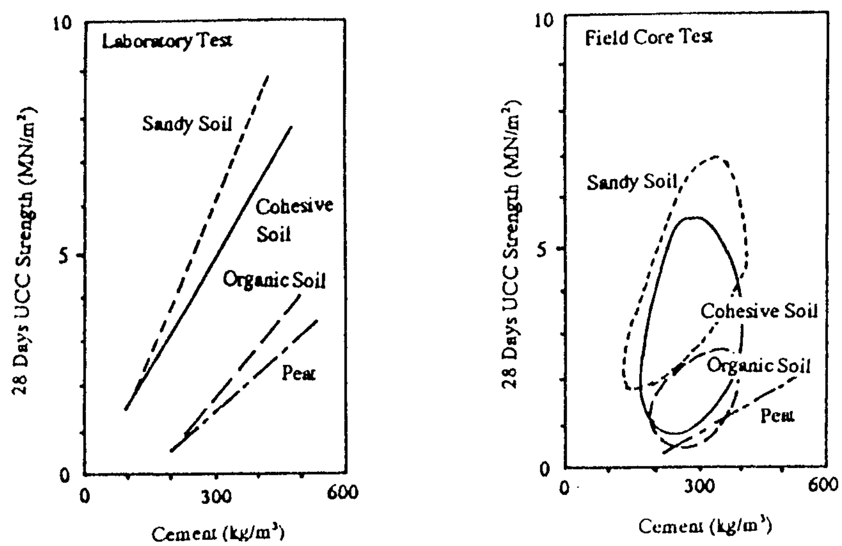


Figure 35. Relationship of U.C.S., cement factor, and soil type (data developed with water/cement ratio of 0.6 to 1.2 and a volume ratio of 23 to 75%) (Taki and Bell, 1998, after Hibino, 1989).

material may prevent the sampler from functioning correctly, and/or from obtaining a wet sample whose composition is truly representative of the overall mixed volume. In this regard, it is typical to screen samples, reflecting the design requirements, and screens with 6- to 12-mm aperture are common.

A good illustration of the usefulness of wet grab samples was provided by Bahner and Naguib (1998). On their DSM project in Milwaukee, WI, wet, remolded samples were obtained for permeability (greater than 14 days' cure) and strength (3, 7, 14, and 28 days' cure) testing. Samples were taken with the tool illustrated in Figure 36 consisting of a steel tube suspending a sample bucket at its tip. The bucket was hydraulically opened and closed. The steel tube was suspended on an excavator. After retrieval, the samples were placed in air-tight containers and wrapped in wet paper towels until testing.

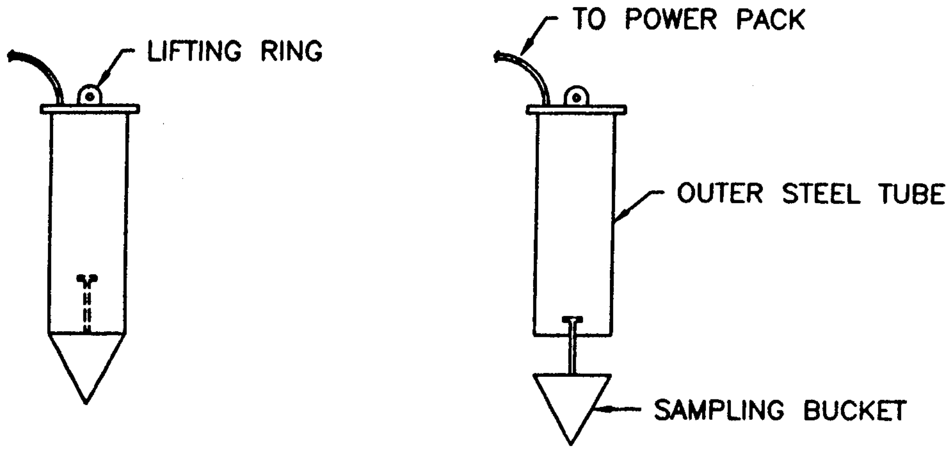


Figure 36. DMM wall sampling tool (Bahner and Naguib, 1998).

3.3.3 Coring

Given that coring is an energetic, local, and invasive technique, even when conducted with the best equipment, skill, and methods (the triple tube core barrel is generally recommended), it is notable that most of the 24 DMM techniques used outside of Scandinavia cite core samples as their prime source of data on treated ground properties in general, and U.C.S. in particular (Table 3), reflecting their largely Japanese origin.

Druss (1998) noted several key elements that promote good and representative core sampling. These include using experienced drillers and logging engineers; taking large diameter cores (greater than 76 mm in diameter); using triple tube methods with very coarse diamond bits to minimize sample washout, and appropriate drilling flush (or mud); and ensuring that the inside surface of the sample tube is lubricated. Lambrechts (1999) observed that the use of wireline core drilling methods improved the quality of core recovery through treated soil of widely varying strength and composition on the Central Artery/Tunnel Project in Boston. The wireline device uses the drill casing to spin and advance the core barrel. Therefore, the "wobble" normally experienced at the top of the core barrel where it is connected to the drill rod string is eliminated. Druss (1998) also prescribed standard methods followed for handling and testing specimens made from wet samples and core samples (Table 7), these procedure having been developed on the Boston Central Artery project.

A useful perspective was provided by Taki and Bell (1998), who wrote (in relation to the SCC method (Method 8)), "Quality control (QC) is required during the installation of soil-cement pile/column to achieve the desired uniformity of the soil-cement mixture and to obtain the design strength. Quality assurance (QA) is obtained from the installation records and from the results of strength tests on representative specimens of the soil-cement. The first step of QC is to develop the mix design. This is usually accomplished by first preparing and testing soil-cement mixtures in the laboratory and then confirming results with a pilot drilling/mixing test program at the project site. The laboratory program should not be expected to produce the same mixing conditions or strength results as the field program. We recommend the emphasis be put on the pilot test in the field. Core drilling is often used to obtain test specimens for quality assurance. Piles to be cored can be randomly selected, but additional core drilling and testing should be performed when questionable soil conditions or mix conditions are observed during installation. Core samples should not be taken exclusively from the middle of the pile but rather throughout the cross section, as shown in Figure 37. Uniformity is dependent upon the degree of mixing and is best with an absence of clods (i.e., unmixed soil masses). Uniformity is evaluated from the core samples by measurement of the amount of soil-clods in Figure 38 (and they recommend less than 5 to 10%). The depths of core samples for strength testing should be measured from the top of the pile and should include intervals containing the weakest soil layers. The top of the pile is often the most severely loaded and therefore should usually be included in the testing program. Core samples should be visually examined for continuity and uniformity of the soil-cement mixture, and for strength. Continuity is defined as the percentage of continuous unbroken, full diameter core. Continuity of the core should be more than 95% in sandy soil and more than 90% in cohesive soil. Table 8 is QC/QA testing items for soil-cement pile work in the field." (Figure and table numbers in this quote were changed to conform to numbering in this document.)

There is healthy debate as to the relationship of unconfined compressive strengths measured from cores and those cast from wet grab samples. It is perceived in some quarters that core

Table 7. Bechtel/Parsons Brinckerhoff Standard Test Method for Coring, Sampling, and Testing of Soil-Cement

I SCOPE

This test method determines the compressive strength of cylindrical soil-cement specimens such as molded cylinders and is limited to soil-cement mixtures placed by mechanical auguring (DSM) or jet grouting (JG).

II APPARATUS

Pan with removable quartering divider
Small steel sample pan
Calibrated sieving screen
Scoop
Digital Scale
Waxed cardboard cylinder molds (3"x 6")
Rubber mallet
Strike-off bar
Plastic cylinder bags, rubber bands, and identification tags
Sulfur capping compound and apparatus
Compression testing machine conforming to AASHTO T22

III SAMPLING AND PREPARATION OF TEST SPECIMENS (DSM AND JG)

1. Obtain from contractor a wet-grab or hydraulic displacement sample from selected depth(s)
2. Transport sample from the field to the Drilling lab
3. Pour material over ½" sieving screen into sample pan
4. Push retained semi-solid material through screen into sample pan
5. Distribute clay and semi-solid particles evenly into pan.
6. Use one quarter of material per cylinder
7. Fill waxed cardboard mold in 3 layers; vigorously tap sides of mold with rubber mallet 10 to 15 times to remove entrapped air after adding material for each layer
8. Finish top of cylinder with strike-off bar
9. Assign an identification number to each cylinder
10. When required, weigh each prepared cylinder
11. Calculate unit weight of soil-cement based on average cylinder weight and volume of cylinder mold
12. Place plastic bag over top of cylinder molds and secure with rubber band and tag
13. Place specimen in cure-room; leave on level cart over-night, cut top of bag off, and move to numbered stationary shelf on following day
14. Maintain cure-room temperature at 100°F and 95% humidity

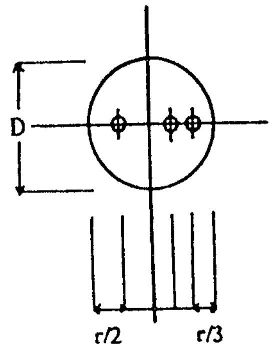


Figure 37. Coring locations (Taki and Bell, 1998).

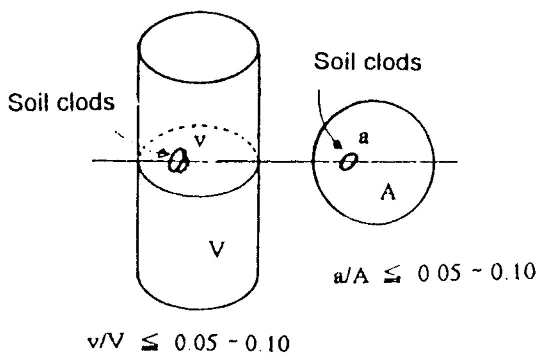


Figure 38. Evaluation of uniformity (Taki and Bell, 1998).

Table 8. QC/QA testing items for soil-cement pile columns (Taki and Bell, 1998).

	Subject	Test Items	Instrument	Frequency
QC	Usage of cement	Total weight	Delivery record	Daily
	Cement grout	Specific gravity	Mud balance	Each batch
	Injection of grout	Volume	Flow meter	Each pile
	Mix condition	Drilling speed Rotation speed Wet sampling	Record Record Trap door	Each pile Each pile 1 to 2 times/day
	Pile length	Shaft length	Drill stem	Each pile
	Pile diameter		Tool diameter	Daily
QA	Continuity	Core drilling	Visual	Random sampling
	Uniformity	Core drilling	Visual	Random sampling
	Strength	Core drilling	UCC test	Random sampling

samples will provide lower strengths given the potential mechanical damage caused to the core during drilling and extraction. (However, it is typical to select only the "better" core samples for testing and this will automatically provide higher apparent data). This argument is refuted by Taki and Yang (1991), who produced data (Figure 39) from various soil types that show that the core strengths were about twice those obtained by samples made from wet grabs. Their view is supported by Burke (1997), who found that on a DMM project in soft clays the core samples always gave higher strengths than wet grab samples, while those data were only 50% of laboratory strengths but with a wider variation. Although pre- and post-construction CPTs claimed to be possible for in situ strengths less than 7 MPa, if in situ strength is expected to average more than 3.5 MPa, Burke considers coring feasible (minimum diameter 76 mm). Recovery rates can vary from 25 to 100% depending on mixing parameters and soil strengths. Burke found that, with his particular WRE technique (Method 15), the grout:treated soil strength ratio was about 4. Jasperse (1989) provided data (Figure 40), which also indicated that, in general, core strengths at 45 to 59 days were equal to or stronger than wet grab samples tested at 56 days.

Mizutani et al. (1996) found that core strengths were only 60 to 75% those of laboratory mixed samples. They estimated that 60 to 80% of the laboratory strength can be achieved in the field with "fairly good" quality control. Typical field DMM coefficient of variation is 20 to 40%.

Taki and Bell (1998) published data on their WRE method (Method 8) illustrating the relationship of U.C.S., cement factor, and soil type (Figure 35), which highlighted also the difference in value and quality between laboratory and core data, which, as discussed above, appears in contradiction to Taki's earlier work (Figure 39) based on a WRS method (Method 2).

Isobe et al. (1996) conducted several field tests using a WRE method (Method 10) in sands and silts at Kunishima, Japan. Cores were taken at various positions across the face of differently sized columns (Figures 41 and 42) and data were also provided of the relationship (Figure 43) between U.C.S. and the "boring parameter" – a measure of drilling energy requirement as recorded by the Enpasol equipment. The authors found that strengths were higher in centrally cored samples, but decreased by up to 50% toward the perimeters of the DMM columns.

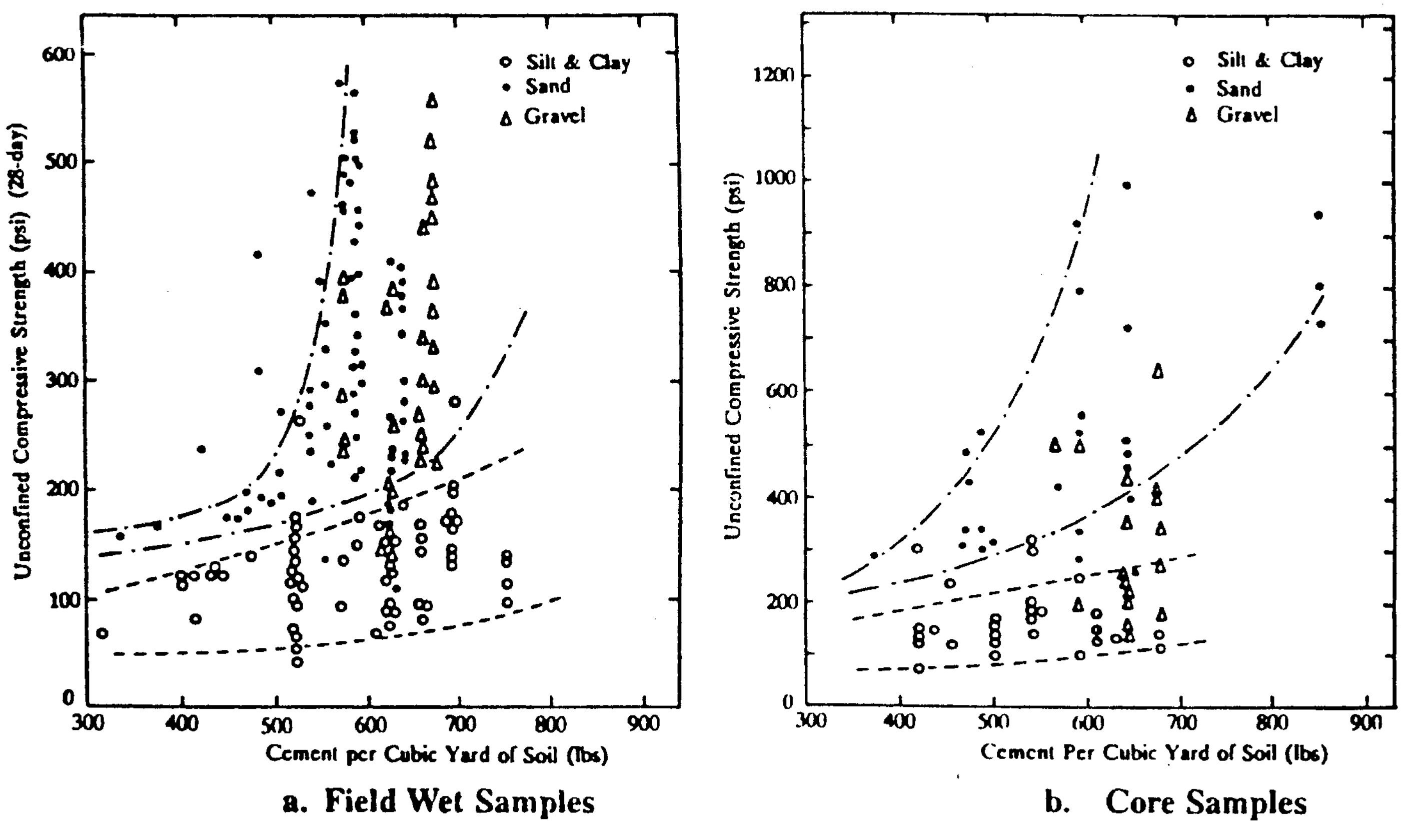


Figure 39. Strength of soils treated by SMW method (Taki and Yang, 1991).

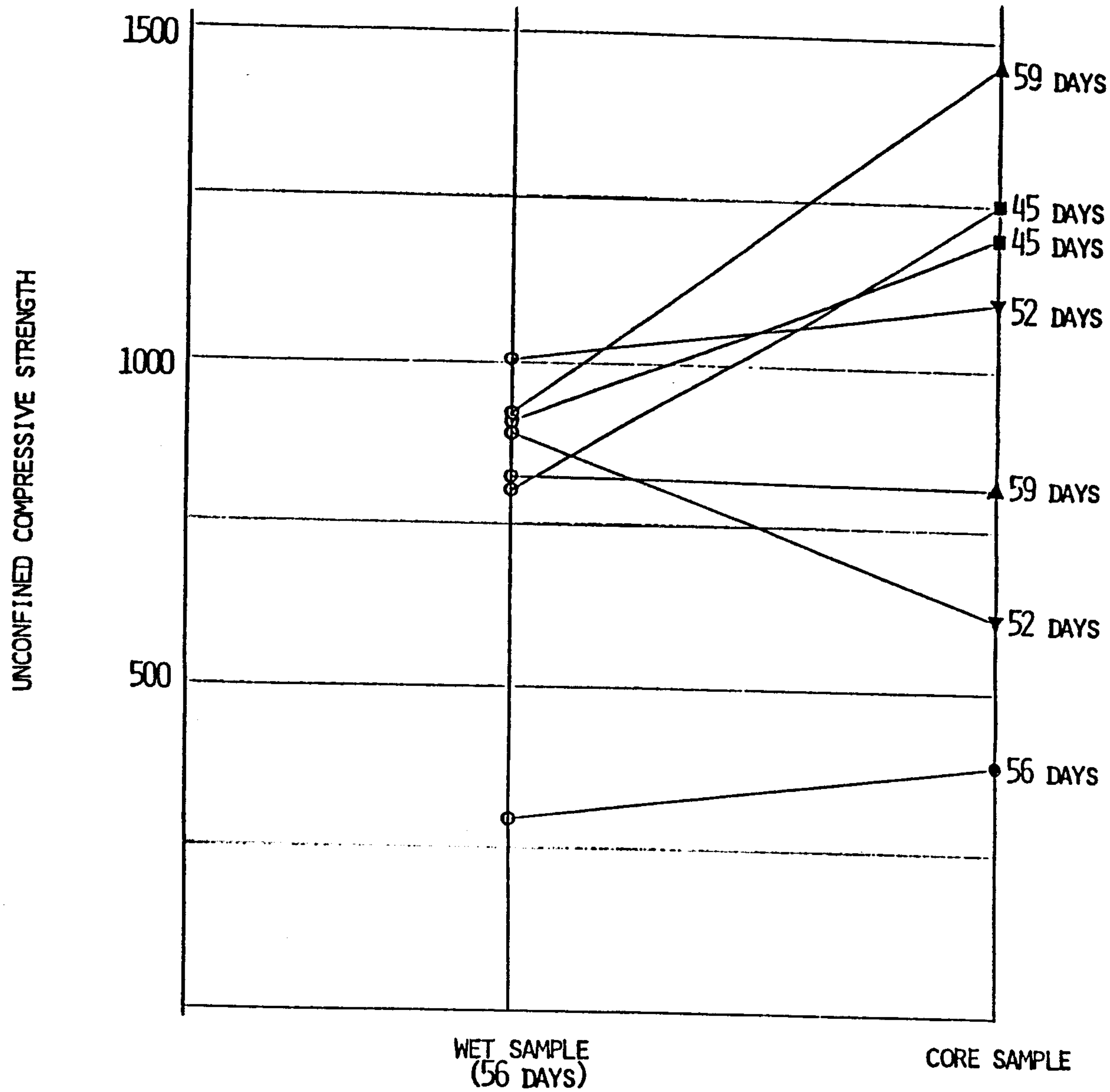


Figure 40. Wet sample strength vs. core sample strength
(Jackson Lake Dam, WY) (Jasperse, 1989).

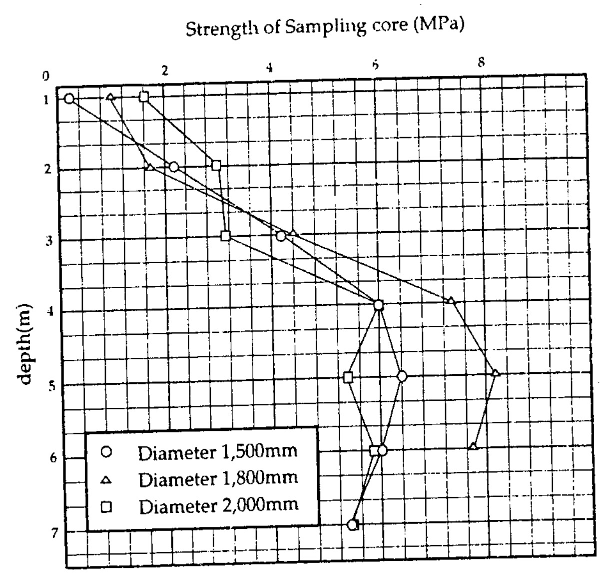


Figure 41. Strength of sampling core from the three columns of ¼ diameter from center (Isobe et al., 1996).

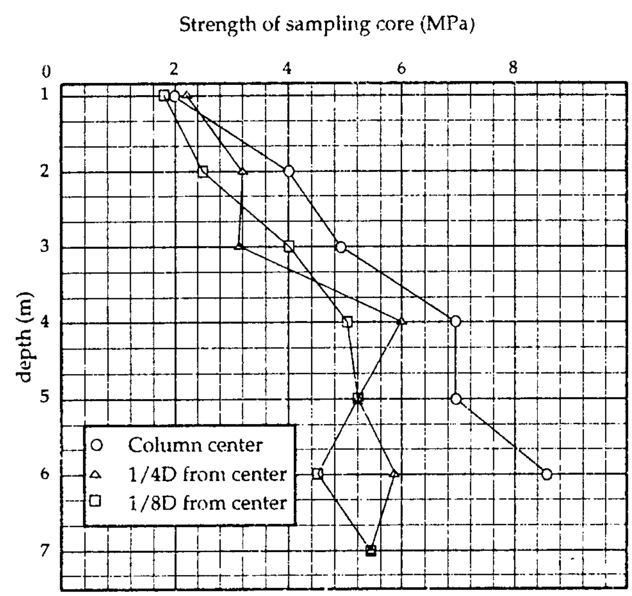


Figure 42. Strength of core samples from 3 different positions with the column (2,000 mm)(Isobe et al., 1996).

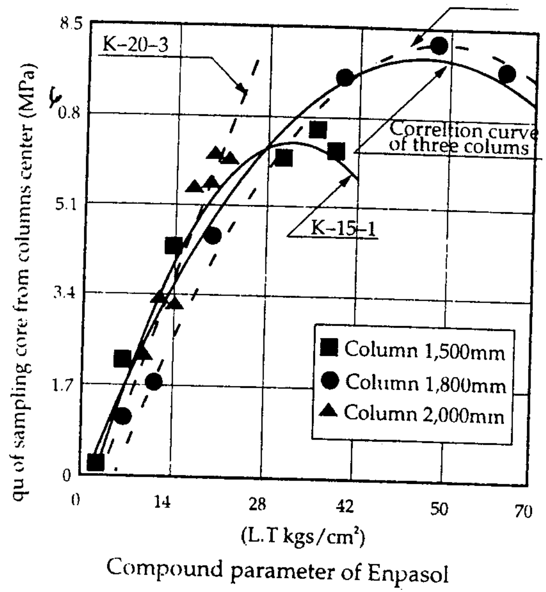


Figure 43. Relationship between q_u and Enpasol parameter (Isobe et al., 1996).

Okumura (1996) stated that, for major Japanese DMM projects, it is typical to core one hole per every 10,000 m^3 of treated soil (marine projects) and 1 per 3,000 m^3 (land projects).

Regarding future developments, Sugawara et al. (1996) presented a most interesting paper on their attempts to produce an improved core sampler. Given the lack of homogeneity of treated ground, their drilling current methods and equipment (Figure 44) usually provided disturbed samples (Figure 45) resulting from a variety of mechanical problems. The goals in developing their new sampler shown in Figure 46 were:

- To prevent vibrations.
- To prevent rotation of core in the inner barrel.
- To reduce stress relief near the bit.
- To reduce friction on the core.

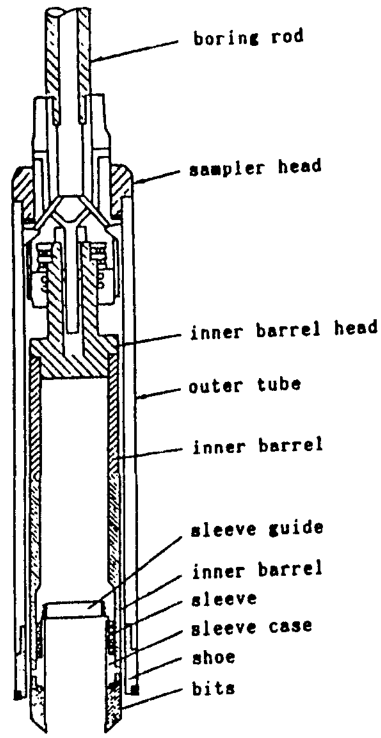


Figure 44. Features of a current core sampler with pre-cutting shoe, standardized by Japanese Society of SMFE (Sugawara et al., 1996).

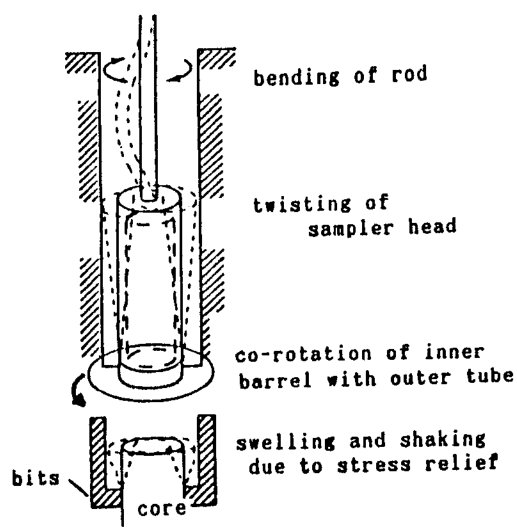


Figure 45. Problems of the current core sampling method with a single rod (Sugawara et al., 1996).

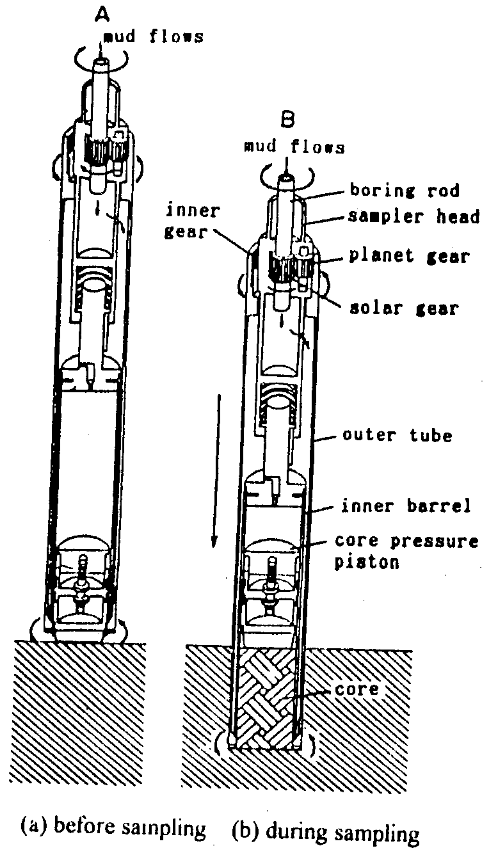


Figure 46. Features of a new core sampler: planet gear and core pressure piston (the core retaining part with sleeve is omitted in this figure) (Sugawara et al., 1996).

Tests were conducted in treated lightly overconsolidated clay (cement factor = 110 kg/m^3) and loose sand (cement factor = 130 kg/m^3) for 1-m diameter columns installed by both CDM and DJM methods. After coring, shear wave velocity (v_s) was measured, as was density. Cores from the new sampler were of higher RQD than those from the conventional equipment. Data are provided on U.C.S. (q_u) and shear wave velocity in Figure 47. Although the trend of the U.C.S. data from the new sampler was in both cases closer to that of the v_s data than the old sampler's data, a considerable variation was still noted. The relationship was therefore explored between q_u and the maximum value of shear modulus at small strains, G_{max}, as determined from:

$$G_{max} = \rho v_s^2$$

where ρ = mass density and v_s = shear wave velocity.

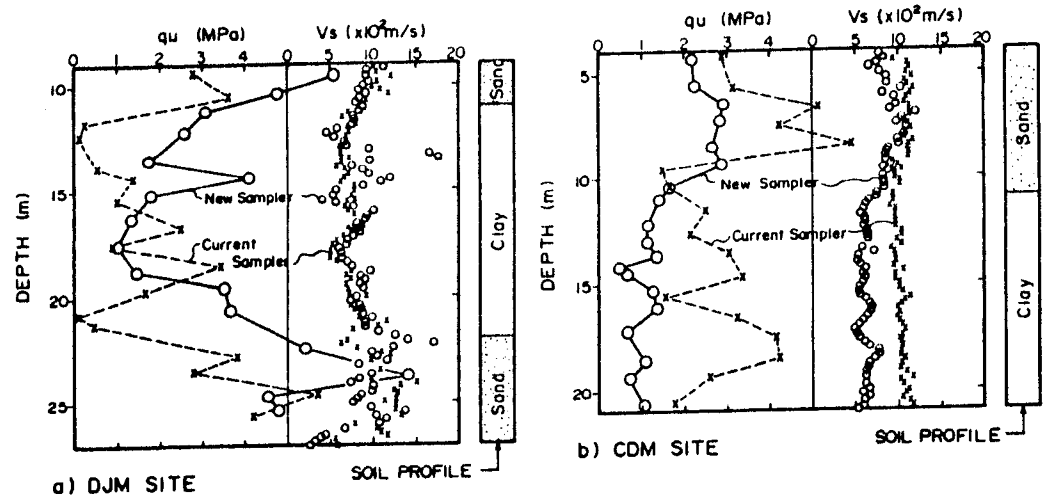


Figure 47. Unconfined compressive strength, q_u, and seismic S-wave velocity, V_s, versus depth (Sugawara, et al, 1996).

The relationship between G_{max} and q_u is shown in Figure 48: many test values with the traditional sampler lie below the 1 x 10^{-3} line, indicating that shear strength is being underestimated, whereas data from the new core sampler appeared to be more accurate. The data in Figure 49 also illustrate clearly that the new sampler generally gives cores that produce higher strengths.

In conclusion, the authors also note that the "shape of the drilling bits needs also to be improved … to obtain better quality core samples."

3.3.4 <u>Exposure, Extraction, and Block Sampling</u>

The opportunity to expose the treated ground allows all parties to observe column shape, homogeneity, diameter, nature of column overlap, and so on. It also permits samples to be taken with different shapes, sizes, or orientations from those that can be obtained by vertical coring.

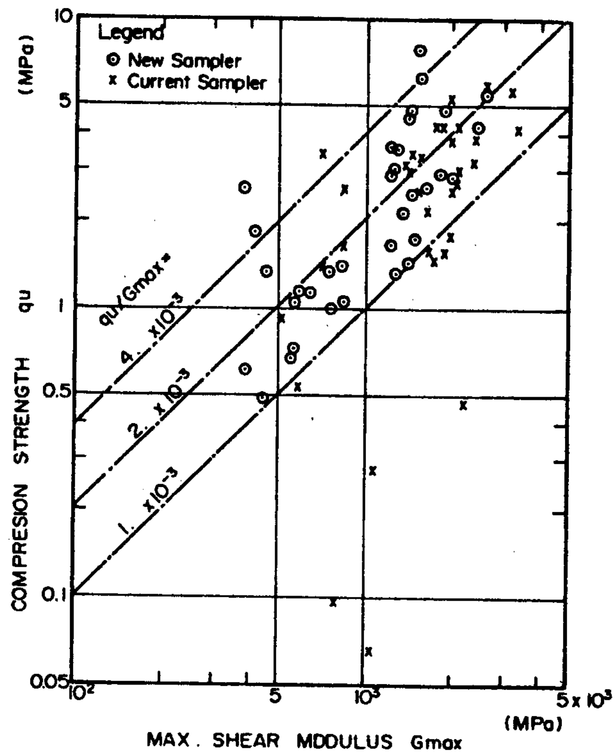


Figure 48. Relationship between unconfined compression strength, q_u, and maximum shear modulus, G_{max} (Sugawara et al., 1996).

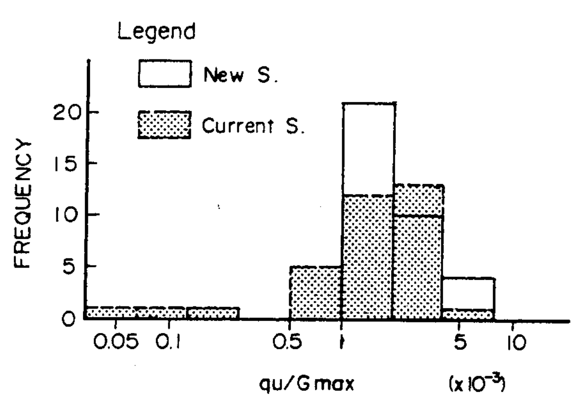


Figure 49. Histograms of q_u/G_{max} (Sugawara et al., 1996).

The value of this kind of testing is underlined when it is recalled that important technical goals of any DMM operation are to provide a uniformly treated mass, with minimal lumps of soil or binder, a uniform moisture content, and a uniform distribution of binder throughout the mass. Exposed treated soil can be sprayed with phenolphthalein solution to help indicate the presence of cement in the mass.

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

Again, the major drawbacks to such exercises are principally cost, time, and site logistics, but on certain projects of critical size, complexity, and significance such as the DMM work conducted at the Fort Point Channel in Boston, MA, exposure is a vital element in verification, both as a pre-production measure, but also as a special demonstration during construction. Burke (1997) is of the opinion that the most efficient method of quality evaluation is to drill a shaft into overlapping columns to allow visual observation of integrity, homogeneity, and sampling of the mixed soils, and therefore to resolve any apparent anomalies identified by coring.

3.3.5 Geophysical and Miscellaneous Methods

Depending on the nature, purpose, and extent of the treatment, a variety of miscellaneous methods have been reported. For example, Methods 11 and 19 both have been developed to reduce adjacent ground and structural movements: inclinometer and borehole extensometer testing results have therefore been reported. Similarly, in those methods (e.g., 15 and 23) focusing on very soft clays and low strength treatment, CPT/SPT testing may be conducted before and after mixing, while most recently, Esrig (1999) described the value of routine pressuremeter testing to accurately indicate in situ shear strengths (Figures 50 through 52). (Druss (1998) also reported an apparent increase of 20 to 30% over data obtained from cores in treated clay when using the pressuremeter.) In higher strength materials, "sonic velocity measurements in three dimensions" have been conducted to verify quality of treatment (Method 4).

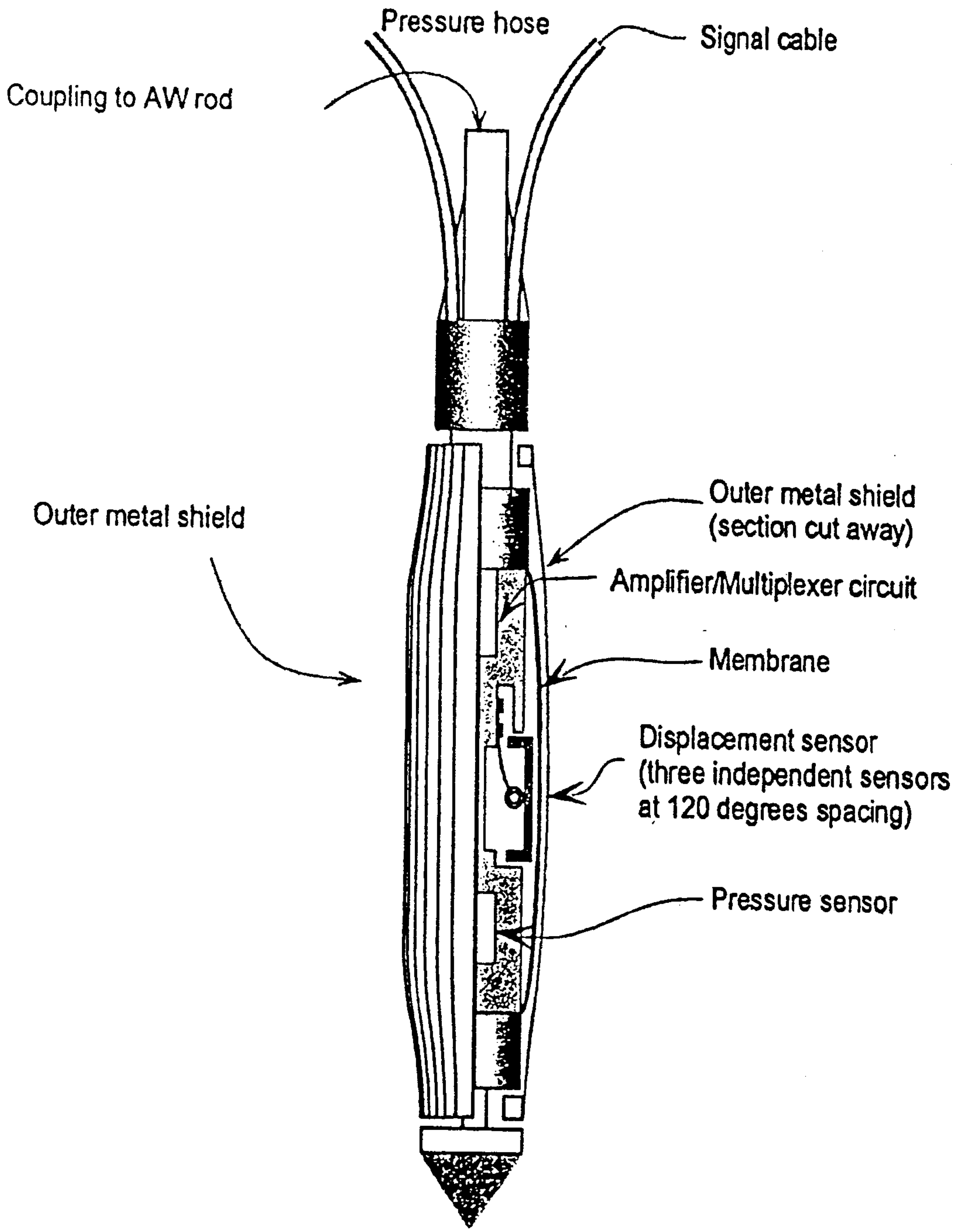


Figure 50. Schematic of pressuremeter (Esrig, 1999).

Hole	Test	Depth from ground surface to center of probe (ft)	Average Shear Modulus, G (psi)	Maximum Shear Modulus, G (psi)
1 (North)	2	10.75	870	5,500
1	1	13.75		
1	3	16.25	2,100	10,510
2 (Center)	5	8.25	3,000	14,500
2	4	9.75	5,000	23,000
2	7	11.25		
2	6	13.25	540	5,400
3 (South)	9	8.25	1,250	14,500
3	8	9.75	1,500	7,300
3	11	11.75		
3	10	13.25	1,900	23,000

Figure 51. Pressuremeter results from BART, San Francisco, showing shear modulus (Esrig, 1999).

Hole	Test	Depth from ground surface to center of probe (ft)	Limit Pressure (psi)	"Shear Strength" (psi)
1 (North)	2	10.75	260	90
1	1	13.75		
1	3	16.25	500	170
2 (Center)	5	8.25	680	206
2	4	9.75	480	90
2	7	11.25		
2	6	13.25	180	60
3 (South)	9	8.25	400	140
3	8	9.75	390	130
3	11	11.75		
3	10	13.25	380	80

Figure 52. Pressuremeter results from BART, San Francisco, showing unit pressure and shear strength (Esrig, 1999).

Hane and Saito (1996) reported on the use of shear wave seismic tomography to explore a treated sandy soil mass, underwater. In this case, the increase in velocity was from 200 to 500 m/s (untreated) to 950 to 1200 m/s (treated), with a very small velocity contrast indicating homogeneity of treatment. These data were consistent "with other mechanical tests."

Hiraide et al. (1996) also assessed the strength of treated ground using shear wave velocities. In situ data were compared with those obtained in laboratory tests on 65-mm diameter cores, from which U.C.S. values had been obtained. Based on the correlation, between U.C.S. and v_s (shear wave velocity, reflecting elastic modulus in situ), the strength of the treated soil mass was "successfully" assessed by geophysical means. Figure 53 shows the relationship between shear wave velocities, v_{SF} (field) and v_{SL}(laboratory), for cores at the same depth, and the authors concluded that v_{SF} = 1.077 x v_{SL}. Figure 54 shows the relationship between elastic modulus E_L (at very small strain) and U.C.S. (E_L = 1250 times U.C.S., unrelated to soil type). Figure 55 shows field U.C.S. as determined by in situ shear wave measurements. The authors claim "success" even in cores with RQD = 0.

Imamura et al. (1996) described the use of a borehole resistivity profiler in evaluating the quality of DMM columns. The method provides a two-dimensional image around a borehole. The change in properties before and after treatment can be demonstrated via this single hole technique. The resistivity is strongly influenced by the porosity, the water content, and the water resistivity. Figure 56 shows the column layout on the site where two resistivity profile boreholes were drilled: one in the middle of the mid pile, the other 6 m away in untreated ground. Figure 57 shows a fair relationship between U.C.S. (q_u) and resistivity, and Figure 58 shows the trend between water content and resistivity. The "before and after" resistivity profiles (Figure 59) indicated the shape and quality of the treatment.

Nishikawa et al. (1996) investigated the use of P- and S-wave logging, and SPT testing, and compared the results with U.C.S. data on three DMM sites in Hokkaido. They pursued this path

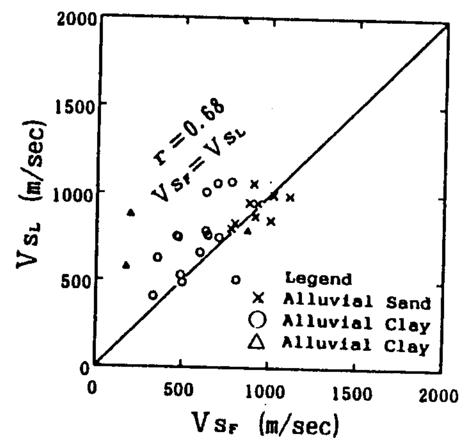


Figure 53. Relationship between v_{SL} and v_{SF} (Hiraide et al., 1996).

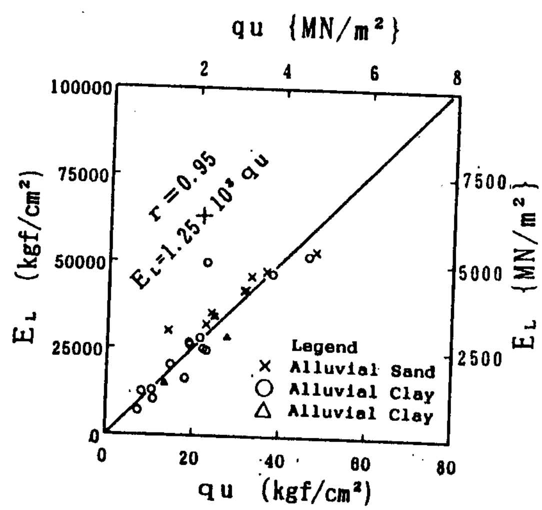


Figure 54. Relationship between E_L and q_u (Hiraide et al., 1996).

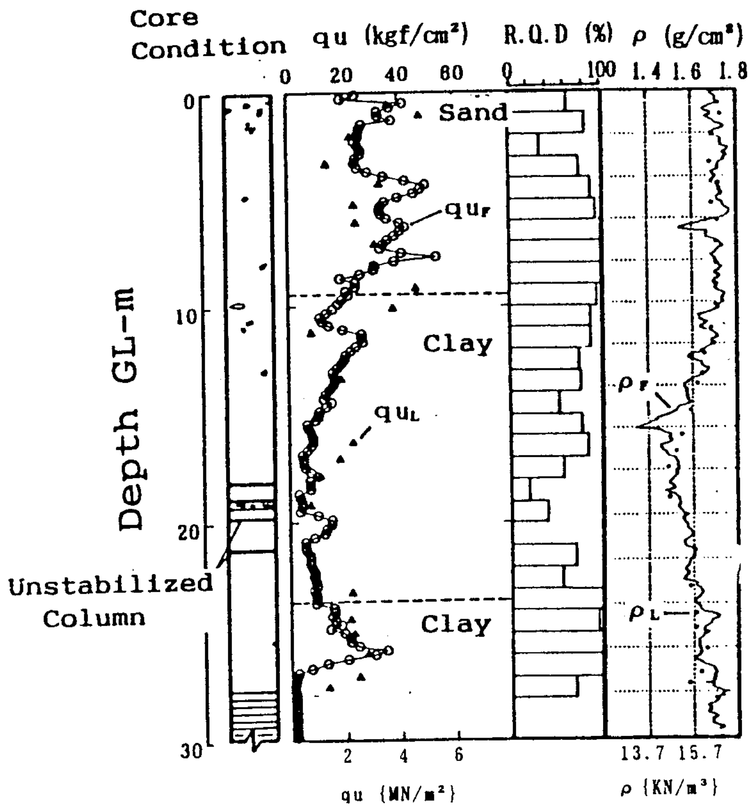


Figure 55. Assessment of strength of stabilized soil column (Hiraide et al., 1996).

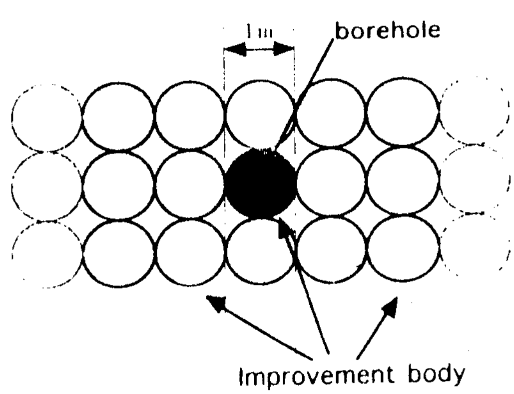


Figure 56. Field arrangement of the test area (Imamura et al., 1996).

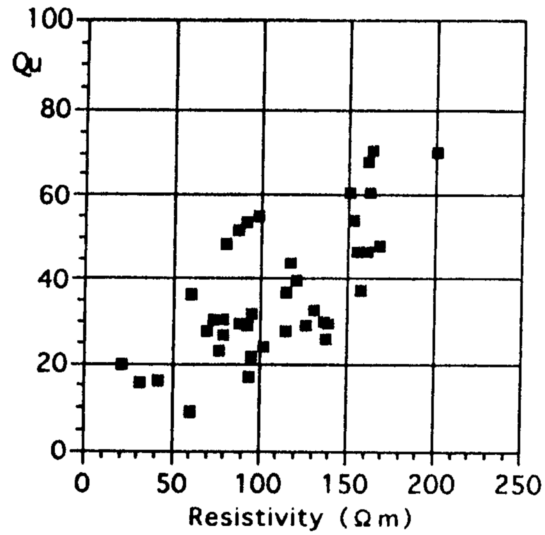


Figure 57. Relationship between compressive strength, q_u (kgf/cm^2), and resistivity (Imamura et al., 1996).

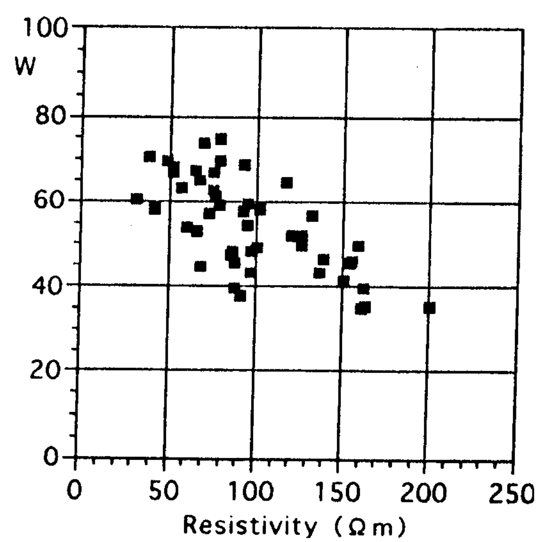


Figure 58. Relationship between water content, W (%), and resistivity (Imamura et al., 1996).

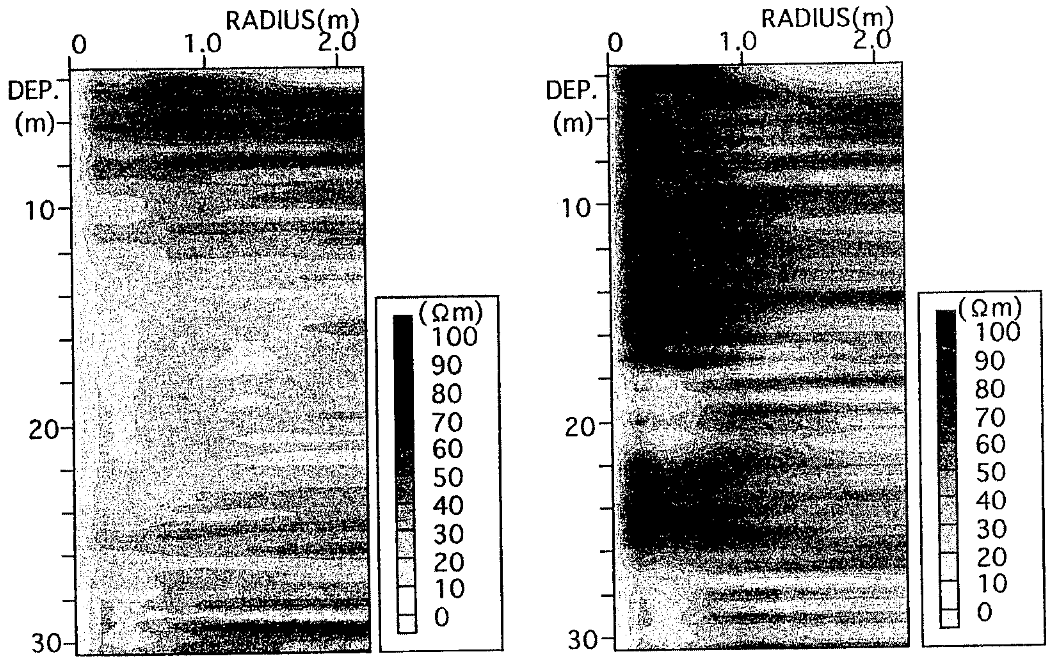


Figure 59. a) Result of borehole resistivity profiles (natural ground), and b) Result of borehole resistivity profiles (improved ground) (Imamura et al., 1996).

because of concerns about "cracks" in core samples reducing the validity of core testing, and also for economic reasons. Data from the three sites are shown in Figures 60 and 61: the first two were treated by CDM, the third by DJM. Coring was done by triple tube. Figure 62 shows the relationship of U.C.S. to their P- and S-wave profiles, and Figure 63 shows the relationship between SPT values and U.C.S.: when U.C.S. is greater than 2 MPa, SPT values tend to be over 50. Assuming U.C.S. (MPa) = N/80 (Terzaghi and Peck, 1948), and that treated soil strengths are 0.3 to 0.4 MPa in such soft clays, N-values of 24 to 32 are to be sought. Using a rough estimate of strength derived from P-waves, the average N-value at each site was calculated as 32, 24, and 39, respectively. This compares with average measured in situ SPT values of 29, 31, and 39, respectively – a reasonable agreement. "Further examination is needed."

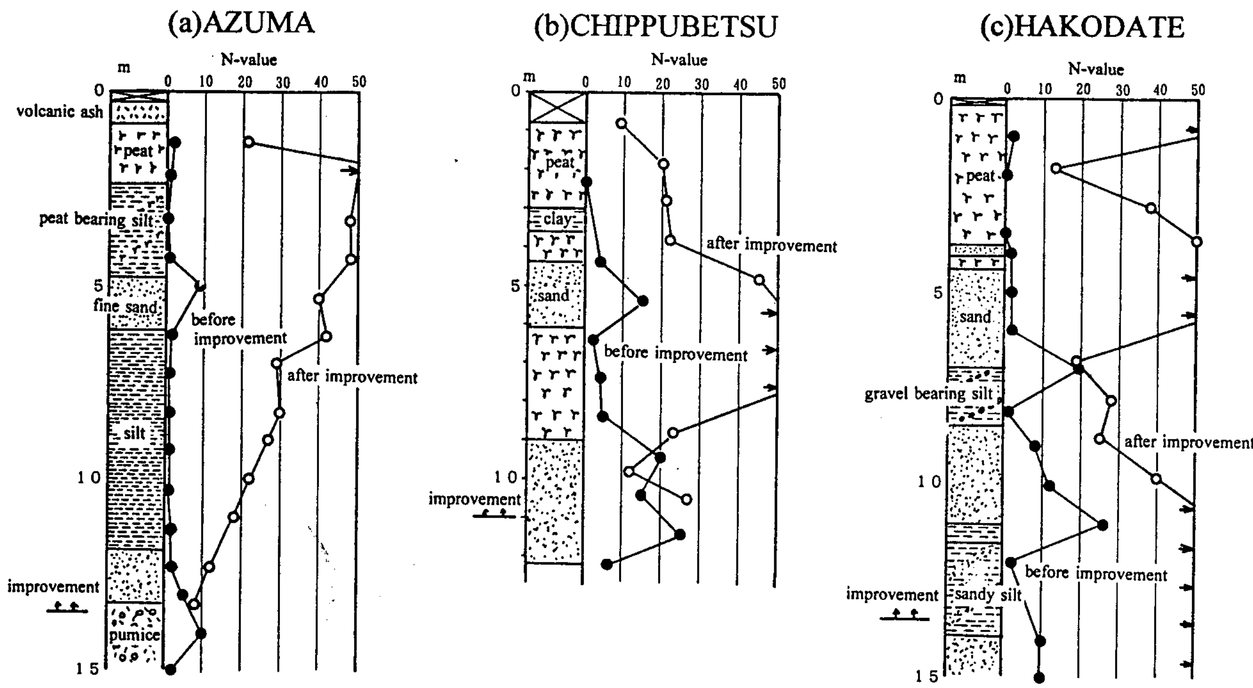


Figure 60. Soil type columns from the sites studied (Nishikawa et al., 1996).

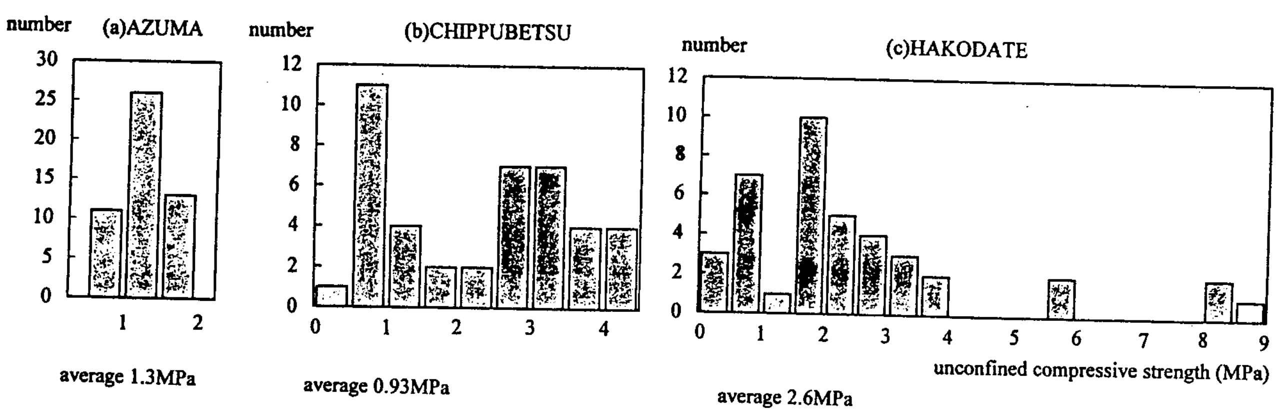


Figure 61. Histograms of the unconfined compressive strength of specimens cored from the improved columns (Nishikawa et al., 1996).

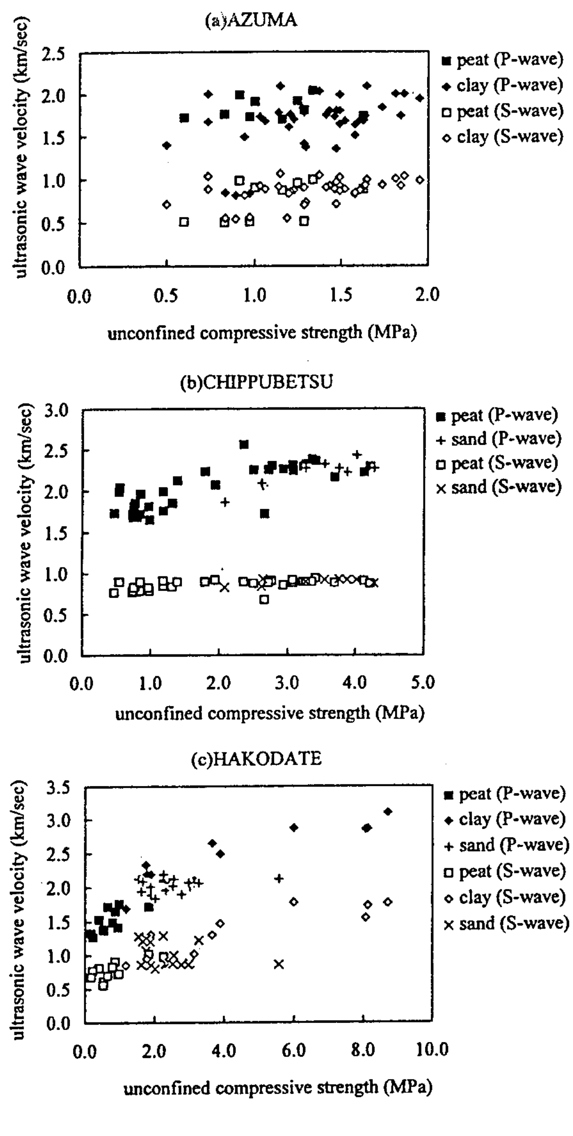


Figure 62. Relationships between the unconfined compressive strength of specimens cored from the improved columns and their P-wave and S-wave velocities (Nishikawa et al., 1996).

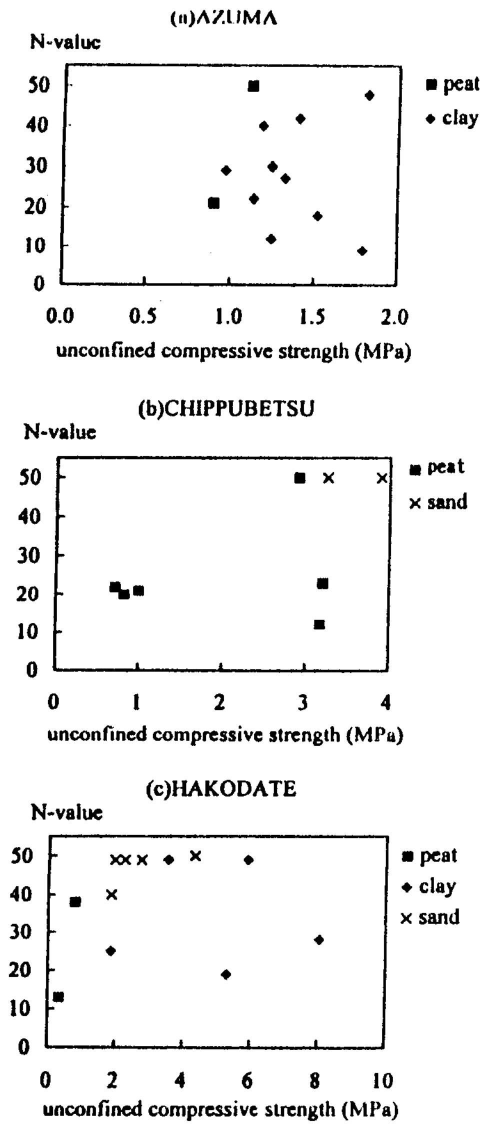


Figure 63. Relationships between the unconfined compressive strength of specimens cored from the improved columns and their in situ N-values (Nishikawa et al., 1996).

Tamura et al. (1996) discussed the use of the low strain sonic integrity test (Figure 64), as used for concrete piles, to evaluate the continuity of the DMM column, and borehole sonar to evaluate shape (Figures 65 and 66). They concluded that, although the studies were at an early stage, the shear modulus of the column could be calculated using both the elastic wave velocity and the borehole scanner.

Barker et al. (1996) described Colmix (Method 4) work in a landfill in Scotland. Preliminary attempts to measure unconfined compressive strengths on samples removed from the mixing augers (by re-drilling), and in situ samples (by "push tubes") gave results that did not appear to be representative. It was therefore decided to carry out in situ mechanical tests using an ultra light dynamic penetrometer: the "Panda" is portable, operated by one person, and requires no external power supply. The energy is delivered by hand-held hammer, and the speed of impact is recorded. The energy imparted is thus calculated, and the point resistance is displayed for each blow. The data are recorded automatically in a "memo block" that can be downloaded via a PC on site to provide a hard copy. The correlation between cone resistance (q_d) and U.C.S. (c_u) is

$$q_d = 10 \text{ to } 15 \text{ times } c_u$$

In the case of a purely cohesive material, U.C.S. is twice c_u, so that

$$q_d = 5 \text{ to } 7 \text{ times } c_u$$

A cone resistance of 1 MPa can therefore be taken as equivalent to a U.C.S. of about 200 kPa. A total of 18 columns were tested, and 3 tests also were taken in an untreated area. Data are provided in Table 9.

Finally, it should be confirmed that the test should match the purpose of the DMM treatment. Where DMM is used to create hydraulic cut-offs, borehole permeability testing to evaluate the material and pump tests to demonstrate the effectiveness of the solution may be conducted. Plate load tests can be conducted – albeit at considerable expense – to demonstrate bearing capacity, while large-scale testing of entire elements can be carried out to measure shear

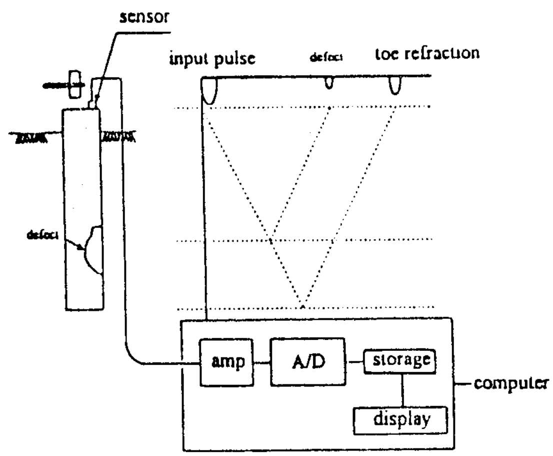


Figure 64. Principle of sonic integrity test (Tamura et al., 1996).

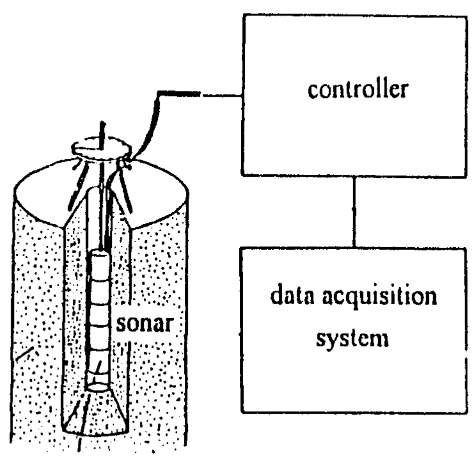


Figure 65. Block diagrams of bore hole sonar (Tamura et al., 1996).

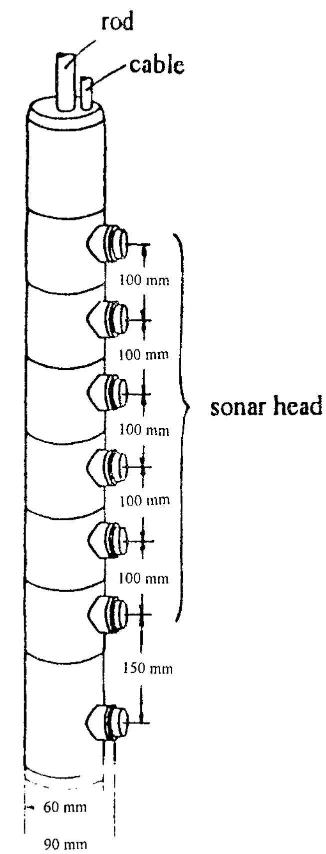


Figure 66. Finished equipment (expected) (Tamura et al., 1996).

Table 9. In situ dynamic penetrometer tests (Barker et al., 1996)

Column Location	Slurry Type	Age at test (d)	Length tested (m)	Cumulation (cm) where qd < 1MPa	% where qd > 1 MPa
1618	48	75	3,25	0	100 %
1619	47	86	5	19	96,5 %
1630	48	78	4,25	0	100 %
1631	48	91	3	0	100 %
1635	48	91	3,5	0	100 %
1636	48	90	3,5	20	94,5 %
1671	48	74	2,5	0	100 %
1672	47	84	3,25	0	100 %
1602	48	76	3	0	100 %
1604	48	76	5	2	99,6 %
1645	48	91	5	4,5	99,1%
1647	48	91	3	0	100 %
1686	X	WASTE	5	84	
1628	X	WASTE	5	4,5	
1870	X	WASTE	5	114	
1985	47	7	3	0	100 %
1927	47	7	3	0	100 %
1869	47	7	5	18,5	96,3 %
2399	47	28	5	0	100 %
2300	47	28	3,25	0	100 %
2242	47	28	3,5	0	100 %

strength and lateral load/deflection performance. The latter was proposed as a test at the Fort Point Channel project of the Boston Central Artery in 1997 (Figures 67 and 68).

3.3.6 Testing Issues Specific to Lime Cement Columns

A considerable volume of recent information has been published in English by practitioners in the Nordic countries relating to the specific problems of Lime Cement Columns. The issue of QA/QC and verification has been a priority of the national research programs in these countries. A review of older and general data (Section 3.3.6.1) is followed by synopses of key papers presented at the October 1999 conference in Stockholm "Dry Mix Methods for Deep Soil Stabilization."

3.3.6.1 General Data

The ASCE Soil Improvement and Geosynthetics Committee Report (1997) addressed the issue of "current QA/QC" with respect to the Lime Cement Column Method. Their summary provides a useful overview and provides logical commentary.

"Lime and cement reactivity testing with the site soils are necessary to determine if the soils are conducive to stabilization. Multiple replications of laboratory prepared samples using different quantities of stabilizers should be fabricated to anticipated unit weights and need to be cured in conditions expected in the field including temperature control. Unconfined testing at various time intervals (over several moths preferably) after molding should be performed well in advance of design to monitor strength gain with time. The shear strength of the stabilized soil has to be at least three to five times the initial shear strength determined by unconfined compression tests before the lime or lime-cement column method becomes economical (Broms, 1991).

Items to be checked during construction include freshness of stabilizers, particle sizes, location of columns, eccentricity, length of columns, quantity of lime utilized, and withdrawal/mixing rate.

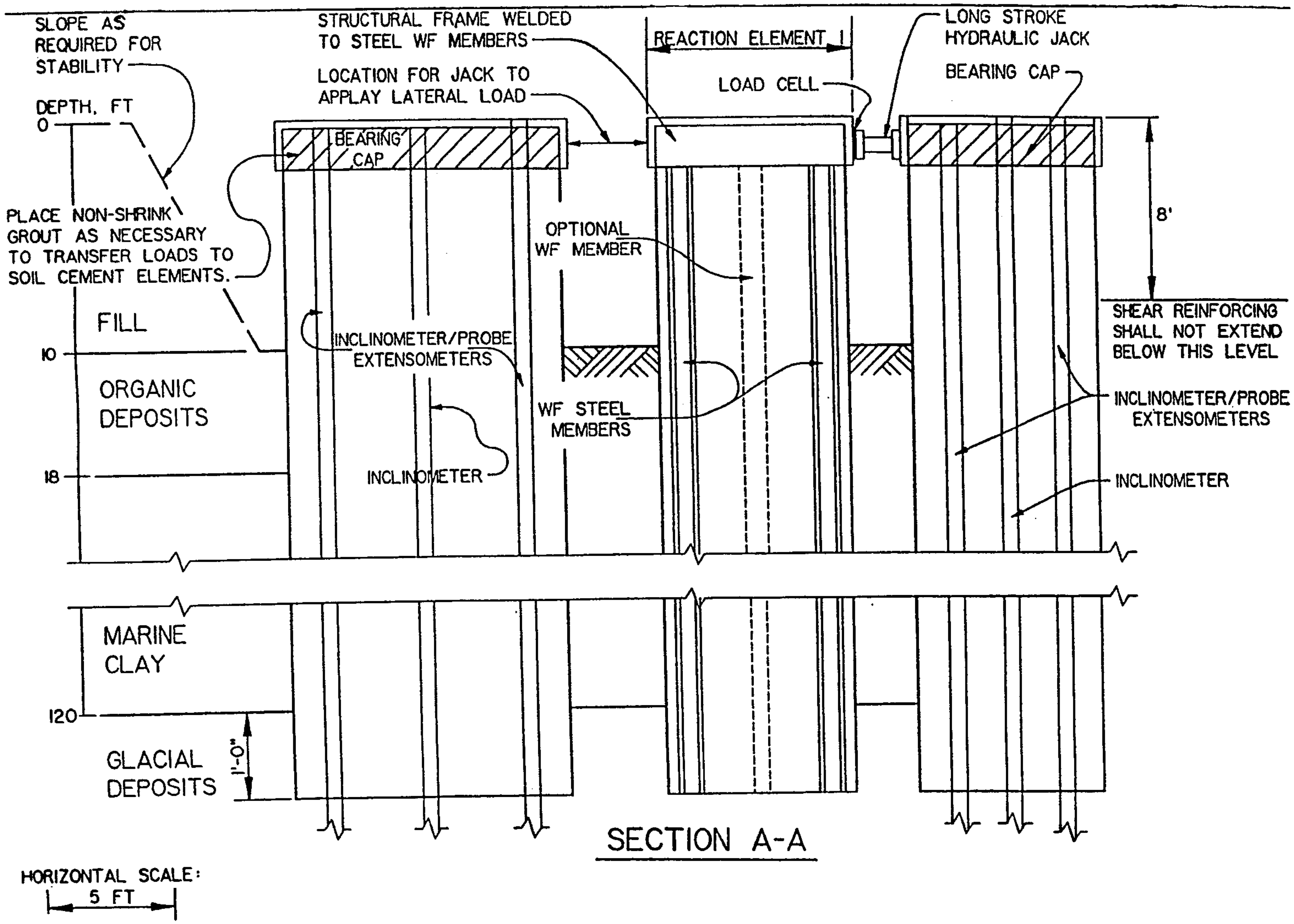


Figure 67. Elevation view of instrumentation layout for Fort Point Channel Work (Boston, MA) (Druss, 1998).

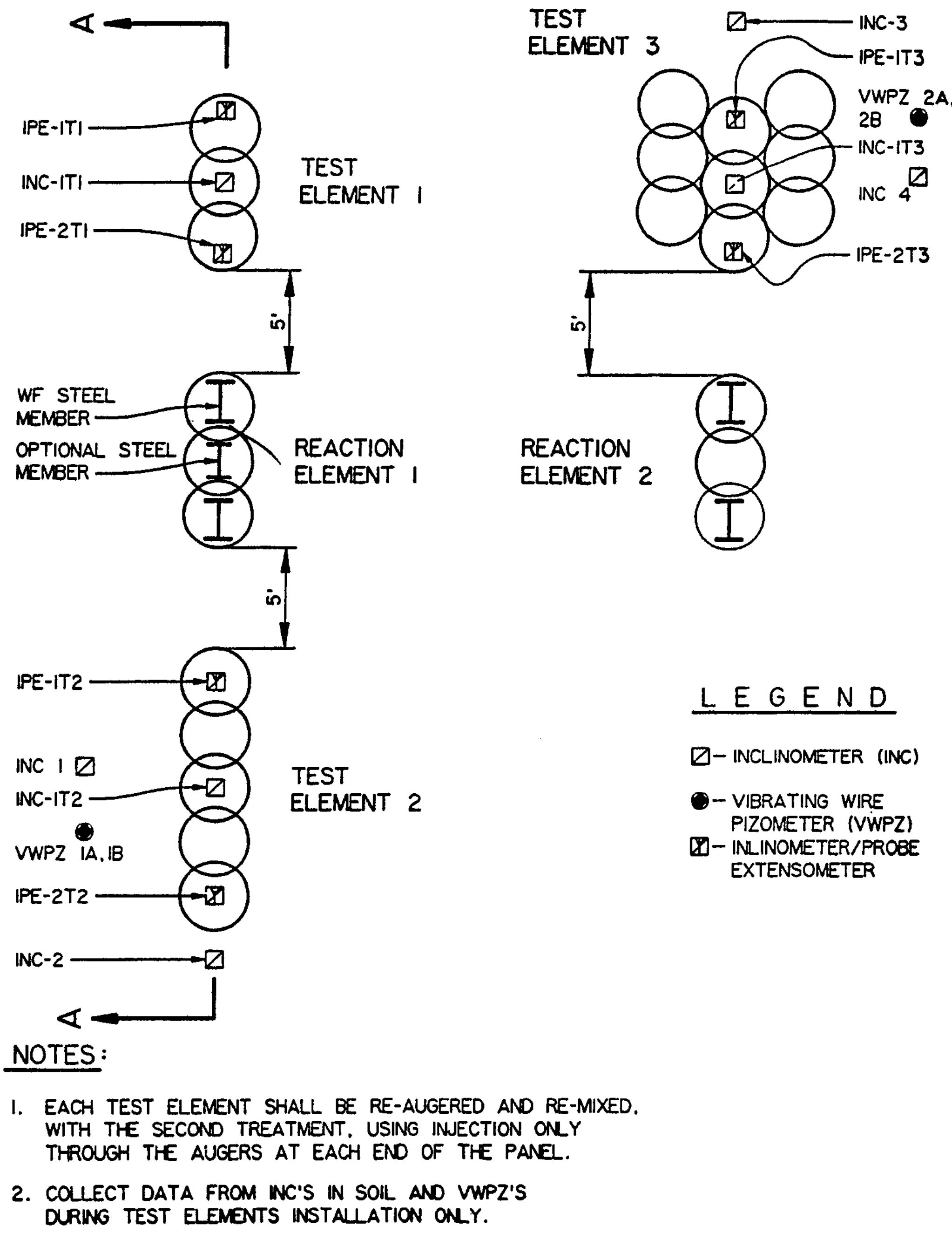


Figure 68. Plan view of instrumentation layout for Fort Point Channel Work (Boston, MA) (Druss, 1998).

On large jobs (> 1000 columns) or with initial unfamiliarity with the method, it is advisable to check the bearing capacity and the creep limit with full scale test columns at the project site.

In addition to load tests, verification strength testing of the in situ mixing operation should be monitored. A specialized column penetrometer has been developed for this purpose that overcomes in situ testing problems encountered with conventional testing equipment and scale of test sample restrictions. That column penetrometer is shown in Figure 69. The column penetrometer can more accurately determine the undrained shear strength of the stabilized column material and should be performed on 1 to 3 percent of the columns on each job. Pressuremeter tests can also be performed on actual columns for the same purposes as the column penetrometer. Caution should be taken in interpretation of pressuremeter results due to the weakened central section of each column. That effect is produced by the relatively uncompacted mix in the hole that is left upon withdrawal of the kelly.

Rathmeyer (1996) reviewed Lime Cement Column practice and noted that variations in shear strength within the stabilized columns (a measure of homogeneity) are measured with special wing penetrometers (Figures 70 through 72). The traditional push-down version is limited to a maximum shear strength of 150 kPa and 8 m depth; the inverted version has limits of 600 kPa strength and 15 m depth. A problem is with the penetrometers actually following the centerline of the column. For higher strength columns, wing penetrometer tests can be done in pre-drilled 55- to 65-mm-diameter holes. He concluded that screw compressometer tests were time consuming and expensive; column sounding, vane tests, and CPT were unreliable or misleading. Methods applied for integrity testing of concrete piles do not work. Therefore "the only reliable test method today is total sampling, managed by lifting up on the entire column."

Markteknik (1999) provided more recent data, from a contractor's viewpoint. It refers to the traditional "pressure sounding probe" as "KPS" (Figure 73), which was used to depths of 4 m. However, there were concerns about the probe remaining wholly within the column, even with pre-drilling, and so to improve quality and reduce costs, it developed the reverse probe or inverted penetrometer, known as "FOPS." Markteknik claims this patented method is now the most

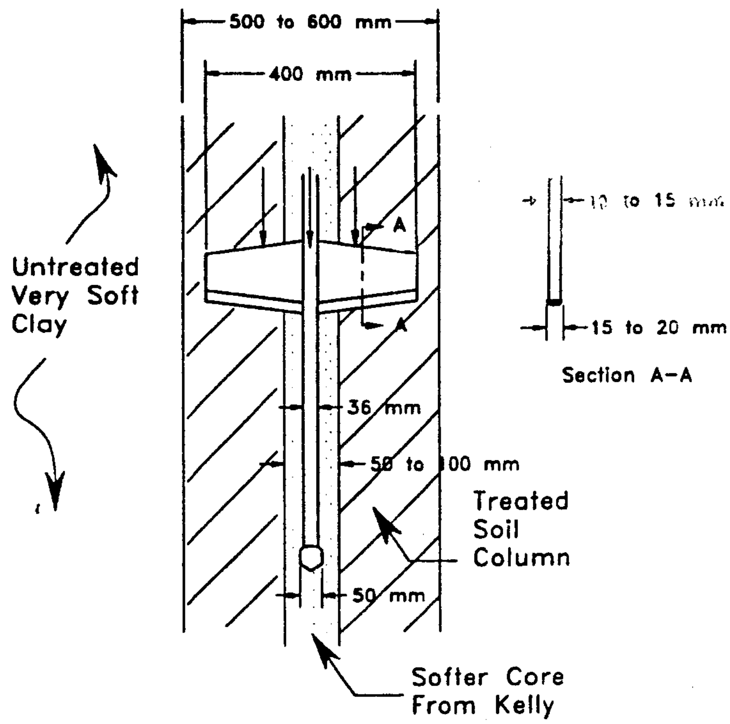


Figure 69. Lime column penetrometer (after Holm et al., 1981).

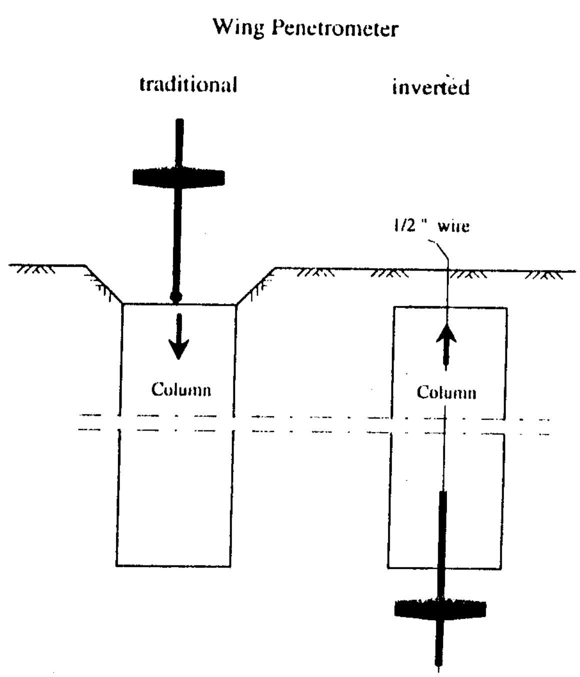


Figure 70. Principles of application of wing penetrometers (Rathmeyer, 1996).

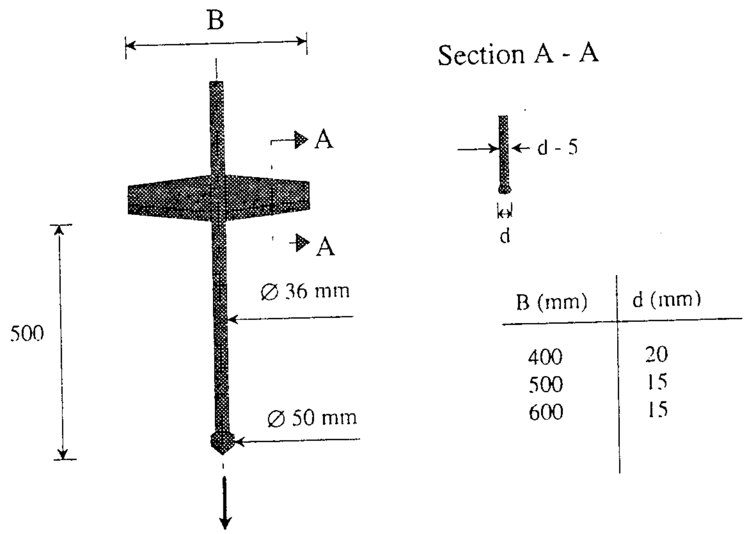


Figure 71. Details of the traditional wing penetrometer for in situ strength testing of stabilized columns (Rathmeyer, 1996).

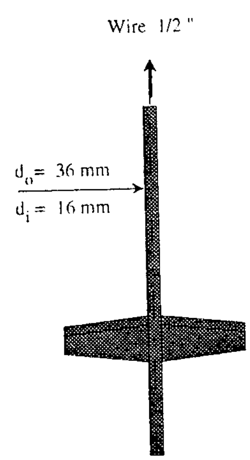


Figure 72. Details of the inverted wing penetrometer for in situ strength testing of stabilized columns (Rathmeyer, 1996).

Inspection of the columns was previously performed using pressure sounding. For columns with a depth of >4 m, however, it is highly uncertain that the whole probe remains within the column even if pre-drilling is performed. In order to ensure the best possible result and to reduce costs, we have further developed the technique with reversed probing to pre-installed, reversed column probing, FOPS (patented). This technique is today the most common method used for testing the continuity and relative strength of the columns.

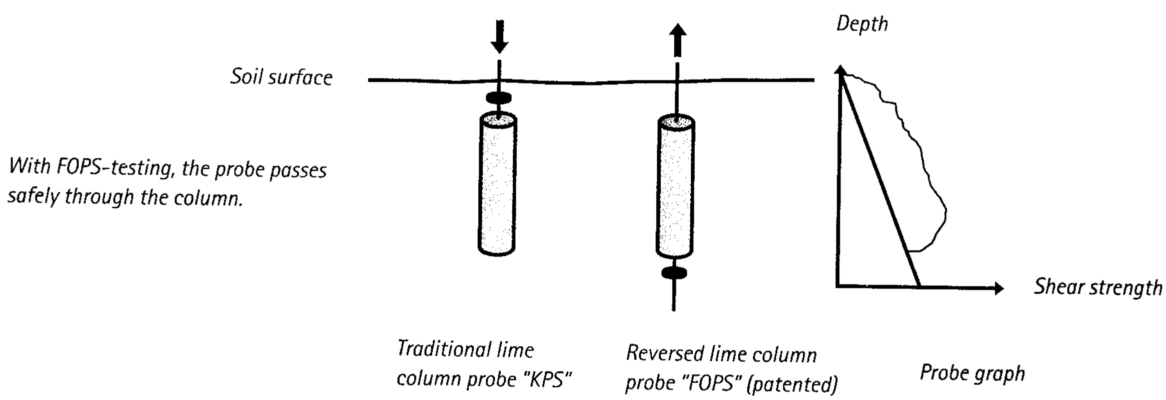


Figure 73. Inspection and testing of columns using the FOPS probe (Markteknik, 1999).

common method for testing the continuity and relative strength of Lime Cement Columns, and to ensure from the onset, that the design requirements will be satisfied.

Typical data presented by Esrig (1999) on the use of the inverted penetrometer are shown in Figures 74 through 77.

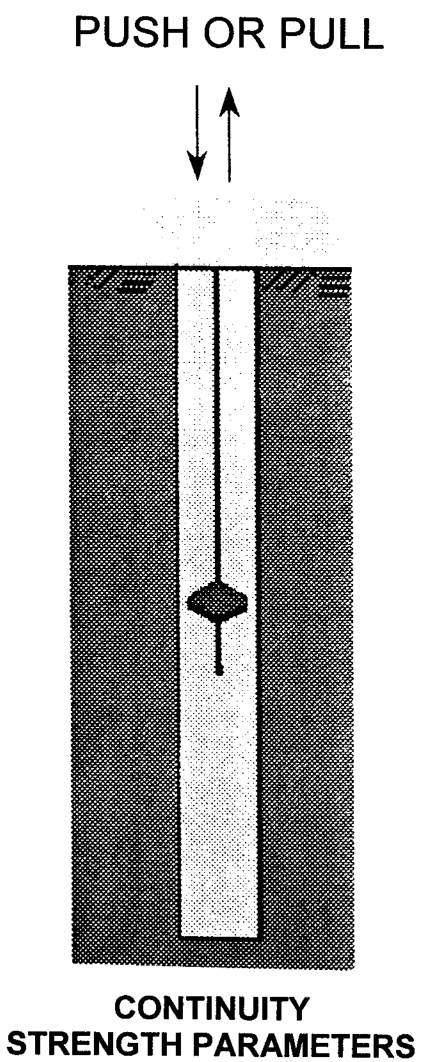
3.3.6.2 Data from Stockholm Conference, October 1999

Norwegian experience was summarized by two groups of authors – Watn et al. (1999) and Braaten et al. (1999).

Watn et al. note the growing use of Lime Cement Columns particularly for slope stabilization, but that failures have occurred. A "public R&D project" – "Grunnforster Kning med Kalksementpeler" by Norwegian Railways and the contractor Veidekke – has contributed to better understanding. This project was completed in 1998 and involved "several Norwegian and Swedish research institutes and contractors." Particular attention was paid to the relationship between shear strength found in the laboratory and effective shear strength from field testing.

<u>Field Testing</u>

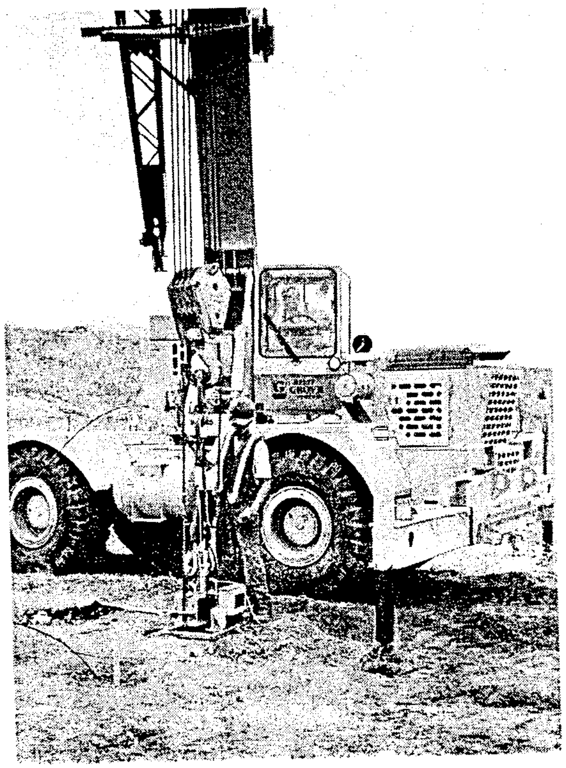
1. FOPS: the resistance to withdrawal is used to correlate with the undrained shear strength. Although a continuous profile is provided, the columns to be tested have to be predetermined and so may not necessarily be representative. Shear strengths are based on empirical Swedish relationships in lower strength clays and so FOPS relationships developed in Norwegian conditions may be different.
2. CPTU (Cone Penetrometer): during testing, tip resistance, side friction, and pore pressure are recorded, and undrained shear strength is determined. Thus, significant weak zones are easily detected. However, the cone can deviate outside the column. A comparison with FOPS data is provided in Figure 78: in general, FOPS values of shear strength average 50 to 100% higher than those determined from CPTU.



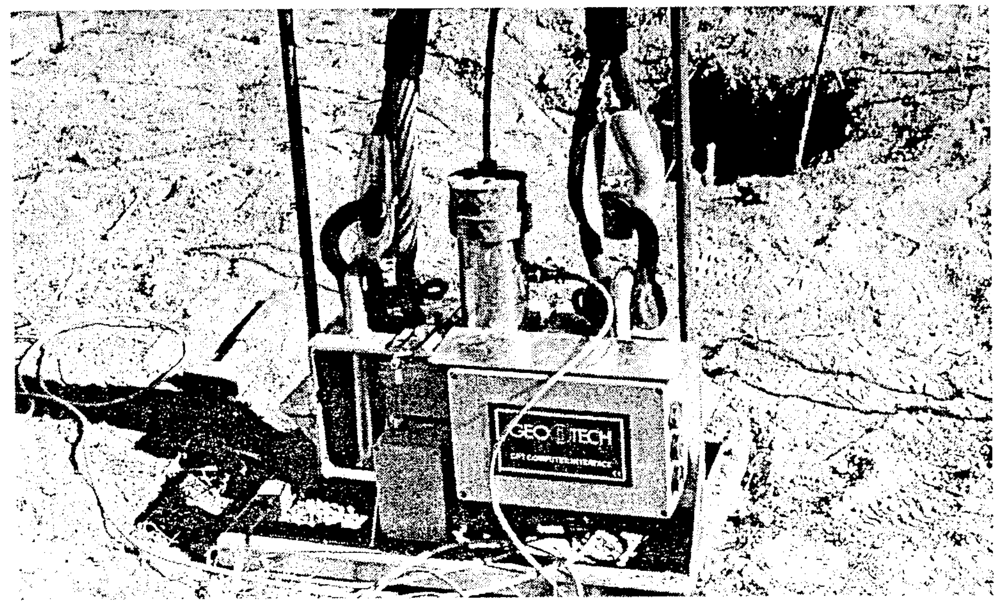
PROBE SIZE 400-600 mm
THICKNESS 15-20 mm

stabilator

Figure 74. Lime cement column probe testing (Esrig, 1999).



PULL OUT TEST



COMPUTER MONITORED
LOAD CELL

Figure 75. Lime cement column probe testing equipment (Esrig, 1999).

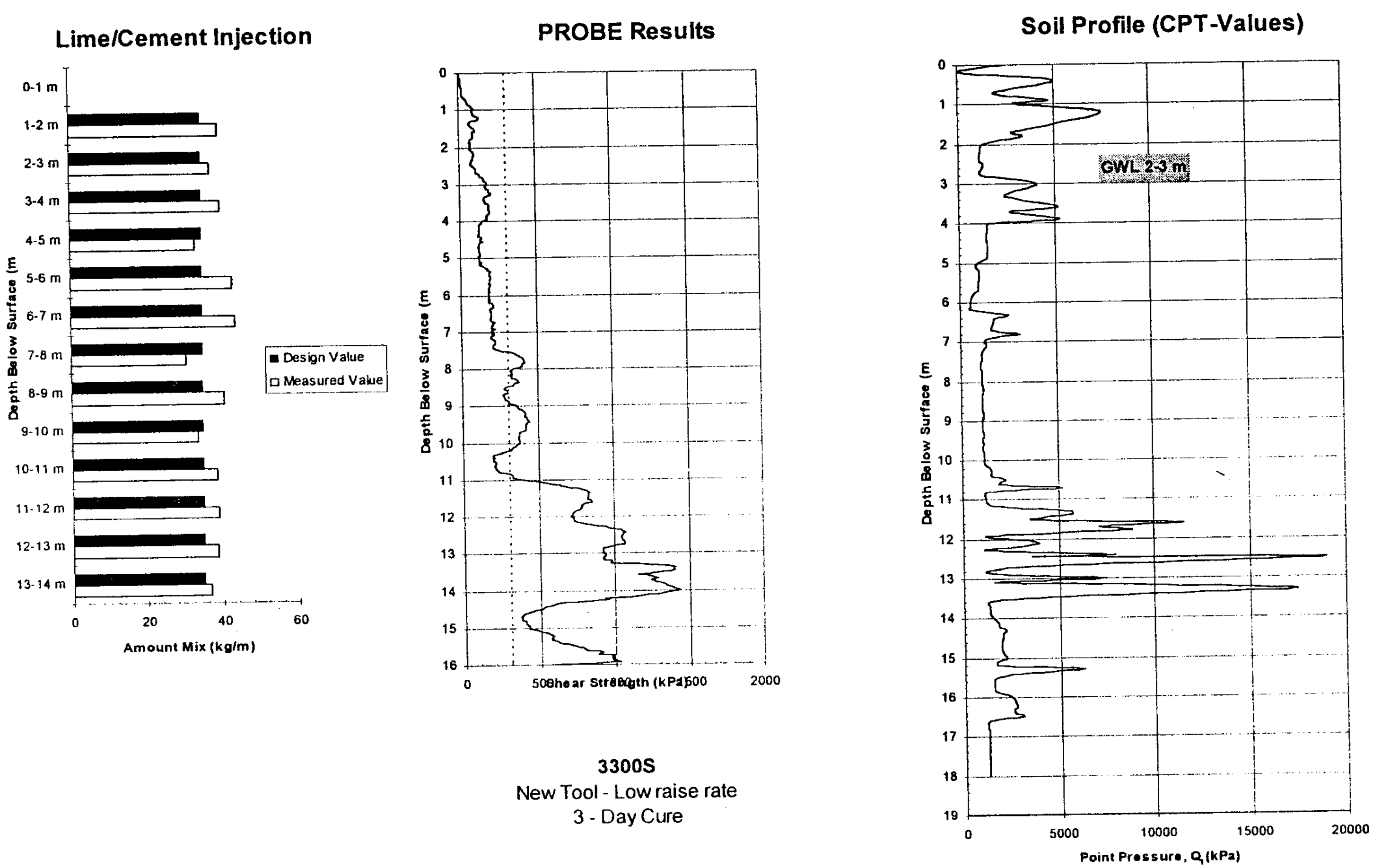


Figure 76. Typical pull out test results for Column A2, 3-day cure (Esrig, 1999).

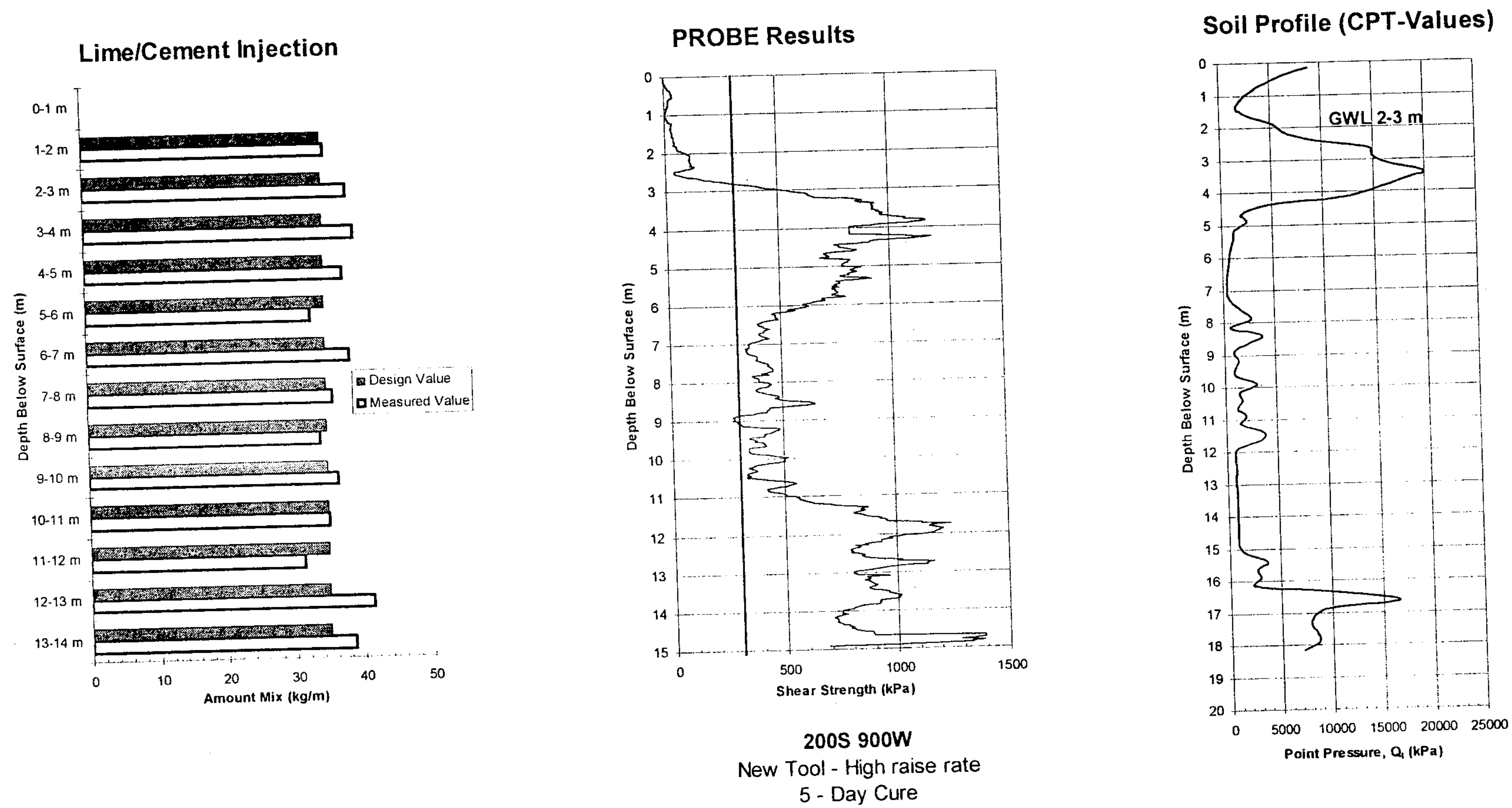


Figure 77. Typical pull out test results for Column C3, 5-day cure (Esrig, 1999).

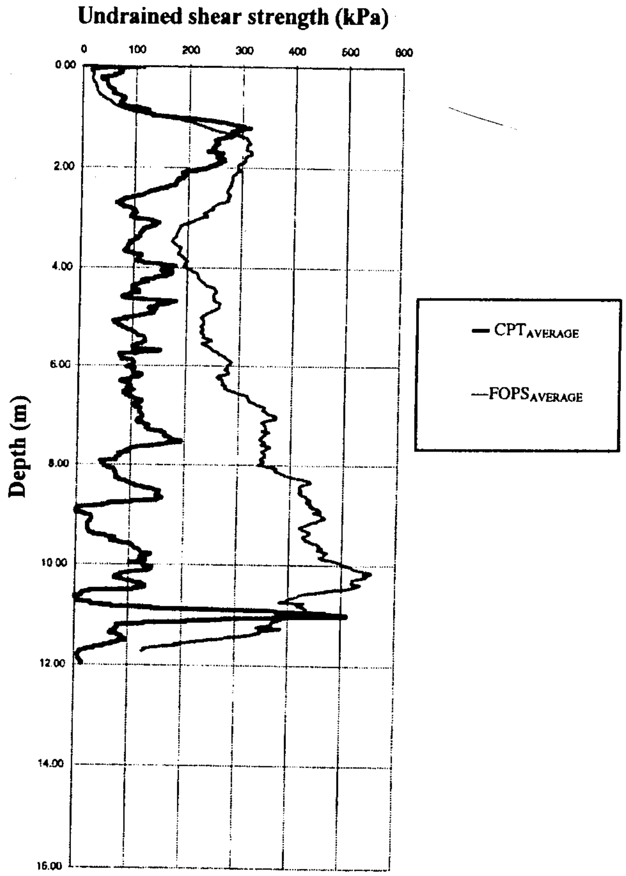


Figure 78. Average undrained shear strength (s_u) determined from FOPS and CPTU ($N_k = 15$) (Watn et al., 1999).

3. ODEX Core Sampling: a 102-mm cased hole provides a PVC-lined core. Clearly any column (not predetermined) can be cored; the major disadvantages are potential deviation and water flush damage to the column.
4. Sampling of the whole pile: a specially designed 12-m-long steel pipe, i.d. 700 mm, has been used to sample 600-mm-diameter columns. The pipe is in two halves and has a "core catcher." It is first driven into the soil concentric with the column. It is then pulled out with a crane and the extracted column exposed for observation, pocket penetrometer and pocket vane testing, and further laboratory testing. Although very useful, it is costly and the sampling method may itself produce undesirable compression and tensile stresses on the column prior to testing.

Laboratory Testing

1. U.C.S.: provides data on load/deformation, and undrained shear strength and strain at failure. Its simplicity is attractive but the accuracy of determining undrained shear strength is limited. Data from U.C.S. (UCT) on cored samples and samples from the whole column are compared with undrained shear data (s_u) from CPTU testing in Figure 79: in some cases a discrepancy of 20 times is evident (at 7 m), judged to reflect the differences in test methods.
2. CU: consolidated undrained triaxial tests provide undrained shear strength and the effective strength parameters (c and Φ), This is a "realistic" test, being the only one providing effective strength parameters. Data from two triaxial tests performed on cored samples gave undrained shear strength values in excess of s_u values from CPT but coincident with the highest values from U.C.S. tests.
3. Other tests: these include visual inspection and description, water content, density, and lime cement content.

Braaten et al. (1999) noted that ground stabilization with lime columns was first used by the Norwegian Public Roads Administration (NDRA) in 1977 at the Rørrik Ferry Terminal, and described experiences on three large recent projects. Selection of design parameters has traditionally been based on shear strength evaluation of remolded laboratory samples, via U.C.S. and (some cases) by CU. Comparison was made between laboratory tests (U.C.S.) and FOPS on test columns (38 days), as shown in Figure 80 at 28 days. FOPS is used routinely in Norwegian practice, but given its "predetermined" nature (and the fact that the contractor often conducts the test himself) "it is not considered as an acceptable objective test method by the owner of the road." The NPRA has therefore tried to develop new methods, including a modified vane test (40/80 mm) and a tube sampler (54 mm). Two projects have been tested to date.

Modified Vane: after 7 days the shear strength was high enough to cause damage to the equipment. At 8 days a shear strength of 0.15 to 0.25 MPa was measured (Figure 81). There were also issues with deviation outside the column below 6 m. However, the development of testing procedures for the modified vane is being continued.

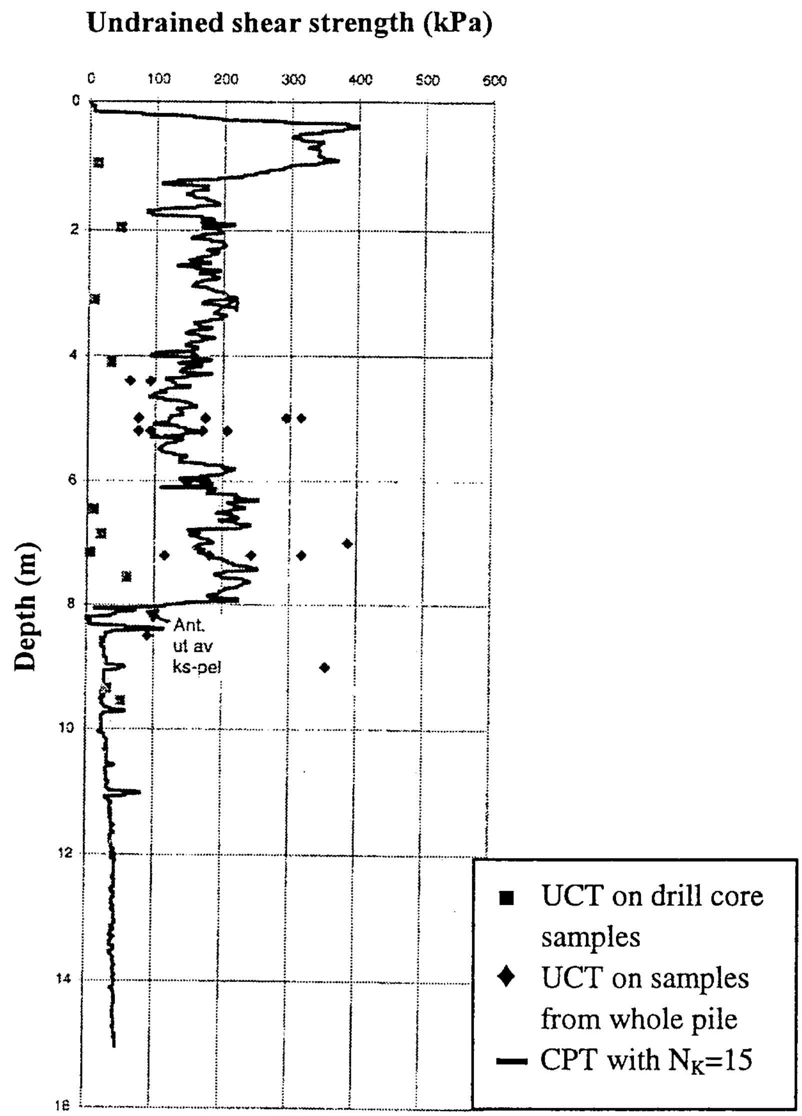


Figure 79. Undrained shear strength profile from pile 2D after 33 to 40 days of hardening (Watn et al., 1999).

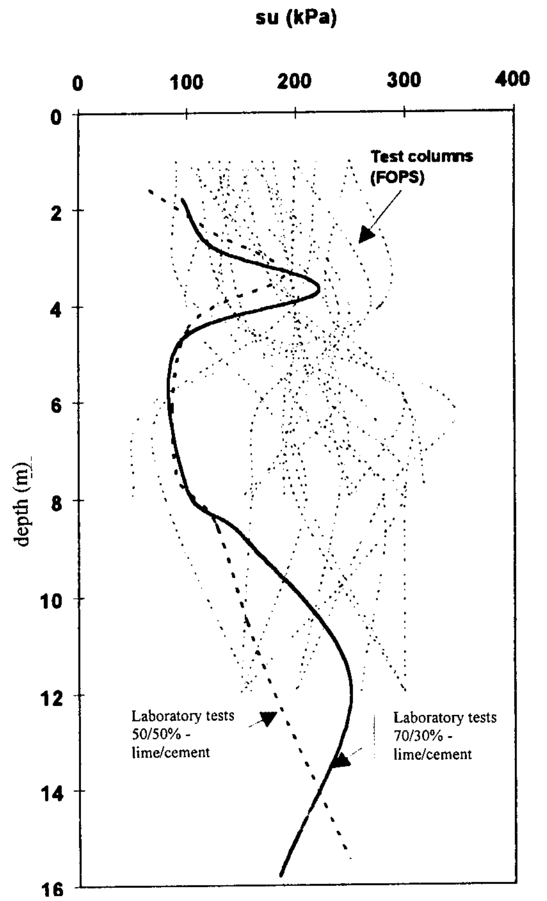


Figure 80. Summary of results from laboratory and field tests (Braaten et al., 1999).

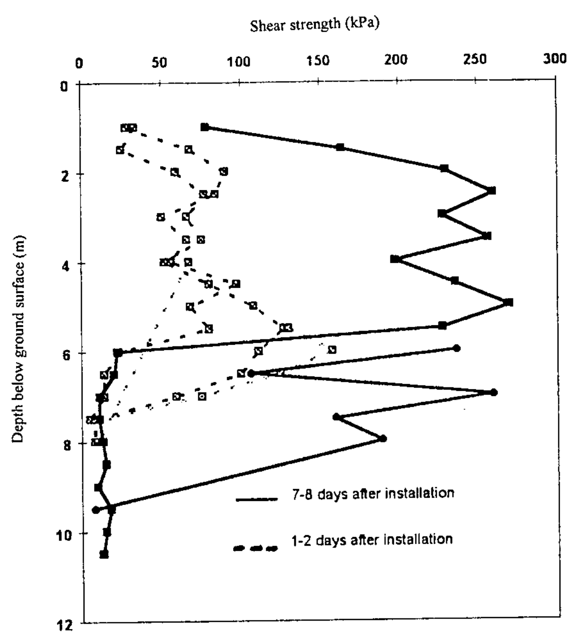


Figure 81. Shear strength measured with the modified vane (Braaten et al., 1999).

Tube Sampler: samples were taken after 1 to 2 days, easily to 9 m, to provide samples for laboratory testing. U.C.S. and triaxial tests were conducted (Figure 82) with good correlation. This figure also indicates a more rapid development of shear strength in vane data than sampled data, while the averaged FOPS data "harmonize quite well" with the vane test and the laboratory samples in general. Similar tests on the second site compared laboratory and "sampled" specimens at 28 days. The latter had highly variable readings, were not homogeneous, and were in general lower than laboratory mixed samples (Figure 83).

Swedish and Finnish practice has been summarized by four main groups.

Edstam and Carlsten (1999) developed a "suitable reference method" for preparing laboratory mixed soil, to be used as standard within all projects coordinated by the Swedish Deep Stabilization Research Centre. Clay and binder are mixed in a closed cylinder by a specially designed mixing tool. Sample tubes are then "punched" into the mixed soil to provide samples of appropriate shape. Extensive testing shows that values of s_u (via U.C.S.) have now a deviation of ±10% from the average: 2 to 3 times better than "older methods." Undrained shear strengths are similar between methods, but occasionally lower stiffness is generated by the newer method.

Holm et al. (1999) confirmed that penetration testing is typically used to verify design values in production columns but that total column extraction offers a "valuable compliment" including samples for field and laboratory testing. For penetration testing, CPT is not favored because of the small cross sectional area of the tool, while conventional (Figures 84 and 85), and reverse (FOPS) (Figures 86 and 87) penetration testing are favored, provided the columns are not "hard." However, none of these methods can detect weak zones deep in a very hard column, or tell anything about the binder distribution or column diameter – this is where extraction can offer many advantages. Column hardness is not an issue while strength, diameter, homogeneity, and permeability can be tested all along and across each column. Expense, however, usually limits such testing to research programs. The authors also describe the 10-m-long, 900-mm-diameter split tube with "bottom shutter." It is vibrodriven to depth, and needs about 20 tonnes of pull-out force. Visual inspection at each of two cuts (one at half radius, the other in the middle) allows relative hardness to be gauged (Figure 88), together with diameter and homogeneity.

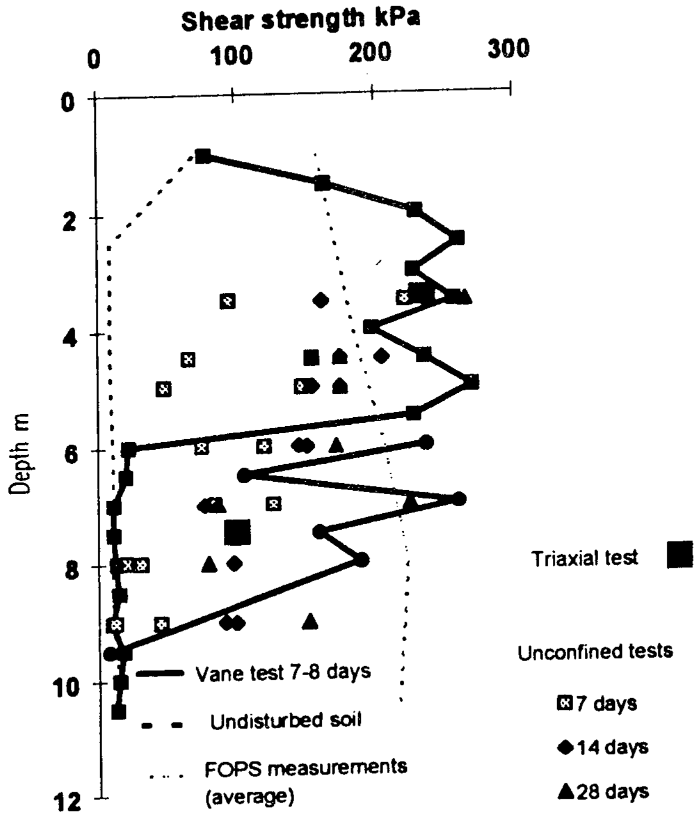


Figure 82. Comparison of results from Timenes (Braaten et al., 1999).

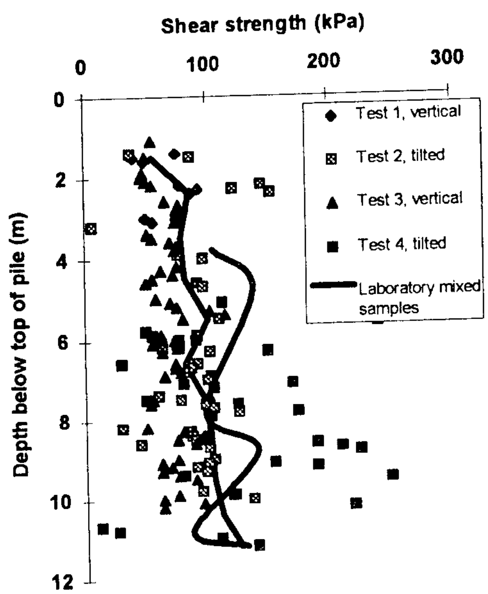


Figure 83. Comparison of results from Moubekken site (Braaten et al., 1999).

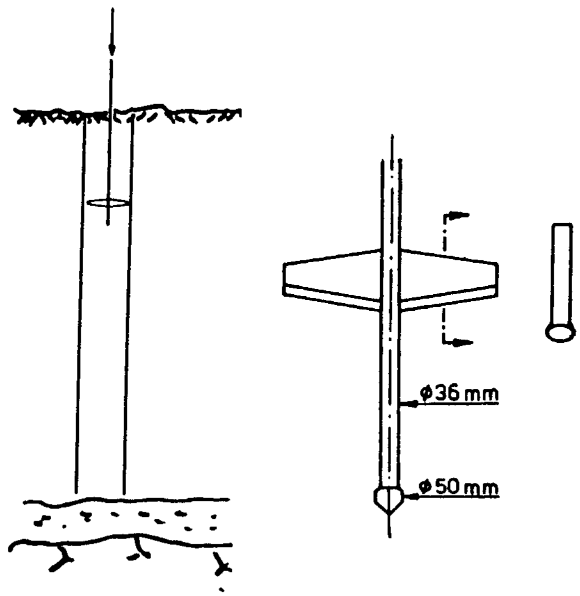


Figure 84. The conventional column penetration test (Holm et al., 1999).

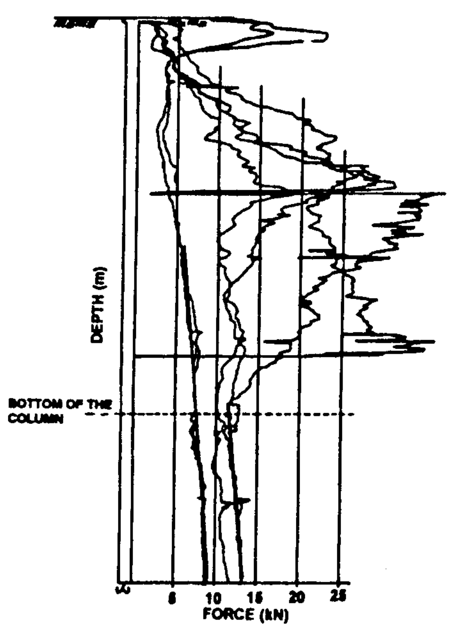


Figure 85. Typical result from conventional column penetration test (Holm et al., 1999).

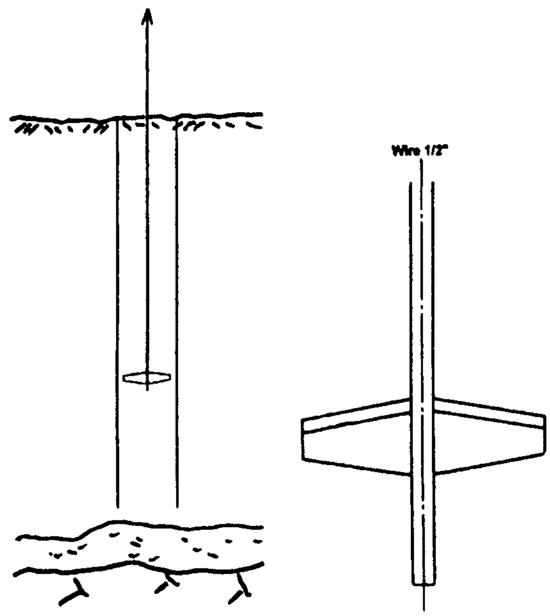


Figure 86. The reverse column penetration test (Holm et al., 1999).

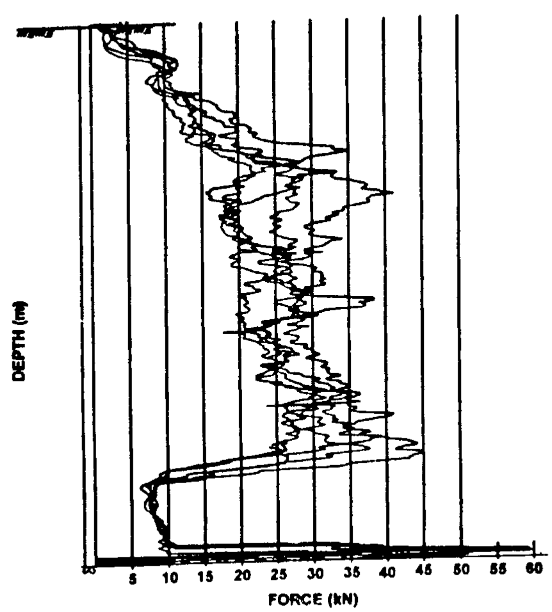


Figure 87. Typical results from reverse column penetration test (Holm et al., 1999).

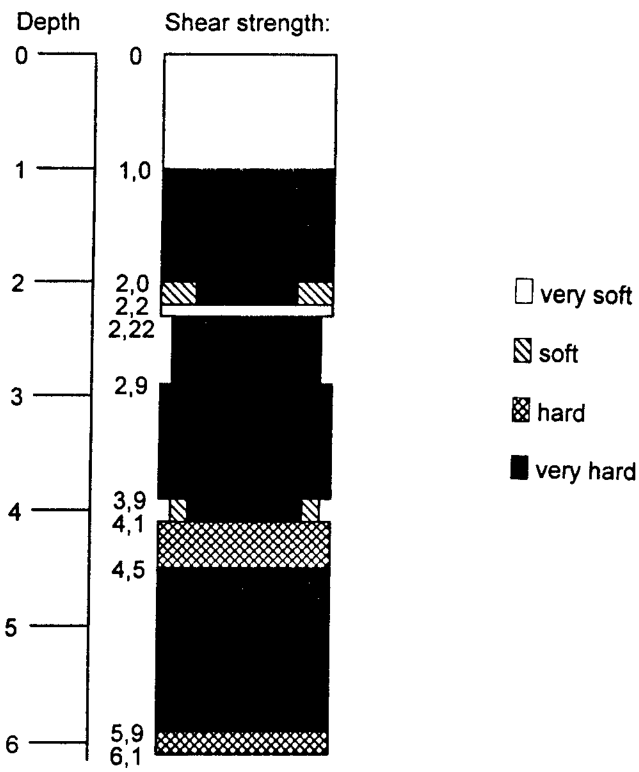


Figure 88. Example of results from visual inspection/classification (Holm et al., 1999).

Shear strength is measured via penetrometer and small vane tester. Laboratory testing on 500-mm-diameter tube samples provides data on density, moisture content, amount of binder, and shear strength (from U.C.S.). Hard, brittle columns tend to provide disturbed samples of below expected strength. Triaxial tests may also be conducted.

Axelsson and Rehnman (1999) discussed the "strategy of the checking" before illustrating the details with respect to an ongoing project. They are researchers at the Royal Institute of Technology in Stockholm, where a research project is ongoing to develop and evaluate methods for quality control. They make the following points regarding the verification strategy:

- Number of columns tested is 0.5 to 2% of total (preferably selected at random, and also to evaluate geological differences).
- Testing should be as soon as possible after installation (to catch systematic errors early).
- The main purpose of the treatment should dictate the testing program and methods, although length and diameter are always important to check.

Regarding field test methods, their review can be summarized as follows (Figure 89):

METHOD	PRODUCT/NATURE	RESTRICTIONS
Conventional Penetrometer Test (KPS)	Wings about 100 mm less than column diameter; pushed at 20 mm/s. Provides mean shear strength. Pre-drilling (44 mm) reduces deviation.	Mean shear strength < 150 to 300 kPa and length < 8 m. Assumes $\Phi_u = 0$
Reverse Column Penetration Test (FOPS)	Pre-installed and pulled at 20 mm/s after 2 to 4 weeks.	Can test to 600 kPa and 20 m. Test is simple but location is predetermined. Bottom 1 m not tested.
Column Vane Test	Developed from field vane. Maximum torque recorded at different levels (often at ½ radius). Shear strength calculation from vane factor times maximum torque.	Disturbance often produces low results. Test data <u>not</u> continuous.
Borehole Sonar	S- and P-waves generated radially and received. Velocities calculated and E and s_u calculated empirically. 75 mm augered hole used.	Prototype but results promising. Light equipment, fast results.
KTH Penetration Test	New type with thin steel plate wings. Adhesion corresponds to s_u.	Newer development.

For the field tests, 54 columns were installed of which 15 were extracted. The whole range of penetrometer and materials testing was conducted. The authors concluded:

- Conventional penetration testing (KPS and KTH) can be performed to 8 m even in 800 to 1000 kPa material using 44-mm-diameter rods to reduce bending and skin friction. A very heavy reaction is required.
- Reversed penetration testing experienced no problems.

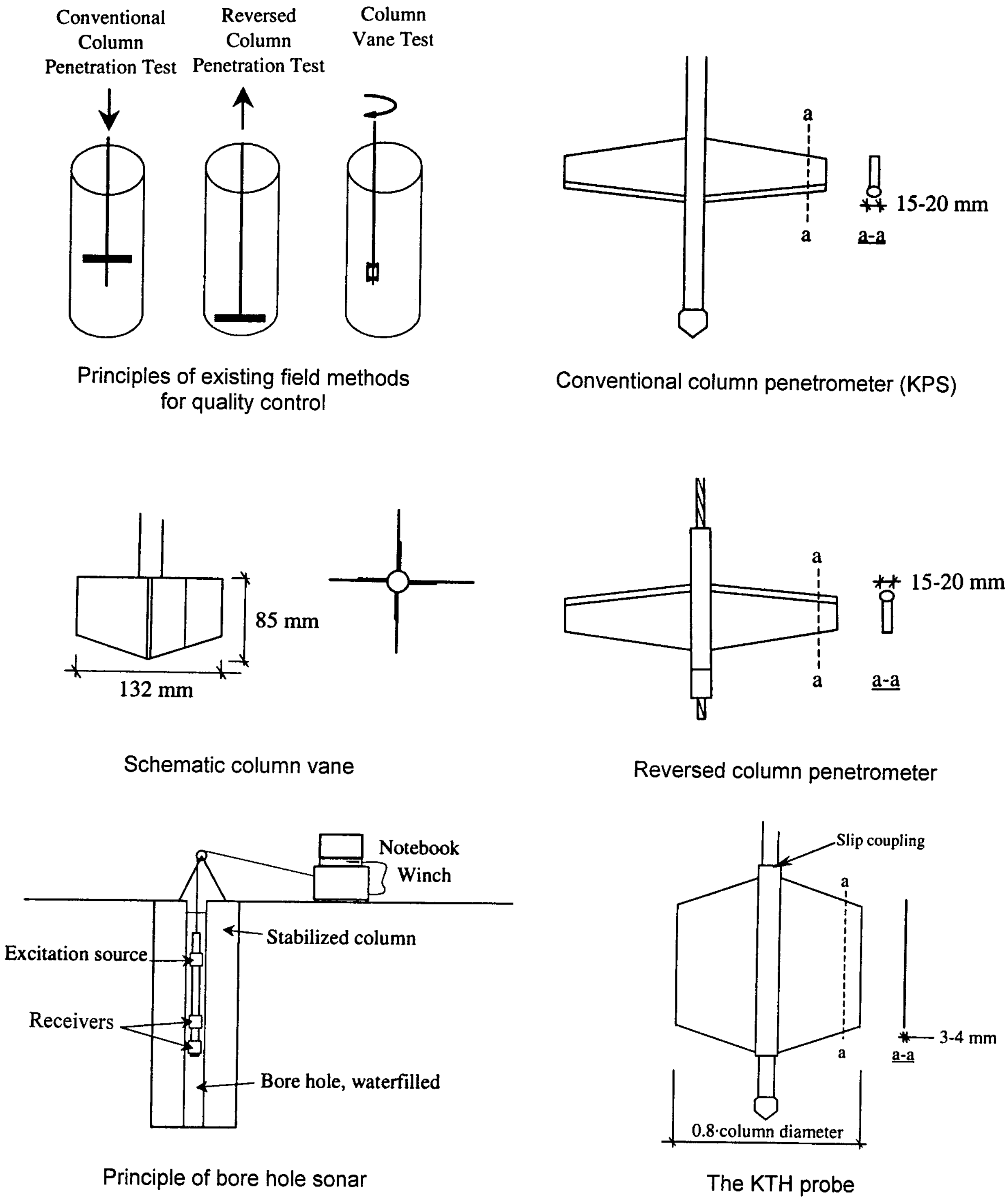


Figure 89. Schematics of field test methods (from Axelsson and Rehnman, 1999).

- Column vane test not satisfactory (the 36-mm rods deviated after 4 to 5 m and the test needed high push forces). Data good for strengths < 150 kPa.
- Sonar was successful.
- U.C.S. tests were similar to penetrometer data especially for s_u < 100 kPa.

Halkola (1999) of the City of Helsinki described experiences in Sweden and Finland. In general he noted that the "extensive non-homogeneity of the columns" makes it impossible to make reliable conclusions on the strength or success of the stabilization based on individual column penetration tests. However, he noted that with the increasing use of cement in addition to lime, the columns became harder (and lose the "weak" central core of 100 to 150 mm diameter). In Finland the decision was therefore made to use CPT and apply statistical methods for interpretation. A pullout resistance test (PORT, assumed equivalent to FOPS) was developed in Sweden for hard, deep columns, tested earlier than later. Undisturbed sampling "became general in the 1990s" but is operationally difficult, while in Japan the "quality control…is based almost entirely on the taking of undisturbed samples whose strength is determined by uniaxial compression tests." Extraction of whole columns is judged expensive and is "not well suited" to columns greater than 10 m long.

A review of "quality inspection" of installed columns is provided in Table 10. Tests are generally conducted at 7 to 28 days in Sweden and at 28 to 56 days in Finland.

Halkola notes that if the total number of successful tests (penetrometer or CPT) is small, or the columns are exceptionally nonhomogeneous, a misleading view of the strength variation can be obtained from the standard deviation. In such cases, it is advisable to use the absolute minimum and maximum values, although the minimum values cannot be used as the design strength, "apart from certain exceptional cases."

He concludes that DMM columns are typically produced quickly and this, plus the natural variability of the soil and the testing methods, will generate a scatter of performance data, even when tested with the recent advances in geophysical methods. He foresees that better

Table 10. Summary of column testing methods (from Halkola, 1999).

METHOD	DETAILS	ADVANTAGES	DISADVANTAGES
1. Column Penetrometer (Conventional)	• Cross sectional areas 100 cm^2. • Developed in Sweden in 1980, modified version in Finland from 1981. • Penetration resistance converted into shear strength by dividing by $N_c = 10$ (Sweden) or 10-15 (Finland) where vane penetrometer helps confirm data. • 3-wing penetrometer experimented with in Finland to prevent deviation out of column. • Minimum five successful readings needed.	• Continuous record. • Quick, easy, cheap.	• Only in soft and semi-hard columns ($s_u < 200$ kPa). • Depth limited to approximately 7 m. • Pre-drilling expensive. • Interpretation complicated by sleeve friction.
2. Pullout Penetrometer (PORT ≡ FOPS?)	• Developed in Sweden in early 1990s.	• Applicable in deeper columns. • Applicable in harder columns.	• Valid up to $s_u < 600$ kPa. • More costly. • Predetermined test location.
3. Column Vane Penetrometer	• Used in Finland since early 1980s. • Tests at 0.5- to 1.0-m intervals. • Used mainly to help interpretation of "conventional" penetrometer test data. • Use decreasing with rise in CPT during 1990s.		• Best for $s_u < 200$ kPa. • Relatively expensive. • Upper "crust" or fill makes it problematical.

Table 10. Summary of column testing methods (from Halkola, 1999) (continued).

METHOD	DETAILS	ADVANTAGES	DISADVANTAGES
4. CPT	• First used in Finland in late 1970s. • Now used in Helsinki "almost exclusively," even for semi-hard and soft columns. • At least 10 successful readings required.	• Very rapid and economical. • Lot of extra test columns possible. • Applicable in very variable conditions (hard to soft). • Provides data at the tip (exact depth profile). • Light equipment.	• Deviation can occur (but is easily recognized): generally < 7 m. • s_u < 1000 kPa. • 75-mm casing can allow testing to 20 m.
5. Static-Dynamic Penetration Test	• Developed for Finland and Sweden in 1980s. • Combines mechanical CPT and dynamic probing with same equipment. • Rods are rotated at 12 rpm, and resistance and torque measured to allow calculation of s_u.	• Used where column is so hard even CPT cannot penetrate (s_u < 2 MPa).	• Determination of sleeve friction and tip resistance in large columns has proven difficult. • Small diameter (45 mm) relative to column diameter. • Not as accurate as CPT. • Casing increases cost.
6. Other Penetration Methods	• SPT (Nishikawa et al., 1996).	• Equipment is "widespread." • Economic and simple method.	• Only number of blows counted: sleeve friction may cause seeming increase in strength with depth.
	• MWD (Measurement While Drilling) (drill parameter recording, experiments in Finland).	• Experiment only.	• Columns have been sufficiently soft not to need it yet.

Table 10. Summary of column testing methods (from Halkola, 1999) (continued).

METHOD	DETAILS	ADVANTAGES	DISADVANTAGES
7. Screw Plate Test	• Developed in Norway in the early 1970s and used in Sweden and Finland for soil compressibility in 1980s. • Plate screwed into column and then progressively loaded.	• "Only method which can be used to determine the column's real settlement properties in situ."	• Siow and expensive. • Determination of results not easy.
8. Geophysical Methods	• Resistance sounding (recent).		• Preliminary. • Expensive. • Difficult to interpret.
	• Seismic S-wave (Japan) data compared with U.C.S. data on disturbed cores.		• Slow and expensive. • Difficult to interpret.
9. Disturbed Samples	• Prepared by all methods used for sampling non-cohesive soil.	• Can be used to estimate binder composition.	
10. Undisturbed Samples	• In Japan, QC is "based almost entirely" on U.C.S. from core samples.	• Finnish simple tube sampler (60 to 95 mm) being experimented with.	• Disturbance of material during coring possible, especially in softer parts. • Expensive.
	• Full extraction. • pH testing gives binder content. • Pocket penetrometer illustrates homogeneity.		• Maximum depth in Finland 13 m.

installation techniques will permit a reduction in the number of columns tested and the use of simple penetration methods (CPT) in softer columns and core samplers in harder columns, "in which case the quality control costs will be at a reasonable level."

In passing, it may be noted from recent project data sheets from the Stabilator company (1999) that 0.5 to 1.0% of production columns are routinely tested (by normal penetrometer) in addition to the test columns which are first installed and tested. Their number varies with the scale of the project and the variability of the soil, but in one case (E18/E20 highway at Glanshammar, Sweden) 225 such columns were tested in 16 different areas along a 17-km project (63000 columns totally 343,000 lin. m). Hercules (1999) reported working on 10-km of the same project wherein 1200 of 140,000 columns were subjected to "production control."

3.4 General Overview

Throughout international practice, there is clear appreciation for the application of the successive steps that are taken to promote a successful DMM project, namely

1. Appropriate site investigation.
2. Laboratory testing of site soils blended with various types of binder to ascertain properties of treated soils, and to guide the subsequent field "starting point."
3. Pre-production field installation, closely monitored, tested, and analyzed.
4. Close and accurate control and monitoring of routine construction parameters during production, and verification that they are within the desired range.
5. Verification of properties of the treated soil.

Regarding the recording of construction parameters, it would appear that there exist three different levels of sophistication of data acquisition and display, ranging from simple manual efforts to total computer acquisition, display, and control. The deployment of each of these levels reflects national practice and history of usage, the nature of the particular DMM technique being used, and what is contractually required of the contractor in the specification. Typically, however, the majority of the methods feature a real-time computer display of relevant desired and actual construction parameters, which still permits operator adjustment whenever appropriate. It is becoming increasingly rare, even in "start up" operations in new geographic markets, for a purely manual system to be used, while total computer control is necessary to accommodate the sophistication and speed of some of the newer methods.

Improvements continue to be made, particularly in Japan, but also in Scandinavia where only one DMM technique is used (i.e., Lime Cement Columns). Prime targets are improving control over slurry (and binder) discharge parameters; improving definition of the "bearing horizon"; automation of the construction equipment and methods; and assessment of column verticality and straightness.

Six basic opportunities exist to determine the engineering properties of treated soil:

1. Laboratory: tests are conducted on (remolded) samples of the site soil that are combined with an appropriate range of binders. This is done prior to construction (or even specification writing) to verify or demonstrate what may reasonably and subsequently be expected of the treated soil in situ. Occasionally, further laboratory testing may be conducted at some later stage in the process to resolve some site-specific or forensic issues. It must be noted, however, that such laboratory tests, even when conducted with "standardized" methodologies (as is the goal in Japan and Scandinavia), provide only an "index" of the properties to be anticipated in the field, due to the strong influences exercised by scale effects, and geological and constructional variations. The most common test is U.C.S., although triaxial testing seems to provide more accurate, consistent, and meaningful results. Permeability is also a typical parameter investigated, while sample homogeneity can also be noted although it may be of limited practical relevance. Researchers report U.C.S. values two to five times higher than values obtained from field samples (especially those from wet grab techniques), but results from cores carefully taken from large DMM installations can provide higher strengths due to in situ "adiabatic" influences. Laboratory test data, typically and unsurprisingly, tend to show less scatter than data from in situ sampling.

2. Wet Grab Sampling: this method is particularly popular in the United States for sampling columns formed by wet methods prior to initial set. However, it encounters a number of practical, sampling problems, and the data are particularly variable and of questionable relevance in poorly mixed soil.

3. Coring: in Japan this remains the most common method of treated soil verification, less so in the United States and quite rarely in the relatively low-strength treated soils of Scandinavia. Relevant results are predicated on the use of good drilling technique and equipment, to ensure that samples are not inherently damaged prior to testing, and that even a heterogeneously treated column is correctly sampled, not just the most resistant portion. Samples are typically drilled vertically and should be taken all across the radius of a column to ensure representative data. There is strong debate as to the true relationship of U.C.S. data from cores to those from wet grab samples. However, it would seem that, provided good drilling technique is exercised, core strength can be as high as twice that of wet grab sample strengths. Relative to laboratory tests, core values seem to be around 50%, although the range can extend to 80% in cases of high mixing efficiency. The scatter of data from core samples appears to be higher than that from laboratory specimens. The Japanese continue to lead research into new coring techniques designed to impart the minimum disturbance to the treated soil sample.

4. Exposure, Extraction, and Block Sampling: this is most popular in Scandinavia (where treatment depths, diameters, and soil conditions are more conducive). Although relatively expensive and logistically demanding, the method provides many advantages including the opportunity to judge column homogeneity and other related strength and permeability parameters in directions other than vertical. On grounds of cost, it is typically restricted to pre-production testing.

5. Modified Geophysical Testing Methods; practitioners in Japan in particular are researching the use of existing geophysical techniques as means of assessing column strength, integrity, and homogeneity. Broadly, each can be described as "promising," having provided

reasonable correlation with data from cores, but it does not seem that any geophysical method is used routinely. Types listed are

- Shear Wave Seismic
- Borehole Resistivity Profiling
- Low strain sonic integrity testing/borehole sonar

6. Modified Geotechnical Testing Methods: especially in the Nordic countries where column strengths are relatively low, it is common to adapt existing geotechnical testing techniques to illustrate primarily undrained shear strength. Virtually all routine testing on installed columns in the Nordic countries is carried out by some form of penetrometer testing. Brief details are as follows; note that each method has its own advantages and disadvantages.

METHOD	RESEARCHING COUNTRIES	NOTES
Conventional Column Penetration (KPS)	Nordic countries	Used since 1980 in columns of s_u less than 200 to 300 kPa. Depth limit 6 to 8 m, aided by pre-drilling.
Inverted Column Penetrometer (FOPS)	Nordic countries	Used in Sweden since early 1990s for strengths up to 600 to 800 kPa and to depths of 20 m.
KTH Penetrometer	Sweden	New simple development, of promise.
Pressuremeter	Sweden/U.S.	Accurate test, especially for stronger columns, being promoted.
Dynamic Penetrometer	France/U.K.	Being used commercially in conjunction with Colmix system (Method 4).
Static/Dynamic Penetrometer	Finland/Sweden	Developed in 1980s but not as accurate as CPT.
Standard Penetration Test	Japan	Widespread, simple test, well known.
Cone Penetrometer (CPT)	Norway and Finland (since 1970s) less in Sweden	Despite systematic problems, can provide data in columns of c_u up to 1000 kPa, 20 m depth.
Modified Vane Test	Norway	Under development for c_u less than 200 kPa but use decreasing with use of CPT in 1990s.
Tube Sampler	Norway	Promising development but gives low strengths in heterogeneous columns.

Method	Researching Countries	Notes
Screw Plate Test	Scandinavia	Developed in early 1970s and is a very precise but expensive test.
Measurement While Drilling (MWD)	Japan, Finland	Good experimental results achieved in stronger columns through real-time monitoring of drilling parameters.

4. PROPERTIES OF GROUND TREATED BY "WET" METHODS

The most common parameters of treated ground that are measured and/or inferred on DMM projects are:

- Unconfined compressive strength
- Undrained shear strength
- E-value
- Tensile strength
- Permeability
- Compressibility
- Unit weights

In addition, there are relatively little data on Poisson's Ratio, drained shear strength, temperature development, or durability, while those DMM applications conducted for environmental remediation purposes typically include very detailed and sensitive chemical analysis data.

This chapter deals with data provided on ground treatment with the 21 "wet" methods identified in Table 1. These fall into the WRS, WRE, and WJE groupings, which employ a slurry as the binder for the native soil, and there is a great deal of data.

Section 4.1 reviews data obtained from "general" publications, or from "broad brush" statements, such as are provided by the specialty contractors in their promotional literature. It reviews the range of "binders" used, and comments on certain related operational factors such as water/cement ratio, cement factor, volume ratio, U.C.S., and permeability. Section 4.2 reviews data from the same or similar sources to investigate simple inter-parameter relationships (such as between U.C.S. and E) and time-dependent phenomena (such as rate of gain of strength development). Sections 4.3 and 4.4 describe the results obtained from particularly extensive and sophisticated research studies, in the laboratory and in the field, respectively. Section 4.5 summarizes general observations on the data presented in this chapter.

Chapter 5, which deals with the three "Dry" methods (i.e., DRE classification) follows a similar structure.

4.1 General Observations

4.1.1 "Binder" Materials

As noted by Harnan and Iagolnitzer (1992), the binder and its composition are most appropriately determined after careful consideration of the following:

- Physical binder/soil mechanism.
- Chemical binder/soil reaction.
- Pozzolanic property of any clay present.
- Required properties of the treated ground.

The following materials have been reported as being used, while new developments are continually under way (in materials as well as operational parameters). Their use is illustrated in Table 11, which also provides data on various operational parameters and the resultant treated soil properties:

- Portland cement: The most common, being typically Type I or II, and occasionally sulfate resistant. Hosomi et al. (1996) reported on significantly different treated soil properties being generated by cement from different suppliers.
- Bentonite: Added in relatively small amounts (less than 10% by weight of cement) to provide stability to high water/cement ratio slurries, and/or to help reduce treated ground permeability.
- Slag cement: For chemical stability and durability especially in brackish environments and to give lower strength (and often lower cost).
- Clay: Incorporated into bentonite cement slurries to promote construction of low permeability cut-offs (less than 10^{-9} m/s). Generally used (ASCE, 1997) in sites underlain by

Table 11. General tabulation of some published data (prior to 1999).

Ref. and Date	Method	Site	Data Type	Soil Type	Binder Type	Binder Composition	Cement Factor (kg/m^3)	Volume Ratio (%)	U.C.S. (MPa)	Shear Strength	E	Tensile	Permeability (m/s)	Others
Trevi ICOS (1998)	Multimix (3)	Casalmaiocco	Field	Granular	Cement, bentonite in sands Fluidifiers/ retarders	w/c = 0.5 to 1.0 (lower end in cohesives) Bentonite = 9% by weight of cement	200 to 250 (typical)	20 to 30 (typical) 15 to 40 (range)	0.5 to 5.0 (sands) 0.2 to 1.0 (clays)				10^{-7} to 10^{-8}	
		Pietrafinna	Field	Cohesives			250						-	
		Thailand	Field	Very loose sands (N = 1 to 3) Low plasticity clays (c_u = 15 to 35 kPa)	Cement plus sand	w/c = 0.6 Sand/cement = 0.5	150 to 320 200 (typical) up to 400 possible	30 to 50					-	
Bachy (1996) Harnan and Iagolnizter (1992)	Colmix (4)	Various France and United Kingdom	Field	Clays	Cement, lime, flyash "Special slurries to absorb heavy metals."	Lime: cement up to 50% WSR = 1.0 but variable	140 to 300	15 to 30	3 to 6 clays (50 to 100% increase from 7 to 28 days) Higher for sands	33% U.C.S.			< 10^{-7}	
CDM (1994)	CDM (6)	Various Japan	Field/ Lab	Clays, silts, fills	Portland cement; slag cement; Accelerators, dispersants, stabilizers (NB FGC)	w/c – 0.6 to 1.3 (typically 1.0)	240 to 320 (up to 400 for lower permeability)	12 to 20	2 to 4 control-lable	50% U.C.S. for U.C.S. < 1 MPa. Great for U.C.S. > 1 MPa			10^{-8} to 10^{-9}	
Geo-Con (1996)	SSM (7)	Various, United States	Field	Granular	Cement, bentonite flyash, lime and proprietary reagents.	Typically w/c = 1.0 (range 1 to 1.75)	150 to 400	23 to 35	3.5 to 10.0				as low as 10^{-10}	
				Sludges	Special reagents for environmental applications.		100 to 300	20 to 35	0.6 to 1.2					
Taki and Bell (1998)	SCC (8)	Various, United States and Japan	Field	Sands, cohesives	Cement; bentonite, lime/flyash for organics	0.6 for soft silts/clays; 0.8 for firm clays and 1 to 1.2 for sand	200 to 400			33% U.C.S.			10^{-8}	
Millgard (1998)	MecTool (9)	Various, United States	Field	Various	Cement; Type F ash; environmental additives; clay dispersant	WSR = 1 to 1.5	100 to 300						10^{-8} to 1.8×10^{-9}	
Mizutani et al. (1996)	Rectangular 2 (11)	Japan	Lab	Fine sands	Cement	w/c = 1.0 to 1.2	100 to 200							
			Field	Clays			150	15 to 30						
Terra (1998)	Single Auger Mixing (13)	Various, United States	Field	Various	Cement and environmental materials	w/c = 0.75 to 1.0								
Hayward Baker (1997)	Single Axis Tooling (15)	Various, United States	Field	Clays	Cement, bentonite, flyash, lime, blast furnace slag	w/c = 1 to 1.5			3.5 to 10 (granular) 0.2 to 1.4 (cohesives)					

Table 11. General tabulation of some published data (prior to 1999) (continued).

Ref. and Date	Method	Site	Data Type	Soil Type	Binder Type	Binder Composition	Cement Factor (kg/m^3)	Volume Ratio (%)	U.C.S. (MPa)	Shear Strength	E	Tensile	Permeability (m/s)	Others
Inquip (1998)	Rotomix (16)	Various, United States	Field	Various	Cement and biological agents	w/c = 0.8 to 2.0	> 100	> 15	> 0,1				10^{-8}	
Kawasaki (1996)	SWING (17)	Various, Japan	Field	Mainly clays	Cement		450		0.4 to 4.4 (lower in jetted zone)				10^{-8}	Lab strength is two times field strength
Miyoshi and Hirayama (1996)	JACSMAN (18)	Test site, Japan	Field	Mainly silty sands and clays	Cement	w/c = 1.0	320 mixed plus 200 jet	20 to 30%	1.2 to 5.8					Air entrainment reduces strength
Condon Johnson Associates (1999)	GeoJet (20)	Various, United States	Field	Mainly clays	Cement	w/c = 0.5 to 1.0	150 to 300	20 to 40%	4.83 to 10.3 (stiff) 0.7 to 5.5 (soft)					
Geo-Con (1998)	Hydramech (21)	Texas A&M	Field	Sands	Cement	w/c = 1.0 to 1.50	100 to 250		2.8		100 to 300 x U.C.S.			
Hosomi et al. (1996)	CDM (6)	Tianjiu Port, PRC	Field	Marine silts and clays	Cement	w/c = 1.1 to 1.5	150 to 170		2.5 target					
Min (1996)	CDM (6)	Yantai Port, PRC	Field	Marine silts and thin sand layers	Cement	w/c = 1.3	170 to 190		2.5 target (90 days)					
Isobe et al. (1996)	RAS (10)	Osaka, Japan	Field	Soft sand and silt (N = 3 to 6)	Cement	w/c = 0.8	300	33%	5.1 to 6.0					
Kawasaki et al. (1996)	SWING (17)	Higashi Ogishima, Japan	Field	Soft clay	Cement	Seawater	200		0.4 target (1.2 lab) 0.7 to 1.8 typical					
Ryan and Walker (1992)	DSM (1)	Crofton, BC	Field	Hard fill over 3.6 m of loose sand and 1.2 m dense sand	Cement	w/cc = 1.8	180		Min. 0.9	30% U.C.S.				
Geo-Con (1998)	DSM (1)	General data	Field	Various	Generally cement ± bentonite ± substitutions, e.g., ash, slag; ± additives ± clay	w/c = ½ to 1/75 (higher during penetration) Bentonite up to 5% by weight of cement	120 to 400	15 to 40%	Organic soft clay Medium/hard clay Silts Silty sands Sands	0.4 to 1.0 0.5 to 1.4 0.7 to 2.1 0 to 2.5 2.1 to 3.5 2.8 to 11.0		300 to 1000 x U.C.S.		10^{-7} to 10^{-8} (10^{-10} with bentonite)
Master Builders (1998)	-	-	Lab	Boston Blue Clay	Cement only Cement and dispersant	w/c = 1.5 w/c = 1.0	370 210			1.2 2.4				

Table 11. General tabulation of some published data (prior to 1999) (continued).

REF. AND DATE	METHOD	SITE	DATA TYPE	SOIL TYPE	BINDER TYPE	BINDER COMPOSITION	CEMENT FACTOR (kg/m^3)	VOLUME RATIO (%)	U.C.S. (MPa)	SHEAR STRENGTH	E	TENSILE	PERMABILITY (m/s)	OTHERS
SMW Seiko (Yang, 1997)	SMW (2)	General Data	Field	General	Cement ± up to 5% bentonite ± flyash ± clay	w/c = 1.25 to 1/5 (sands) and up to 2.5 (clays)	250 to 350 (typical) up to 750	50% (sand) to 100% (clay)	0.3 to 1.3 (clay) 1.2 to 4.2 (sand)	2 x U.C.S. for U.C.S. < 1 MPa, reducing as U.C.S. increases	500 to 1350 x U.C.S. 400 to 600 for cohesives	8 to 14% U.C.S.	10^{-7} to 10^{-10}	
		Lake Cushman Dam, WA	Field	Glacial till and outwash	Cement and bentonite	w/c = 1.3 Bentonite 7%	220		Up to 1.28					
		C07 Central Artery, Boston	Field	Various glacial soils and fill	Cement and bentonite	w/c generally 1.25 but various (1.0 to 1.25)	200 to 450 Typically 250 to 350		Target 0.6 (but not strong correlation) typically 0.5 to 2.5		50 to 150 x U.C.S. (lab)			Lab tests indicate strength related inversely to water content. Unit weight = 14.7 ± 1.3 kn/m^3 (20% less than virgin)
		EBMUD, CA	Field/Lab	Plastic clay	Cement and bentonite	w/c = 2.62 Bentonite 2%	286		0.48 to 1.03					
		Ikoma Tunnel Japan	Field	Sands and gravels	Cement and bentonite	w/c = 1.50 Bentonite 4.5%	450							
		Various	Field	Sand Gravel Silt and Clay	Cement and bentonite	Various (typically 1.0 to 1.5) but Bentonite up to 5%	up to 380 up to 400 up to 450		up to 4 (wet samples) and 7 (core) up to 4 (wet) and 4.6 (core) Up to 1.4 (wet) and 2.1 (core)					

interbedded fine and coarse grained soils. Typically used for pollution containment at sites with low differential heads.

- Sand: Very infrequently used, but can act as a cheap filler and to provide mechanical strength in very soft soils.
- Kiln dust: Used alone, or together with cement, flyash (and lime) for treating soils with heavy metals, acidic soils, and sludges.
- Flyash: Added as a substitute for cement (up to 25% by weight of cement) to reduce cost, increase chemical durability, and reduce heat of hydration within the treated mass.
- Lime: Used occasionally for commercial reasons and/or when treating soils with high organic content. It can also be used to substitute up to 50% of the cement (Matsuo et al., 1996) to provide a fluid, economic, but lower strength grout and treated ground (Figure 90).

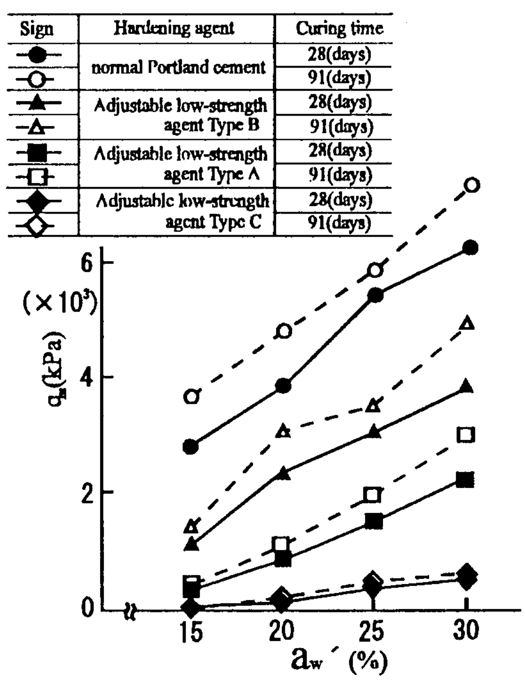


Figure 90. Effects of the adjustable low-strength agent (Matsuo et al., 1996).

- Additives: Principally dispersants (to provide good fluidity for lower water/cement ratio slurries) or retarders. Dispersants have also been used to improve drill penetration rates and enhance mixing efficiency in stiff cohesive soils, and anti-washout agents may be used in dynamic water situations (Gause, 1998).
- Gypsum: Being used in FGC variant of CDM (Method 6) in an effort to reduce the cost of slurry, and control the strength of treated ground.
- Others: For special environmental purposes, various proprietary admixtures and materials are used. These are not common in geotechnical applications. Porbaha (1998) notes that cement may be used in conjunction with silicates, thermoplastics, and polymers for the treatment of organic contaminants.

The CDM Association (1994) notes that both fresh and seawater can be used in grouts without affecting strength, with the former being most commonly used in land-based projects.

4.1.2 Water/Cement Ratio

There is a wide overall range (0.5 to 2.5 by weight), although most techniques use a much narrower band (0.8 to 1.2). There is a tendency for each technique to use lower values when treating clays and silts (to minimize the extra water introduced into the system), and higher ratios (by say 50%) in sands and gravels to promote mixing efficiency. In addition, certain techniques (e.g., DSM) use water/cement ratios up to 25% higher during the penetration phase, to try to help the penetration rate and mixing efficiency and to try to slow hydration-related stiffening. Bentonite is often added to slurries used in the WRS methods. The actual in situ water/cement ratio of the treated soil will reflect the natural moisture content of the ground, as influenced by any pre-drilling activities.

4.1.3 Cement Factor

This would seem to range usually from 100 to 400 kg/m^3 (although some projects may need higher values) and has a clear control over subsequent treated ground strength and other properties. Like volume ratio, these data are taken to refer to the injected quantities, which may not necessarily be those that remain in place.

4.1.4 Volume Ratio

Volume ratio reflects the type of DMM technique used, being higher for certain WRS methods to promote efficient penetration and to enhance mixing efficiency, but lower in other groups where the mixing efficiency is higher due to higher rotational speed or jet assistance. The higher the volume ratio the higher the amount of spoil (and also the amount of binder wasted in the spoil). In addition, higher volume ratios at water/cement ratios above, say, 0.6, will introduce increasingly large volumes of free water into the ground/slurry system, leading to lower set strengths, reduced durability, higher permeability, and more binder wastage (as surface spoil).

4.1.5 Unconfined Compressive Strength

The "wet" methods are usually designed to provide higher U.C.S. values (greater than 1 MPa) than the "dry" methods (up to 0.5 MPa). One notable exception is the FGC-CDM, where mix designs are selected to deliberately provide lower strengths to aid subsequent excavation and tunneling activities through the treated soil.

Bearing in mind that the unconfined compressive strength of the treated ground is, however, a result of many variables, including construction variability itself, the following broad observations can be made.

- Soil type is the most sensitive factor influencing the strength of treated ground. The same degree and type of treatment used in different soils produces results with a wide variation. The effect is attributed to the variations in the adsorption and pozzolanic reaction in the various soils as well as the reaction of the binder itself. Soil strength may limit the strength of the treated ground to a certain range beyond which the design is not cost efficient or even practical. Figure 91 shows the unconfined

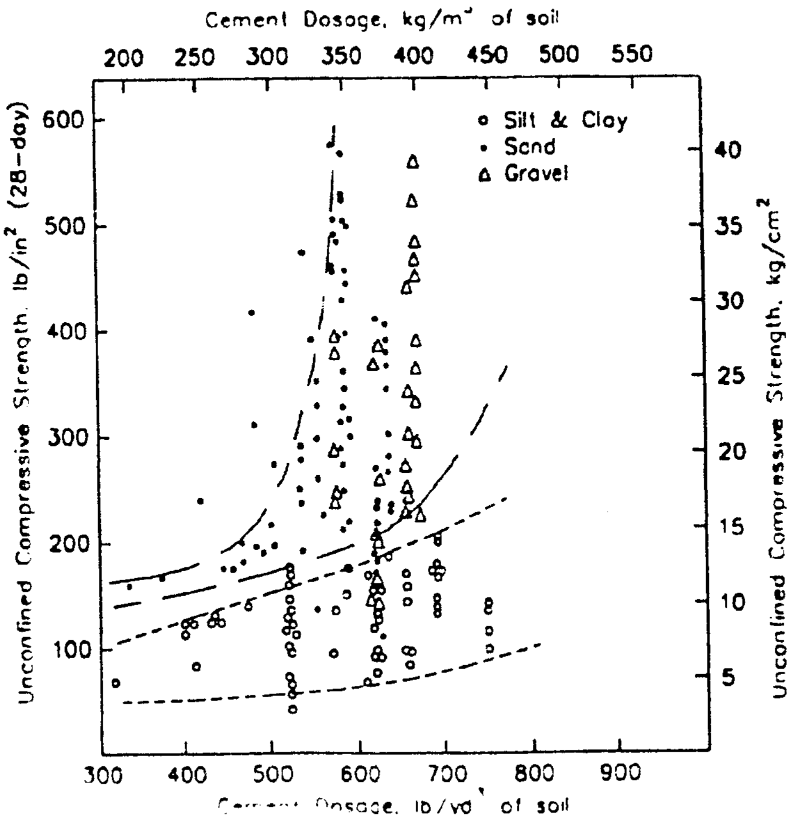


Figure 91. Data from excavation support wall projects
(Taki and Yang, 1991 and GeoLogan, 1997).

compressive strength (from wet grab samples) of SMW walls installed in clayey soil, sandy soil, and gravelly soil based on dozens of such projects. Although the strength increased with cement factor for each type of soil, the increase of strength in cohesive soils was slower in comparison with those in sand and gravelly soils. These data were for a range of water/cement ratios (typically 1.0 to 1.5) but most incorporated up to 5% bentonite (by weight of cement).

- Strength is related proportionally to apparent cement factor a_w (Figure 92) in the range of 10 to 30% at least.
- The strength of treated sandy soils is higher than that of treated cohesive soils by factors ranging from 2.5 to 5, all other variables being common.
- The denser or stiffer the virgin ground, the higher the strength of the subsequent treated ground.

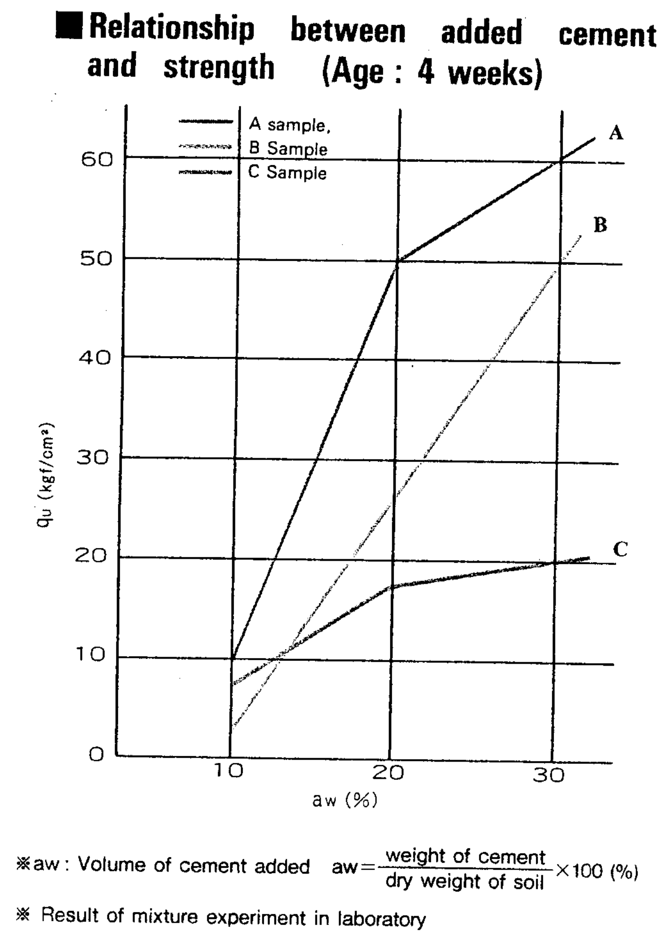


Figure 92. Relationship of treated soil strength to apparent cement factor (CDM Association, 1994).

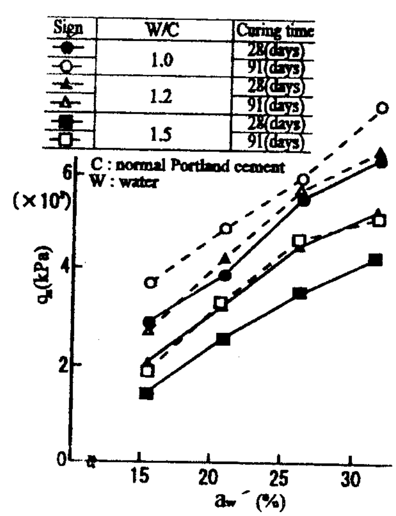


Figure 93. Relationship between cement slurry density and unconfined compressive strength (Matsuo et al., 1996).

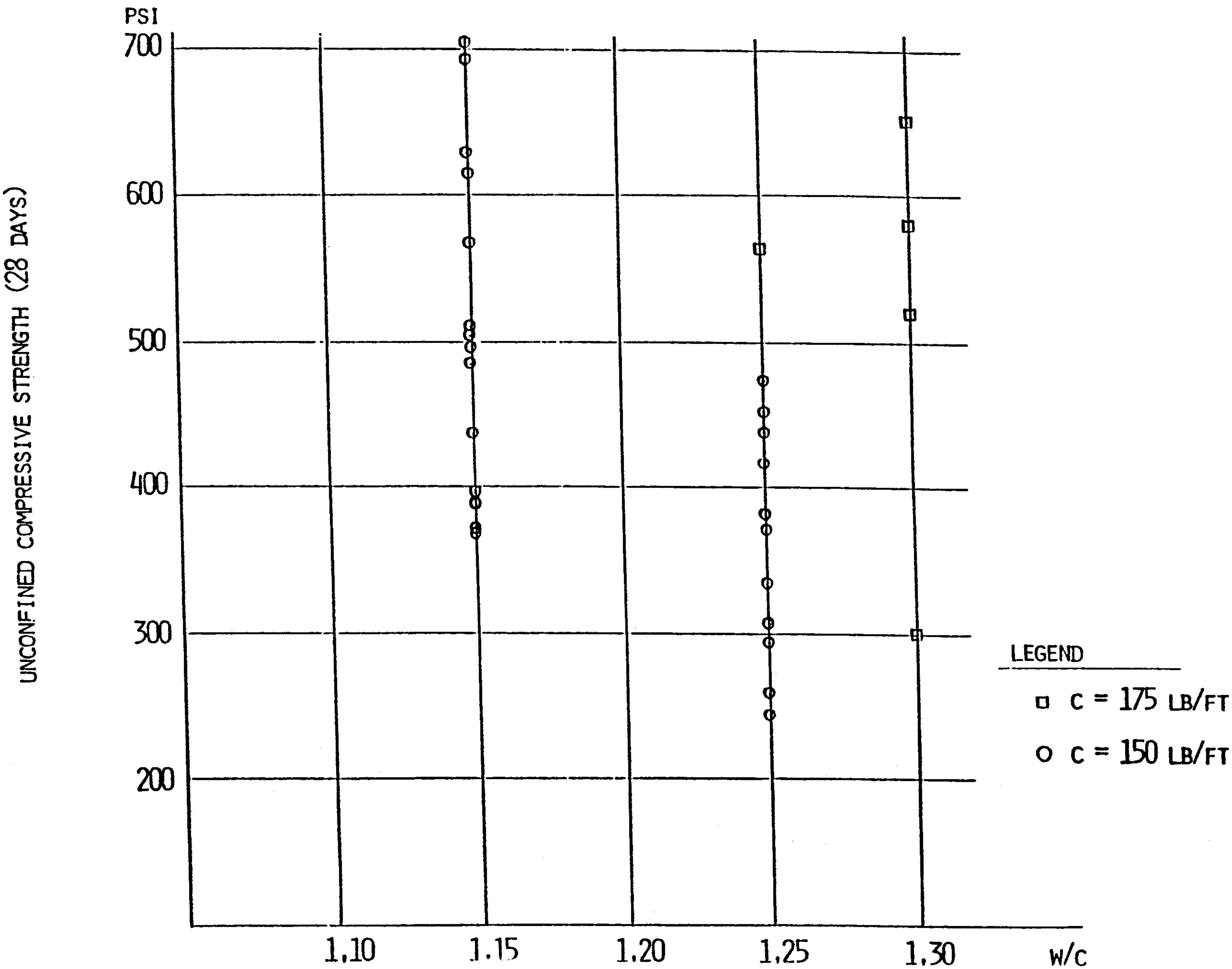


Figure 94. Unconfined compressive strength vs. water/cement ratio (Jackson Lake Dam, WY) (Jasperse, 1989).

- The water/cement ratio of the grout also controls strength (e.g., Figure 93 – between apparent cement factors of 15 and 30%, and Figure 94) possibly even more directly than cement factor.
- Air entrainment significantly reduces treated ground strength (Figure 95), although it may increase freeze-thaw resistance).
- There is a great deal of control available to designers in achieving certain design strengths in a wide range of soils by careful selection of binder type and appropriate injection parameters (e.g., Figures 96 and 97).

Notwithstanding all the known variables, both natural and selectable, it is reasonable to expect 28-day U.C.S. values in the following broad ranges (as determined by cores):

Organic and very plastic clays, sludges	Up to 1.0 MPa
Soft clays	0.4 – 1.5 MPa
Medium/hard clays	0.7 – 2.5 MPa
Silts	1.0 – 3.0 MPa
Fine-medium sands	1.5 – 5.0 MPa
Coarse sands and gravels	3.0 – 10 MPa

Unconfined compressive test data are used for design, construction quality control, and quality assurance. Sugimura (1997) proposes that the "design strength" (F_C) of the treated ground (Figure 98) can be calculated from the average of the unconfined compressive strengths (q_{uf}) of core samples.

$F_C = q_{uf} - 1.3\sigma$, and $f_C = 1/3\ F_C$

where $F_C =$ Design strength.

$f_C =$ Required strength under normal static loading.

$\sigma =$ Standard deviation of test results.

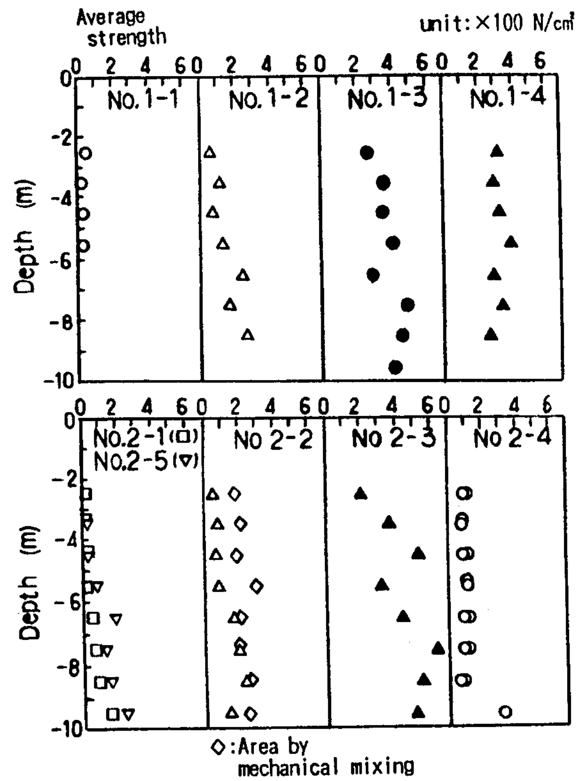


Figure 95. Effect of air entrainment on strength (Miyoshi and Hirayama, 1996).

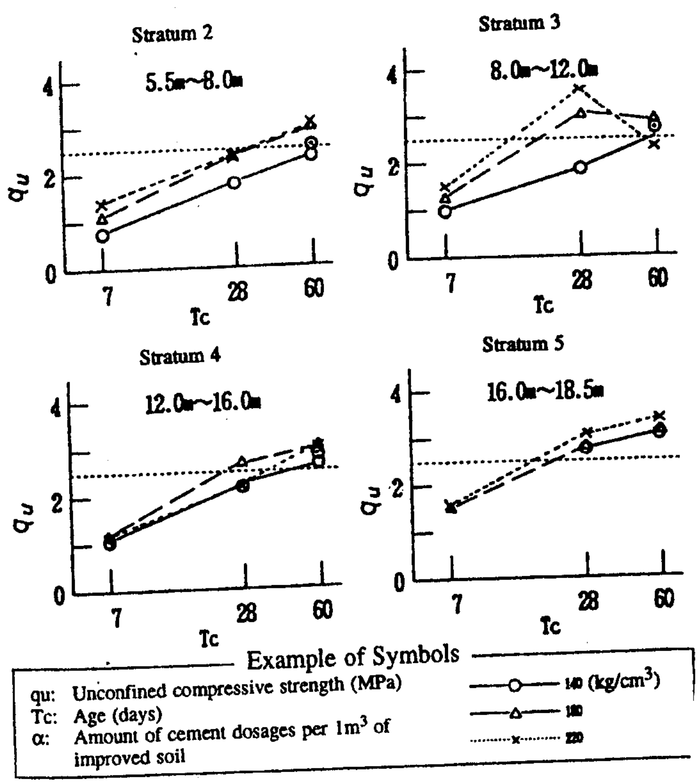


Figure 96. Laboratory test results with #500 cement, water/cement ratio = 1.00 (Hosomi et al., 1996).

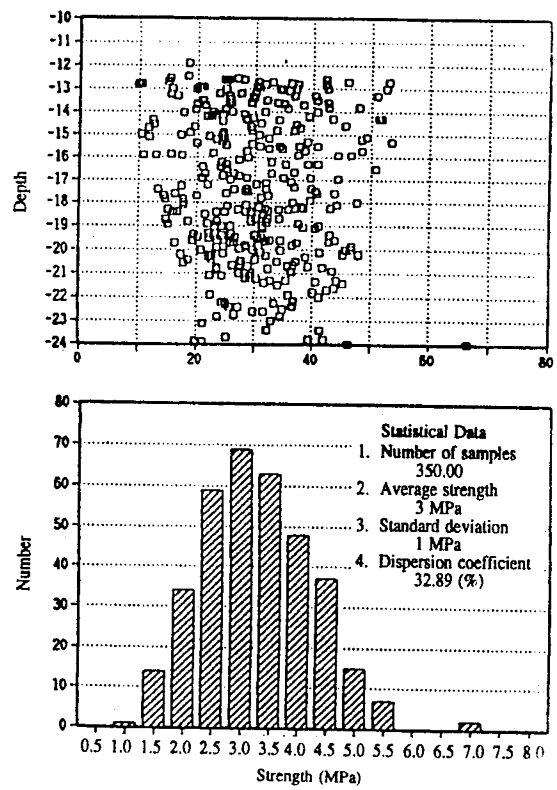


Figure 97. Results of unconfined compression tests from field samples (Hosomi et al., 1996).

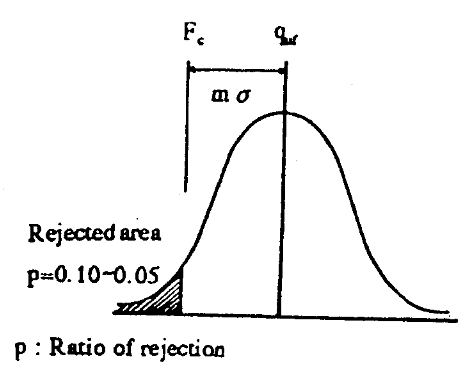


Figure 98. Design strength concept (Sugimura, 1997).

4.1.6 Permeability

Table 11 indicates that 10^{-7} to 10^{-8} m/s is a routinely achievable goal, but that results approaching 10^{-10} m/s are achievable only at considerable extra cost (e.g., by using more cement (Figure 99), or the use of bentonite) and in soils of a higher native clay content.

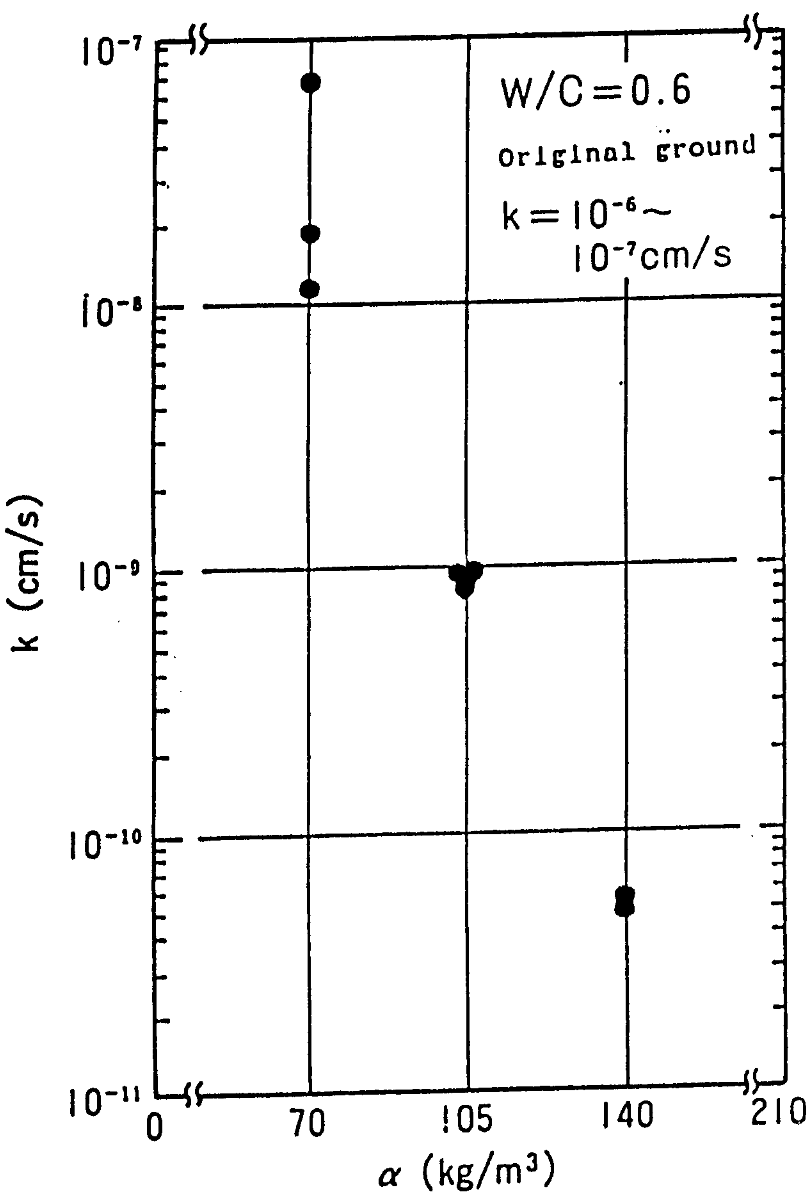


Figure 99. Relationship between coefficient of permeability and cement content for laboratory improved soils (CDM Association, 1994).

Taki and Yang (1991) stated that permeability is "affected" by soil type, cement factor (Figure 100), presence of bentonite, water/cement ratio, volume ratio, and age. They reported that permeabilities of 10^{-7} to 10^{-10} m/s are recorded from laboratory tests on wet grab samples taken during construction.

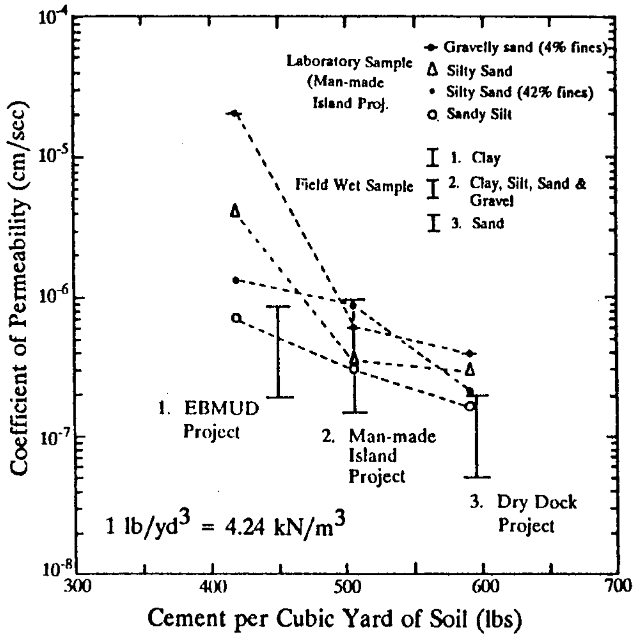


Figure 100. Permeability of treated soils (Taki and Yang, 1991).

Geo-Con, Inc. (1998) presented the data in Table 12 and summarized that strengths can generally be increased with additional cement. Permeability can be lowered with the use of bentonite or proprietary reagents. Geo-Con further stated that, although adding cement does decrease the permeability of soils, it generally is not to the order of magnitude generally required for effective in situ barriers. Bentonite is the superior material to mix into the soil for this purpose. Generally, cement is also added to create a soil-cement bentonite product, which will be relatively impermeable, flexible, and will have some reasonable strength. Table 13 shows results compiled from three projects.

Table 12. Strength and permeability data for ranges of cement factors and soil types (Geo-Con, Inc., 1998).

SOIL TYPE	CEMENT FACTOR	U.C.S.	PERMEABILITY
Sludge	240 to 400 kg/m^3	0.7 to 0.35 MPa	1 x 10^{-8} m/s
Organic silts and clays	150 to 260 kg/m^3	0.35 to 1.4 MPa	1 x 10^{-9} m/s
Cohesive silts and clays	120 to 240 kg/m^3	0.7 to 2.1 MPa	1 x 10^{-9} m/s
Silty sands and sands	120 to 240 kg/m^3	1.4 to 3.5 MPa	1 x 10^{-8} m/s
Sands and gravels	120 to 240 kg/m^3	3.0 to 7.0 MPa	1 x 10^{-7} m/s

Table 13. Strength and permeability data for typical bentonite-cement mixes (Geo-Con, Inc., 1998).

Soil types	Job type	Water/ cement ratio	Bentonite/ water ratio	Additional reagent ratios*	Grout/ soil ratio	U.C.S. 28-day (MPa)	Hydraulic conductivity m/s
Medium dense sand to sandy gravel, loose silty sand to soft sandy silt	Deep Soil Mixing (DSM)	1	0.04	None	0.32	0.42	9 x 10^{-10}
Sand, sand with clay	DSM	1.2	0.064	FA/W = 0.21	0.30	1.4 to 8.4	6 x 10^{-9} to 2 x 10^{-10}
CL-ML	DSM/Jet grout	1.5	0.02	FA:W = 0.4 G:W = 0.2	0.38	1.7	1 x 10^{-9}

*These include Flyash (FA) and Gypsum (Gyp)

4.2. Derived and Related Parameters and Properties

4.2.1 Rate of Gain of Unconfined Compressive Strength

Published rate of strength gain data are shown in Table 14.

Table 14. Published sources of rate of strength gain data

SOURCE	STATEMENT (ON U.C.S.)
Walker (1994)	28 days = 2.0 x 4 days
CDM (1994)	60 days = 1.4 to 1.5 x 28 days
Taki and Bell (1998)	28 days = 1.4 to 1.5 x 7 days (silts) 28 days = 2.0 x 7 days (sands) Mixes with high water/cement ratio have no strength gain after 28 days
Geo-Con, Inc. (1998)	56 days = up to twice 28 days with slower decrease to 160 days
Bachy (1994)	28 days = 1.5 to 2 x 7 days (sands)
Hosomi et al. (1996)	60 days = 1.4 to 1.5 x 28 days
O'Rourke et al. (1997)	28 days = 3 days + 0.62 MPa (clays)

Typical data are provided in Figures 101 through 103 (U.S. data) and Figure 104 (Japanese data).

It may be concluded from the information available that:

- Strength gain is slower in cohesive soils.
- Strength gain is slower when high water/cement ratio grouts are used, possibly being minimal after 28 days.
- 28-day strengths are 1.5 to 2.0 times 7-day values, and twice 4-day values.
- 56-day strengths can be up to twice 28-day values.
- Long-term strength gain generally continues after 56 days.

4.2.2 Tensile Strength

Takenaka (1995) reports that direct uniaxial tensile tests on 50-mm-diameter laboratory samples indicate a range of tensile strengths equivalent to 10 to 20% U.C.S. Saito et al. (1980) report that laboratory results from core splitting tests were lower than for direct uniaxial tests: for U.C.S. less than 6 MPa, the splitting tensile tests were 8 to 14% U.C.S. Takenaka (1995) confirmed this, citing strengths up to 10% U.C.S. CDM (1994) also summarized that the tensile strength

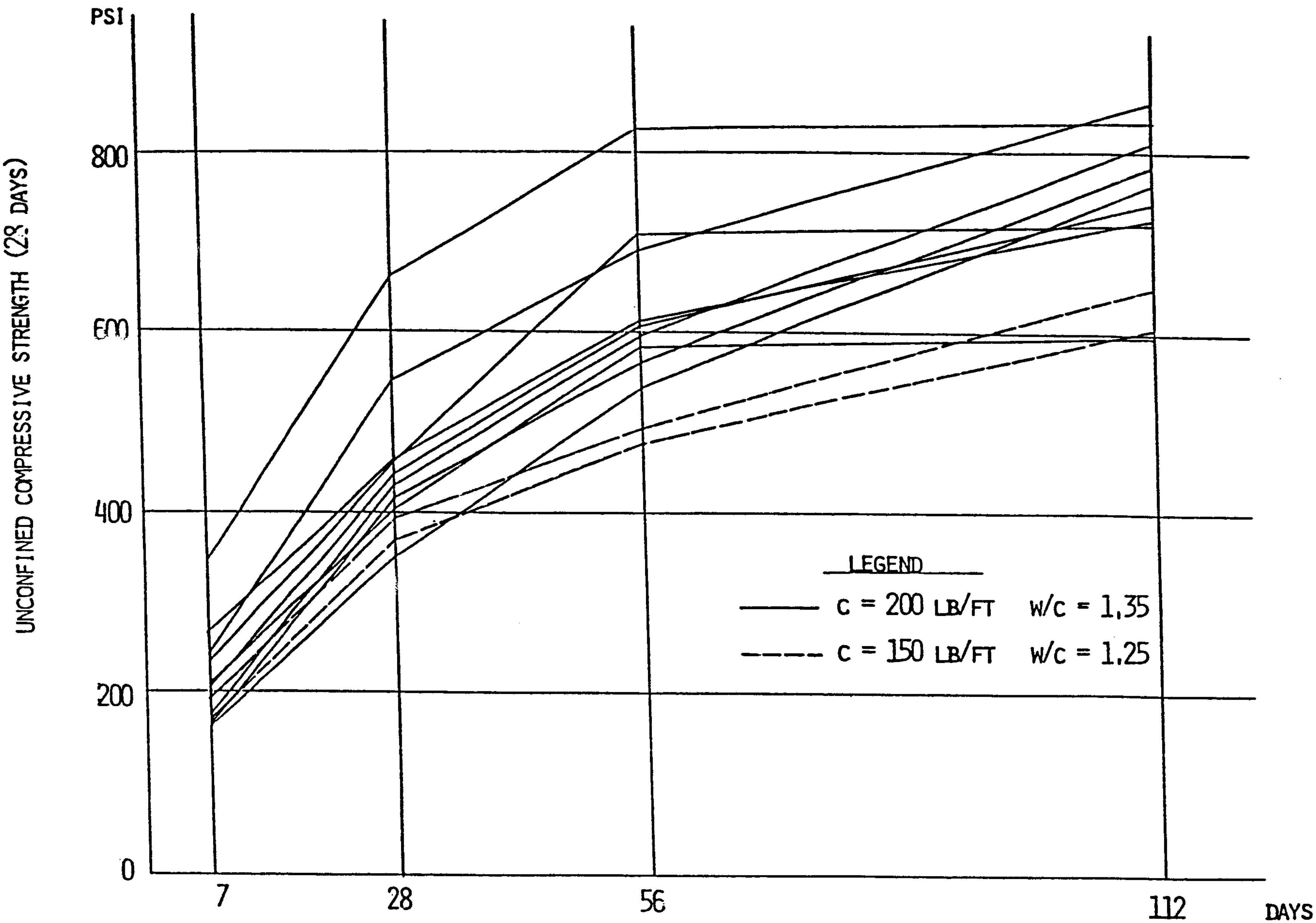


Figure 101. Unconfined compressive strength vs. time (Jackson Lake Dam, WY) (Jasperse, 1989).

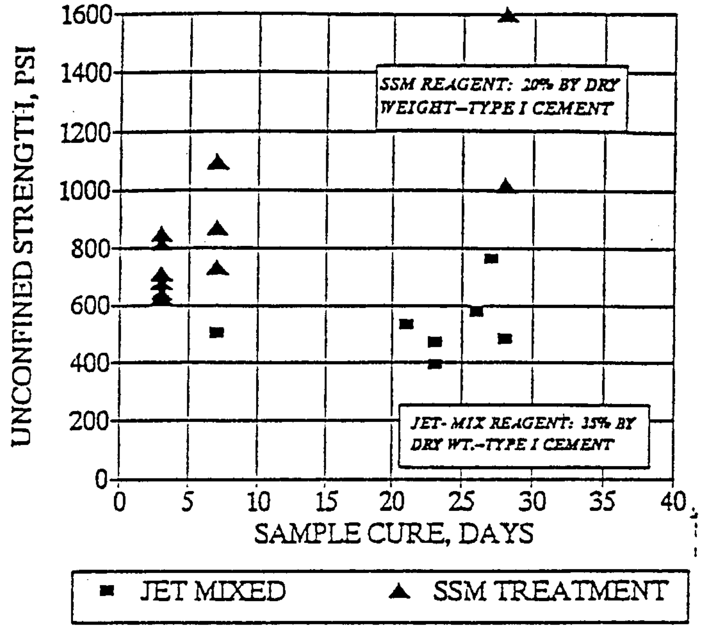


Figure 102. Strength (short term) in contaminated sand and silty clay (Ryan and Walker, 1992).

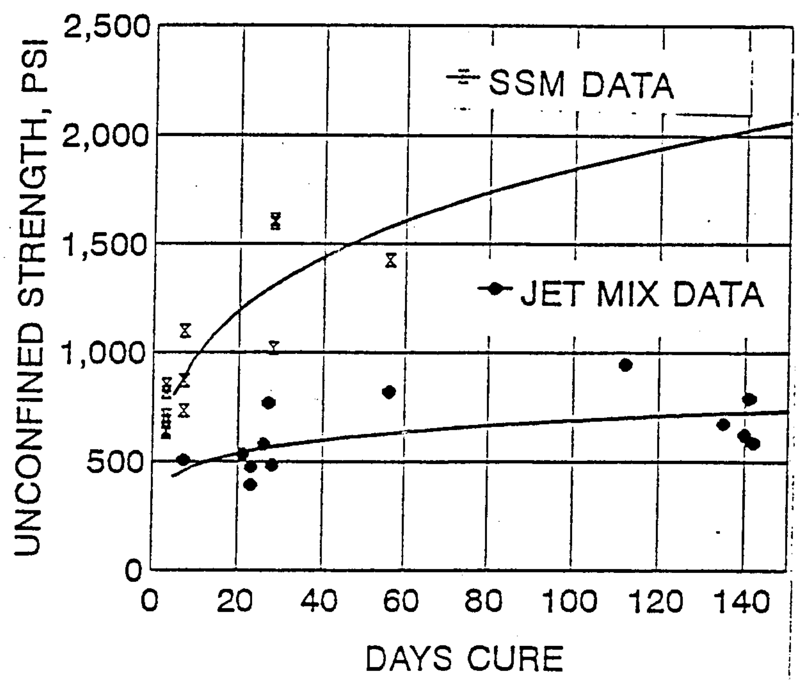


Figure 103. Strength (long term) in contaminated sand and silty clay (Ryan and Walker, 1992).

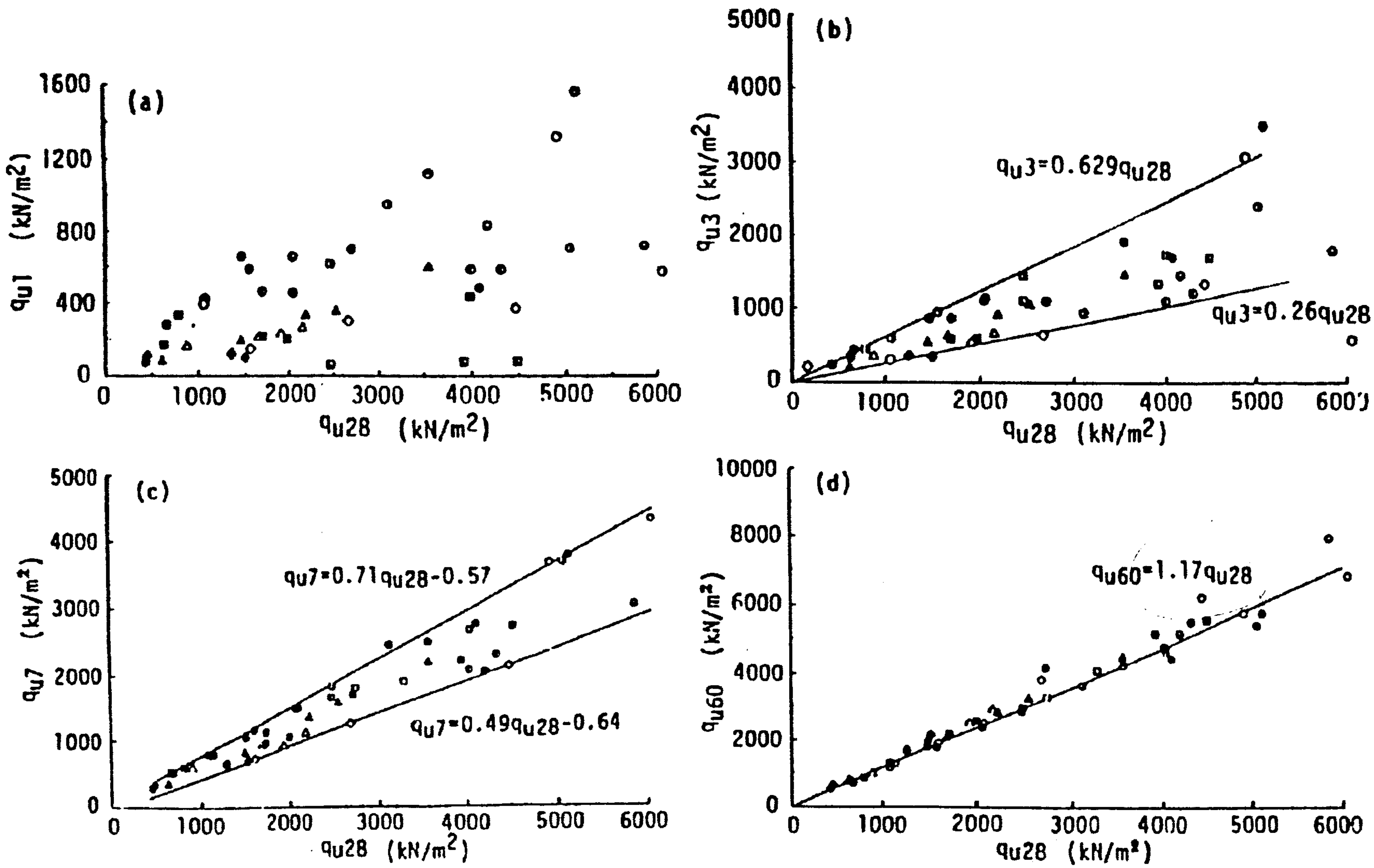


Figure 104. Relationship between unconfined compressive strengths q_u1, q_u3, q_u7, q_u60, and q_u28 (from Takenaka, 1995).

usually is 8 to 20% and that 15% is used regularly for design purposes. It may be concluded, therefore, that:

- The ratio of tensile to unconfined strengths is 8 to 20%, being in the upper ranges when measured by direct uniaxial tensile tests, but 8 to 14% when measured by splitting tests.
- Although 15% is used regularly by CDM for design, a lower value (e.g., 10%) may be prudent without site-specific direct tensile testing.

4.2.3 Shear Strength

Saito et al. (1980) reported that the relationship between U.C.S. and direct undrained shear strength is

$$\tau_o = 0.53 + 0.37q_u - 0.0014q_u^2 \ (q_u \leq 6 \text{ MPa})$$

where τ_o = 28-day shear strength (direct shear, no normal stress); and
q_u = 28-day U.C.S. (kg/cm^2).

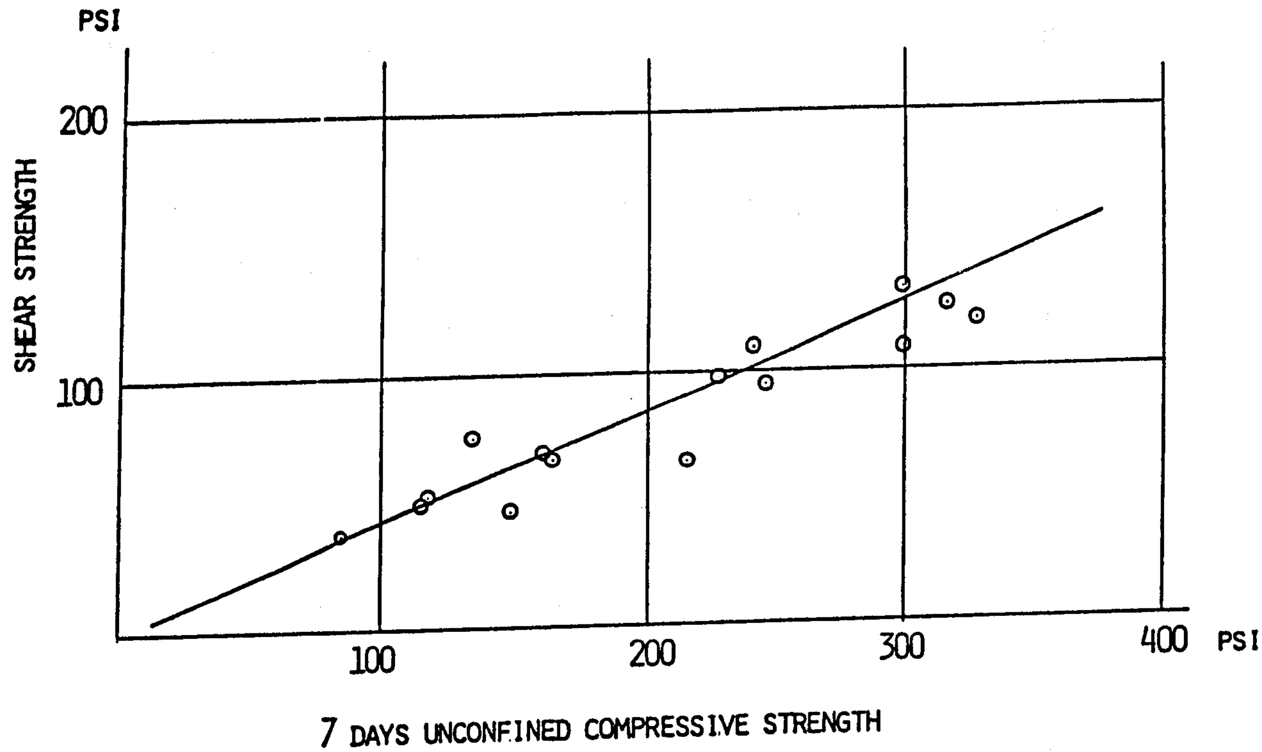
The q_u to τ_o ratio is about 2 when q_u is less than 1 MPa and reduces gradually as q_u increases. This relationship has been reaffirmed by CDM (1994).

Jasperse (1989) provided data (Figure 105) relating shear strength to 7-day (40% ratio) and 28-day (35% ratio) U.C.S., while Taki and Bell (1998) estimate shear strength as 33% U.C.S., as do Harnan and Iagolnitzer (1992).

Most recently, Sugimura (1997) proposed that the shear strength (F_τ) can be calculated from:

$$F_\tau = \tau_{so} + \sigma_n \tan \phi$$

where τ_{so} = shear strength at $\sigma_n = 0$ and σ_n = normal stress.



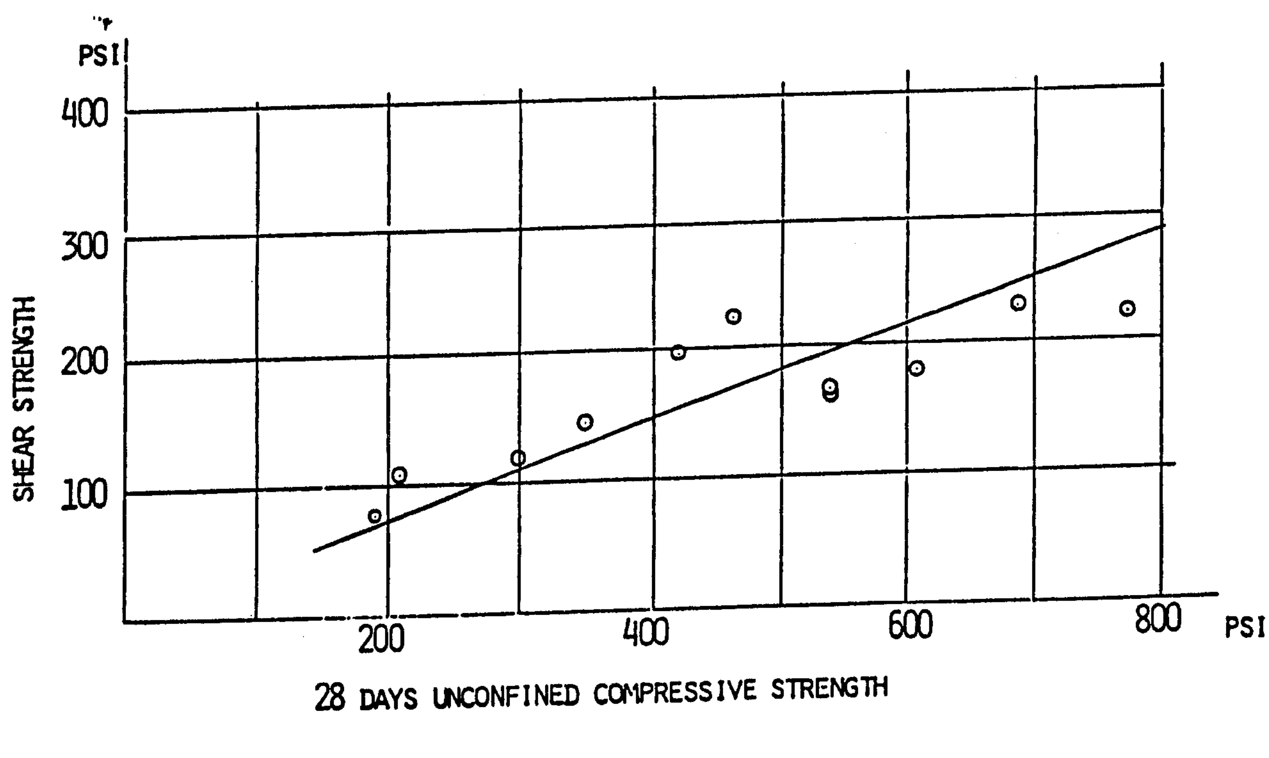


Figure 105. Unconfined compressive strength vs. shear strength (Jackson Lake Dam, WY) (Jasperse, 1989).

Japanese test data appear to indicate that the ϕ value for treated ground is about 30° for most soil types, and Sugimura notes that from direct shear tests $F_{\tau so} = 0.29\ F_c$, whereas from past research $F_{\tau so} = 0.337\ F_c$. For typical DMM situations, where $F_c \leq 2$ MPa, $F_{\tau so} = 0.3\ F_c$, where $F_{\tau so}$ is the design shear strength.

Yoshida (1996) conducted both triaxial (unconsolidated – undrained) and direct shear testing on 68-mm core samples from SCC treated silts and sands. He found that, within a range of U.C.S. from 2 to 5 MPa, direct shear strength was 23.3% of U.C.S. (Figure 106). However, data from the triaxial shear tests were 1.57 times higher, implying that the triaxial shear test data were 37% U.C.S.

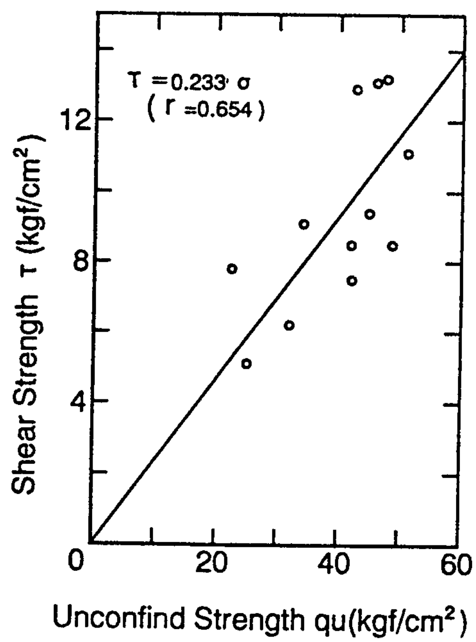


Figure 106. Correlation of unconfined compressive strength, q_u, and shear strength, τ (Yoshida, 1996).

It may be concluded, therefore, that the limited data suggest that ratios of 30 to 35% are found in treated ground of unconfined compressive strength over 1 MPa, while lower ratios (40 to 50%) may be more representative in weaker ground.

4.2.4 E_{50} Value (E_{50} is the secant stiffness modulus at 50% peak strength)

Published E_{50} versus U.C.S. relationships are shown in Table 15.

Table 15. Published E_{50} relationships with U.C.S.

SOURCE	ASSUMPTION (TIMES U.C.S.)	COMMENT
Suzuki (1982)	350 to 1000	Cores from field test
Taki and Yang (1991)	500 to 1350	SMW technique with bentonite mixes (Figure 107)
Bachy (1992)	50 to 100	Field data – clay: higher for sands.
CDM (1994)	400 to 600	Field data (Per Yang et al., 1998)
Takenaka (1995) and CDM (1994)	350 to 1000	Laboratory data (Figure 108)
Asano et al. (1996)	140 to 500 50 to 300	CDM technique (Figure 109) FGC technique (Figure 109)
Futaki et al. (1996)	144 (organics) to 209 (sand)	Cores from field test
Okumura (1996)	75 to 680 but ratio highest above U.C.S. = 1 MPa	Field data (Figure 110)
Kawasaki et al. (1996)	150 (higher strength) 100 (lower strength)	Field data
Ou and Wu (1996)	200 to 500 (typically 400)	Several sources
Okumura (1996) and Saito et al. (1980)	400 to 600	CDM in soils with less than 15% sand
Tatsuoka et al. (1996)	250 to 1000	Field and laboratory (Figure 111)
Yoshida (1996)	$-1280 + 263\ q_u$ (kgf/cm^2)	Field data (Figure 112)
O'Rourke et al. (1997)	50 to 150	Laboratory data on SMW (Figure 113)
Geo-Con, Inc. (1998)	300 to 1000 100 to 300	DSM technique SSM technique
Taki and Bell (1998)	180	For design purposes
Unami and Shima (1996)	50 to 500 (typically about 100 to 200)	Mass of CDM field data

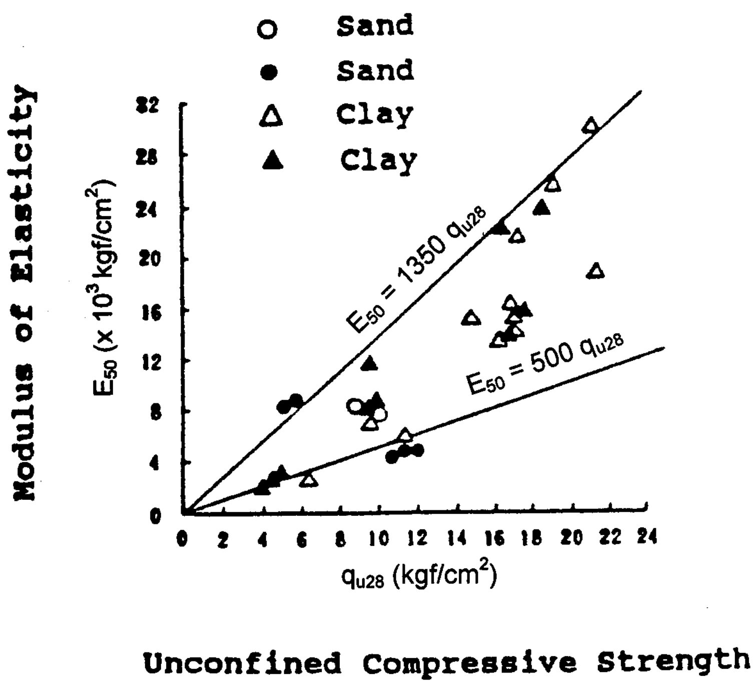


Figure 107. Relationship between modulus of elasticity and unconfined compressive strength in SMW (Taki and Yang, 1989).

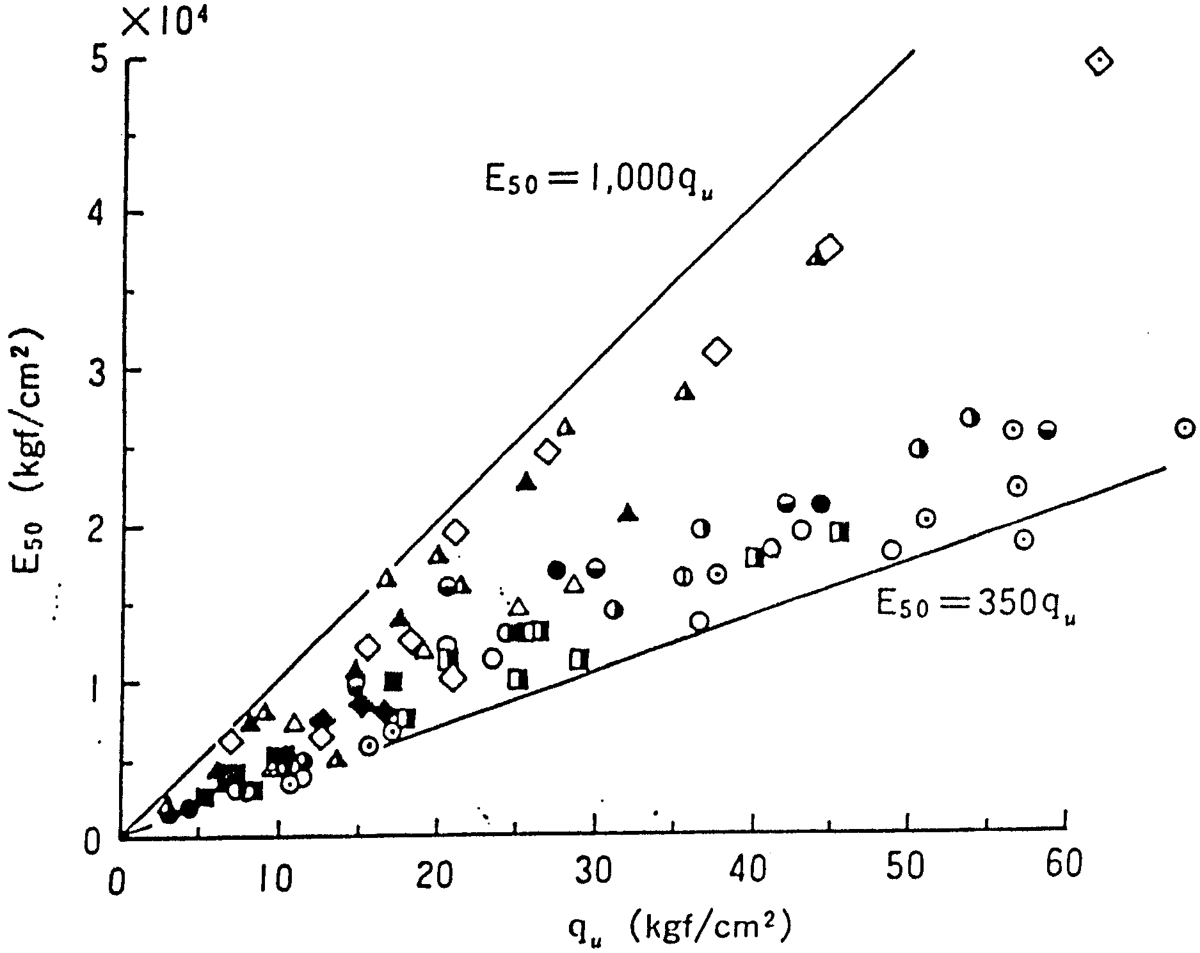


Figure 108. Relationship between unconfined compressive strength, q_u, and static linear modulus, E_{50}, for laboratory improved soil (CDM Association, 1994).

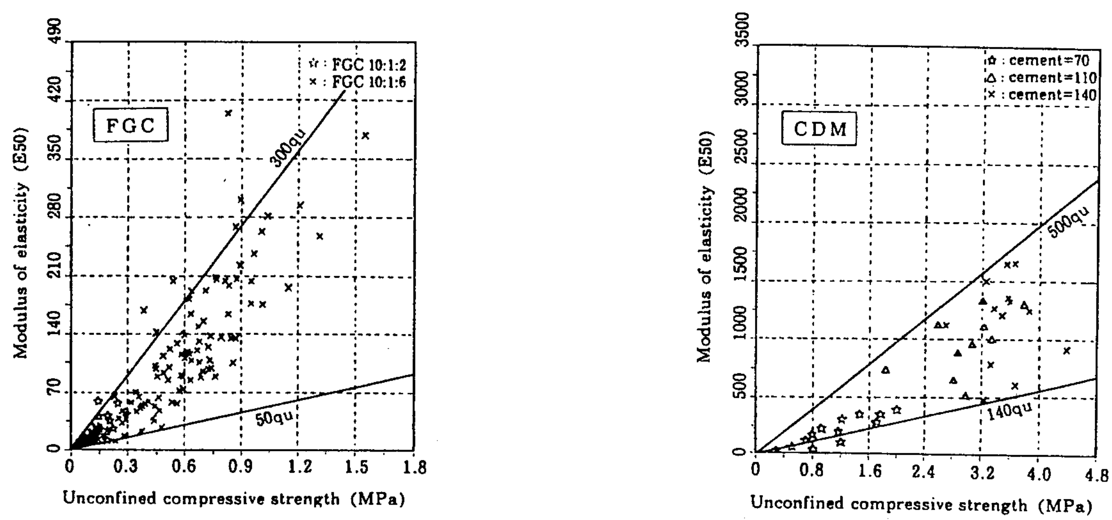


Figure 109. Relationship between unconfined compressive strength, q_u, and modulus of elasticity, E_{50} (Asano et al., 1996).

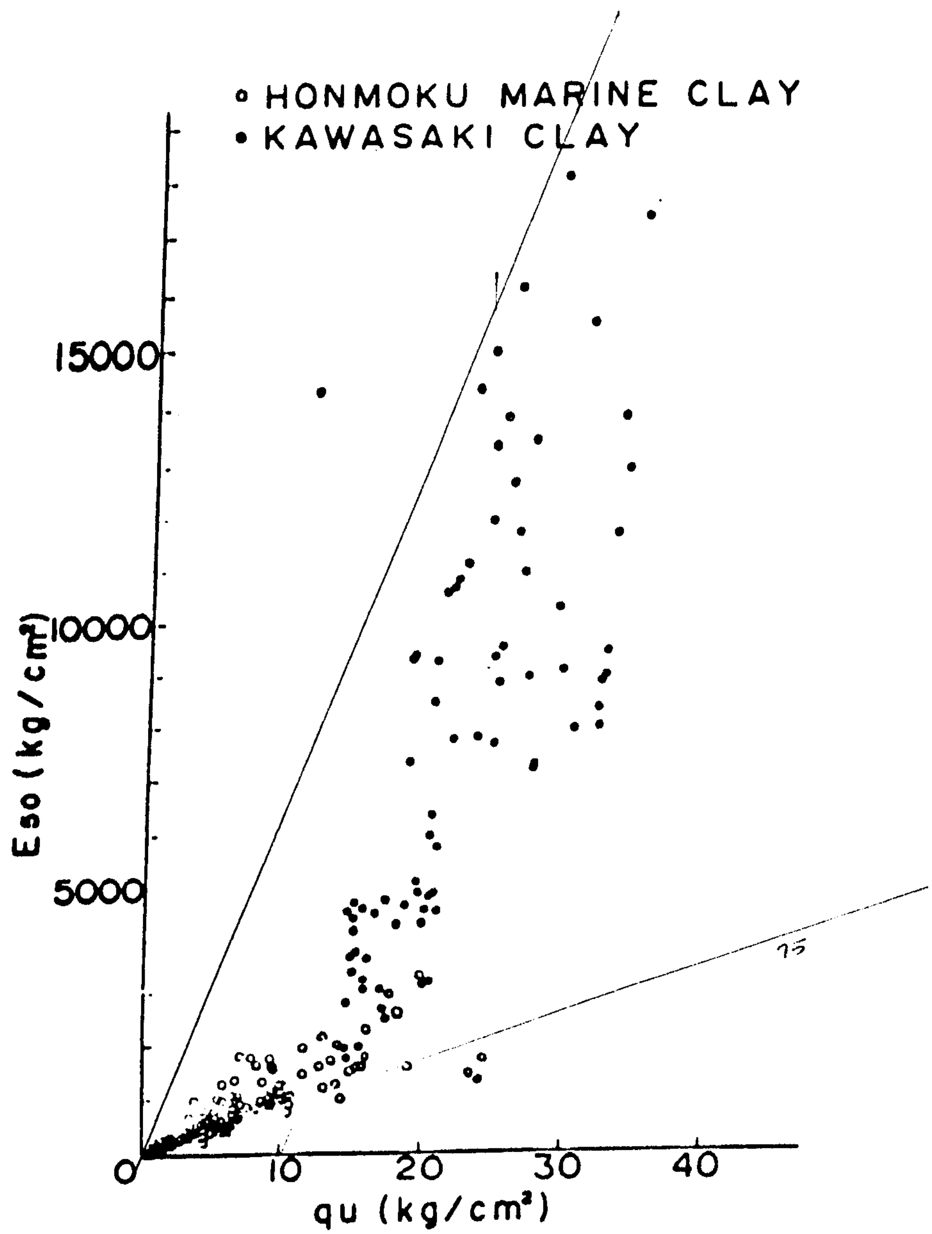
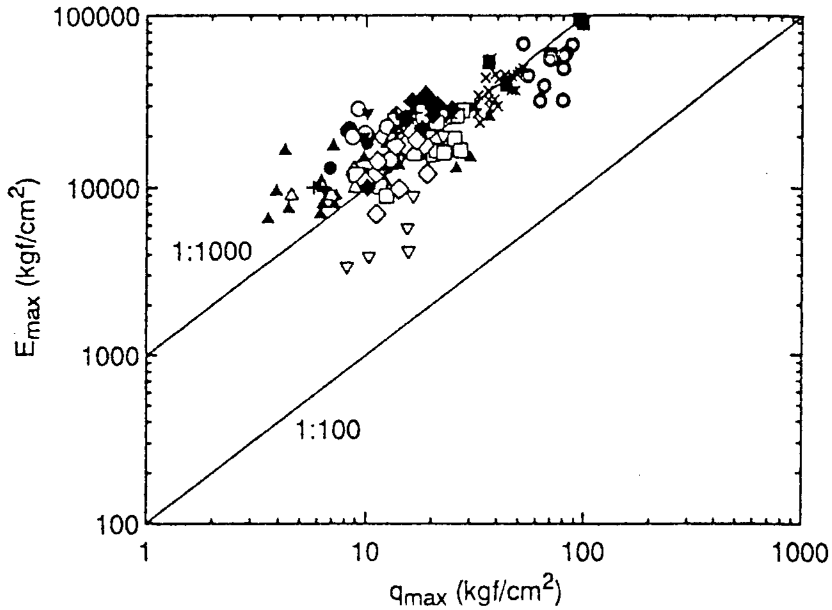


Figure 110. Relationship between strength and modulus of deformation for field samples (Okumura, 1996).



	Site and Material	Testing method
■	Hawaii, Mauna Kea, Cement-mixed cinder soil	CD TC tests
▲	Prepared in the Lab., Cement-mixed sandy soil	CD TC tests
△	Prepared in the Lab., Cement-mixed sandy soil	CU TC tests
▽	Ukishima Access, Low strength-type DMM, tc=91 days	CU TC tests
◎	Kawasaki Island, DMM, tc=2.5 years	CU TC tests
□	Kawasaki Island, Slurry type fill, tc=28-91 days	CU TC tests
◇	Prepared in a mold, Slurry-type, tc=14-91 days	CU TC tests
◆	Full-scale experiment, Slurry-type fill, tc=116-190 days	CD TC tests
+	Full-scale experiment, Slurry-type fill, tc=100-188 days	CU TC tests
×	Ukishima Access, Slurry-type fill, tc=300 days	CU TC tests
●	Full-scale experiment, Cement-mixed sandy soil (dry type)	CD TC tests
○	Full-scale experiment, Cement-mixed sandy soil (dry type)	CU TC tests
▼	Kisarazu Island, Dry-type fill, tc=120-290 days	CD TC tests

Figure 111. Summary of E_{max} versus q_{max} relationships for cement-treated soils (Tatsuoka et al., 1996).

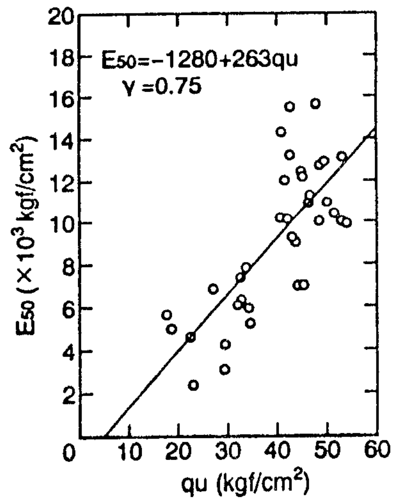


Figure 112. Correlation of unconfined compressive strength, q_u, and modulus deformation, E_{50} (Yoshida, 1996)

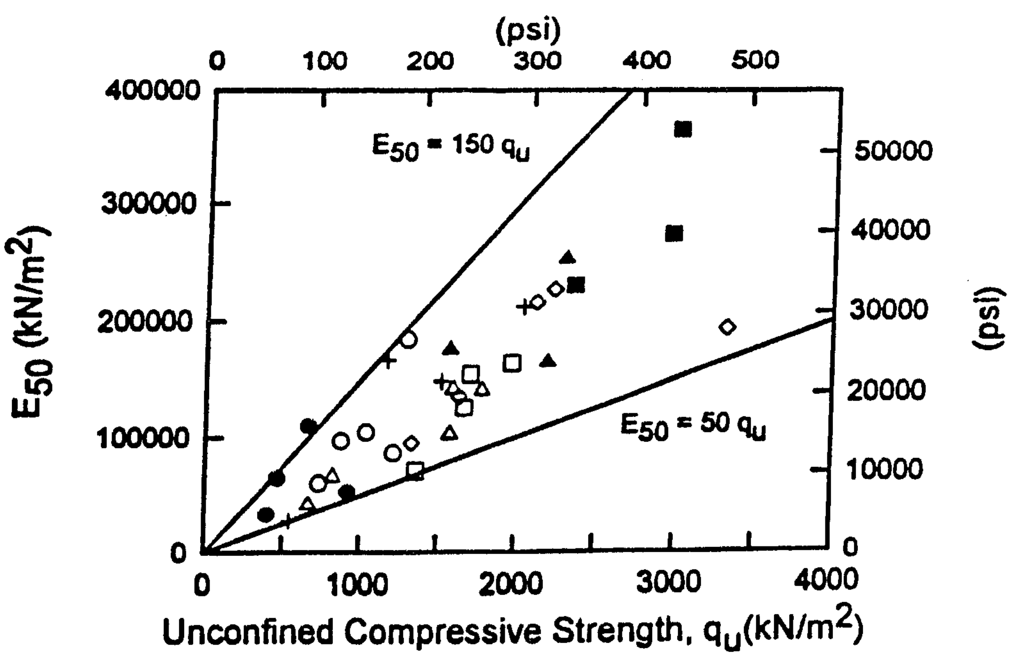


Figure 113. E_{50} versus q_u for laboratory prepared and cured soil-cement tested by Geotesting Express (O'Rourke et al., 1997).

It may be concluded, therefore, that

- There is a proportional relationship but it is very difficult to identify reliable boundaries to it.
- However, there is some evidence that the ratio is lower (50 to 300) in lower strength materials (say up to 2 MPa), but higher (300 to 1000) in higher strength treatments, with the ratio increasing with increasing strength. (The ratio for concrete is reported by Suzuki (1982) to be 1010.)
- It is controlled by the same parameters that control strength.
- It may be speculated that higher degrees of mixing efficiency may contribute to higher ratios (less untreated and potentially compressible native soil inclusions). In this regard, O'Rourke and McGinn (1999) noted considerable variability in stiff cohesive soils treated by the SMW method (Method 2) in a site in Boston, even over small distances (0.8 to 1.5 m).

Overall, O'Rourke et al. (1998) summarized that the Japanese data are largely taken from small core samples. They cannot therefore represent the large-scale effects that reflect variability in situ and relate to the overall design and construction concepts. Equally long-term relationships also are not well addressed.

4.2.5 Poisson's Ratio

The static Poisson's Ratio is as high as 0.49 if the material is undrained, and ranges from 0.3 to 0.45 under other loading conditions (CDM, 1994). Sugimura (1997) writes that from the results of triaxial compression tests and unconfined compression tests, Poisson's Ratio varies from 0.19 to 0.30. A value of 0.26 is usually assumed for design analyses in his experience.

4.2.6 Miscellaneous Properties

CDM (1994) notes that the rate of deterioration of soilcrete under exposure to marine conditions increases with the log of time.

Yoshida (1996) concluded no noticeable relationship (Figure 114) between density and U.C.S. for SCC columns cored in treated silts and sands.

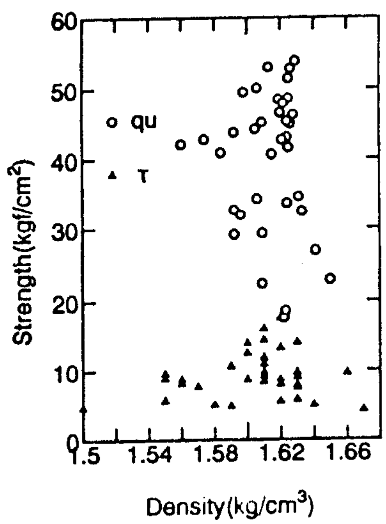


Figure 114. Correlation of density and unconfined compressive strength (Yoshida, 1996).

4.3 Special Laboratory Test Data

Taki and Yang (1990) presented data from laboratory samples made from a variety of sands and silts. The influence of cement factor (250 to 350 kg/m^3) on 7-day and 28-day U.C.S. is shown in Figures 115a and 115b, respectively; the effect on permeability is shown in Figure 115c, and the relationship between U.C.S. and permeability is shown in Figure 115d. It may be noted that permeability can be reduced in sands by adding cement (and bentonite), whereas reduction is much smaller and more costly to achieve in clays.

Pagliacci and Pagotto (1994) described a laboratory and field test of their WRE system (Method 3) for the foundation of a new steel plant at Bang Saphan, Thailand. The alluvial

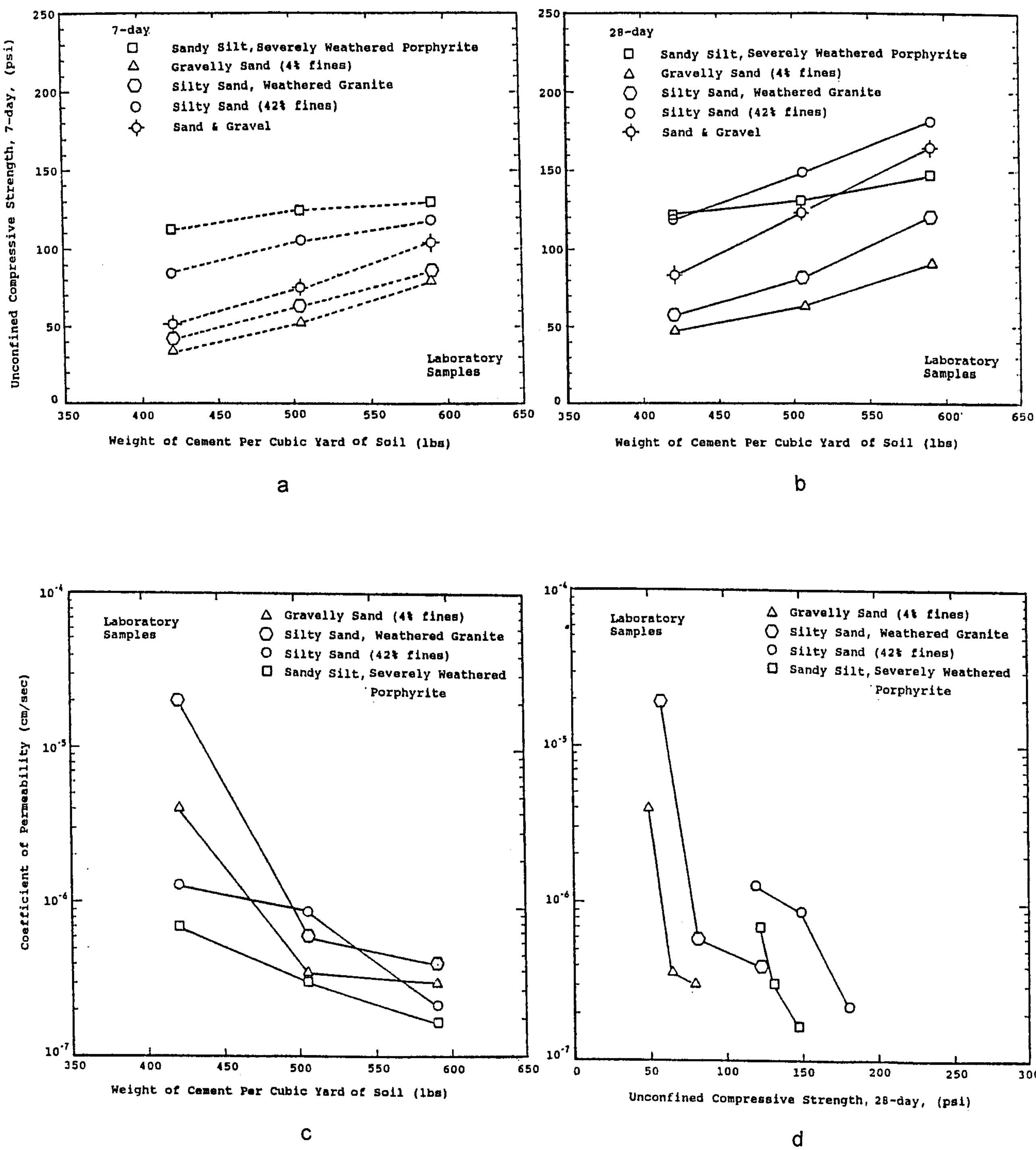


Figure 115. a) Average strength vs. weight of cement at 7 days, b) Average strength vs. weight of cement at 28 days, c) Coefficient of permeability vs. weight of cement at 28 days, and d) Coefficient of permeability vs. strength at 28 days (from Taki et al., 1990).

sediments were loose sands and soft clays of low plasticity from 5 to 16 m deep. The underlying marine sediments consisted of alternations of hard clay and dense sand from 16 to 38 m deep. Where treatment was anticipated, the soils in the first 5 to 12 m were very weak (N = 1 to 3; c_u = 15 to 35 kPa (CL); and about 20% of the relative density of the sands). The soil was slightly organic, and at a moisture content equal to or higher than the liquid limit.

Target U.C.S. was 1.2 MPa with E = 150 to 320 times this value. Initially a laboratory test was performed on a very soft plastic slightly organic clay (in Italy). Four pumpable grouts with different sand/cement ratios were used (Table 16) and tests were run at a cement factor equal to 250 kg/m^3. Two limit hypotheses were considered (Figure 116):

1. Volume of spoil = volume of slurry, and comprises both soil and slurry.
2. Volume of spoil = volume of slurry, but consists only of soil.

Experimental data are provided in Tables 17 and 18, and Figure 117, indicating the contribution of sand to strength development. Hypothesis 2 is preferable in the field, where Type C grout was used with a water/cement ratio of 0.6, sand/cement ratio of 0.5, and cement factor of 250 kg/m^3. Cores at 28 days gave 1.5 to 2.5 MPa (Figure 118). They estimated the wet method provides U.C.S. values in cohesive soils 30 to 50% lower than for dry methods (due to higher total water content).

Saitoh et al. (1996) demonstrated clearly how high quality laboratory testing should be conducted. Work by Takenaka engineers had indicated E_{50} = 350 to 1000 U.C.S. (Figure 108), while Figures 119 and 120 illustrate the influence of sand content (increases modular ratio), and water content (decreases modular ratio), respectively. Regarding tensile strength, Figure 121 shows the relationship of direct and splitting tensile tests to U.C.S. data obtained from lab samples, and Figure 122 shows the influence of water content. They concluded that the split tensile strength is 8 to 12% U.C.S. with the direct testing giving higher ratios. Regarding shear strength, rapid direct shear testing ($\sigma_n = 0$) of laboratory samples (Figure 123) indicated a ratio of 25 to 50% (decreasing as strength increases), while Figure 124 shows the effect on in-situ-treated clay of varying σ_n.

Table 16. Main characteristics of cement grout/mortars (Pagliacci and Pagotto, 1994).

MORTAR TYPE	S/C	C/W	FLOW-CONE VISCOSITY	U.C.S. 28 DAYS
(-)	(-)	(-)	(sec)	(MPa)
A	2.0	1.18	25	12.20
B	1.0	1.52	19	20.50
C	0.5	1.66	20	22.50
D	0.0	1.80	12	24.30

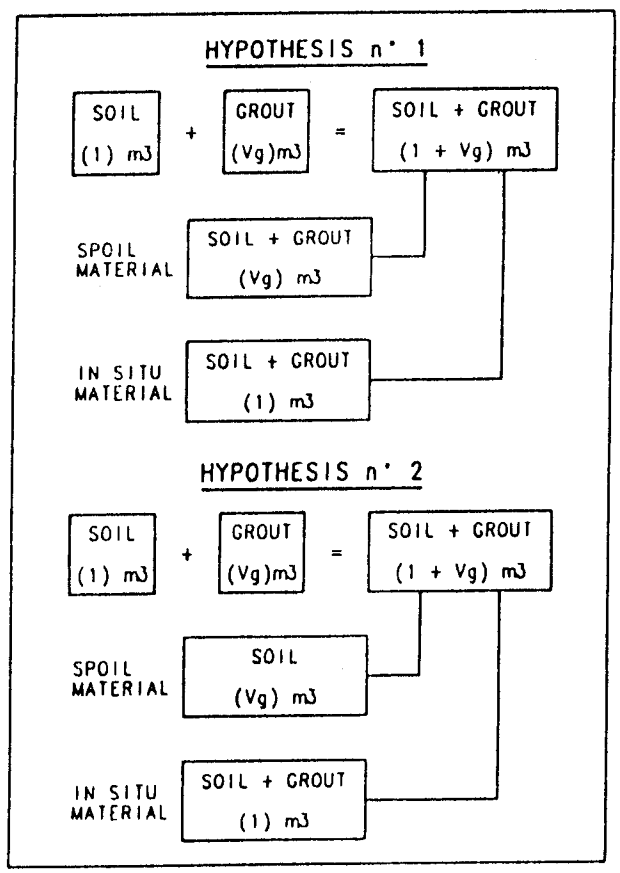


Figure 116. Hypotheses assumed in soil-cement mortar sample preparation (Pagliacci and Pagotto, 1994).

Table 17. Main characteristics of soil-cement mortar samples
(Pagliacci and Pagotto, 1994).

MORTAR TYPE	CEMENT PER C.M. OF SOIL	C/W	SAND PERCENTAGE
(-)	(kN/m3)	(-)	(%)
HYPOTHESIS n° 1			
A	1.68	0.295	36.6
B	1.86	0.313	22.4
C	1.95	0.318	12.6
D	2.04	0.323	0.0
HYPOTHESIS n° 2			
A	2.50	0.465	52.9
B	2.50	0.430	30.6
C	2.50	0.413	16.8
D	2.50	0.396	0.0

Table 18. Unconfined compressive strength values for different mortar type, mix hypothesis, and curing time (Pagliacci and Pagotto, 1994).

MORTAR TYPE	HYPOTHESIS n° 1		HYPOTHESIS n° 2	
	U.C.S. 7 DAYS	U.C.S. 28 DAYS	U.C.S. 7 DAYS	U.C.S. 28 DAYS
(-)	(MPa)	(MPa)	(MPa)	(MPa)
A	0.23	0.60	0.65	1.45
B	0.22	0.54	0.42	1.20
C	0.22	0.49	0.34	0.99
D	0.24	0.60	0.37	0.86

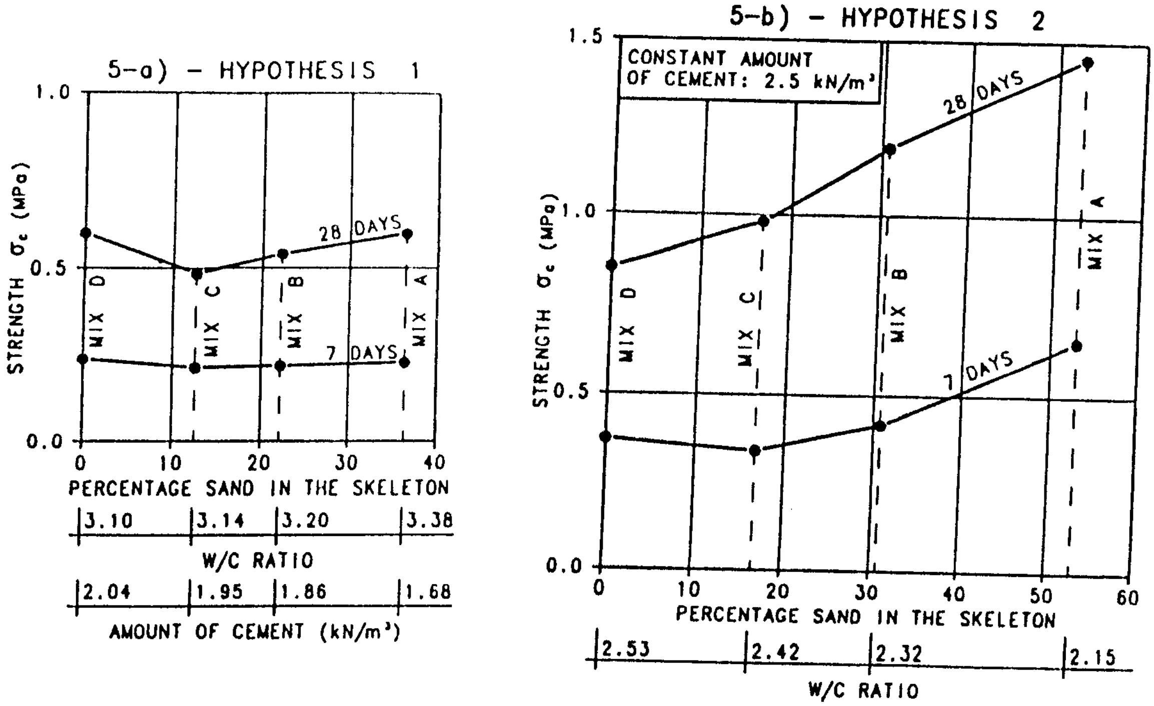


Figure 117. Unconfined compressive test results on soil-cement mortar samples (Pagliacci and Pagotto, 1994).

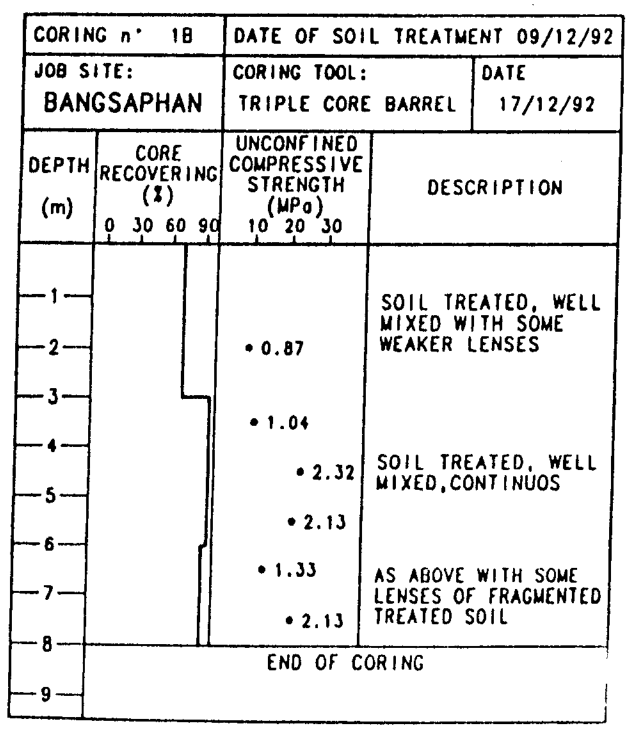


Figure 118. Results from coring of an improved column and laboratory testing on recovered samples (Pagliacci and Pagotto, 1994).

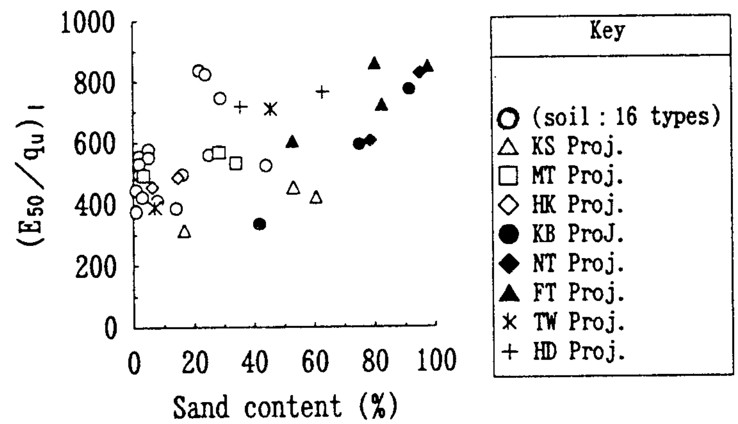


Figure 119. Relationship between E_{50}/q_u and sand content of laboratory improved soil (Saitoh et al., 1996).

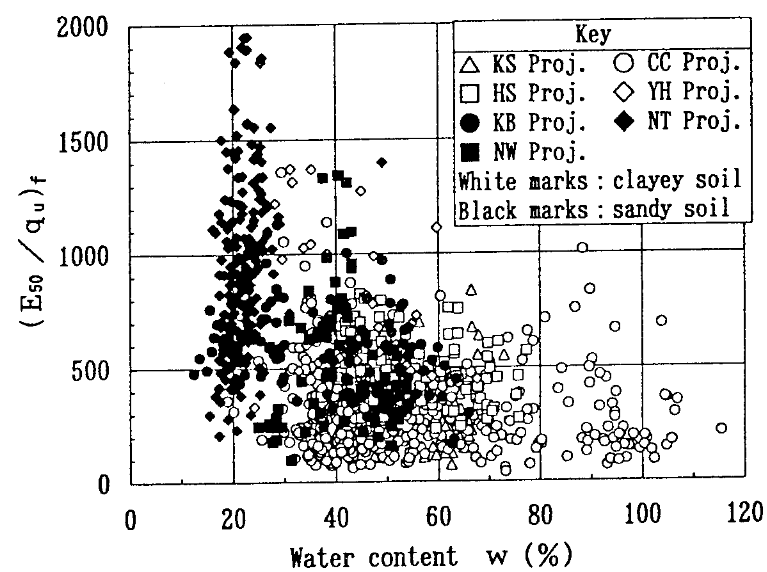


Figure 120. Relationship between water content and E_{50}/q_u of in situ improved soil (Saitoh et al., 1996).

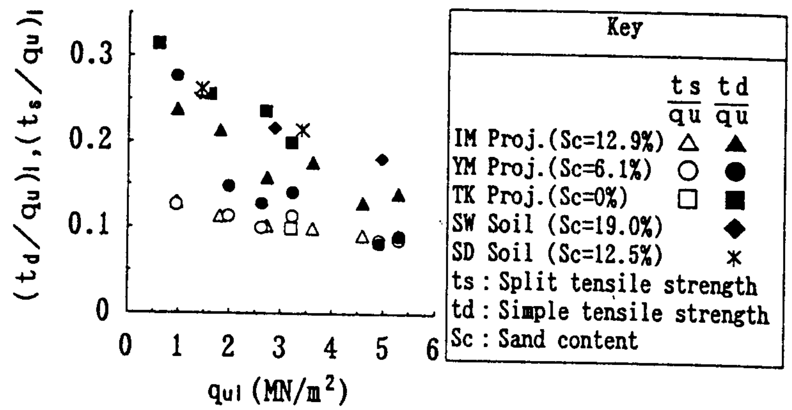


Figure 121. Relationships between q_u and t_d/q_u, and q_u and t_a/q_u for laboratory improved soil (Saitoh et al., 1996).

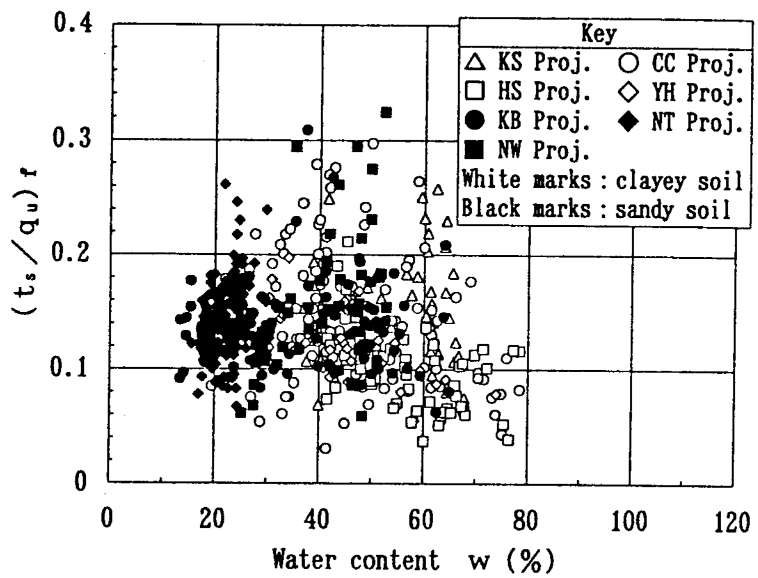


Figure 122. Relationship between water content and t_s/q_u of in situ improved soil (Saitoh et al., 1996).

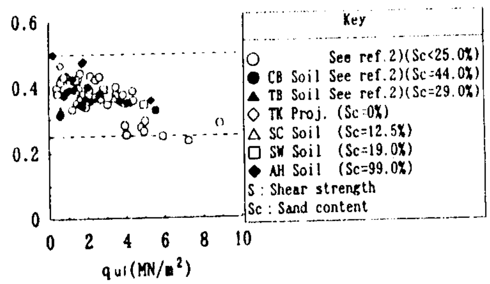


Figure 123. Relationship between q_u and s/q_u of laboratory improved soil (Saitoh et al., 1996).

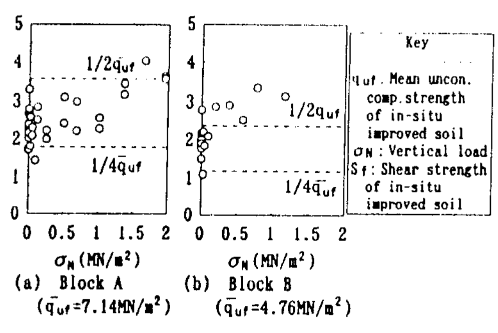


Figure 124. Examples of relationship between s and σ_n of in situ improved soil (Saitoh et al., 1996).

Referring to the relationship between laboratory sample U.C.S. (q_{ul}) and field U.C.S. (q_{uf}), the authors wrote:

"The target unconfined compressive strength ${}_{T}q_{ul}$ of laboratory improved soil is expressed…as follows:

$$ {}_{T}q_{ul} = \frac{F \cdot {}_{a}q_{u}}{\gamma} \cdot \frac{1}{\lambda} \; or \; \frac{F \cdot {}_{a}q_{u}}{\gamma} \cdot \frac{1}{\beta} \cdot \frac{1}{\gamma} $$

where $\lambda = q_{uf}/q_{ul}$.

λ is a coefficient allowing for mixing equipment, construction conditions, ground conditions, cement type and amount of cement slurry. The data obtained on λ through the author's on-land work projects is shown in Figure 125. λ is in the range of 0.5 to 2.5 for on-land work projects with the λ value for sandy ground remarkably large…(The λ factor for sea work projects is reported to be approximately 1.0). The target unconfined compressive strength for laboratory mixed soil in the mix proportion test is given by substituting F, ${}_{a}q_{u}$, γ, β and λ given by engineers into the above equation."

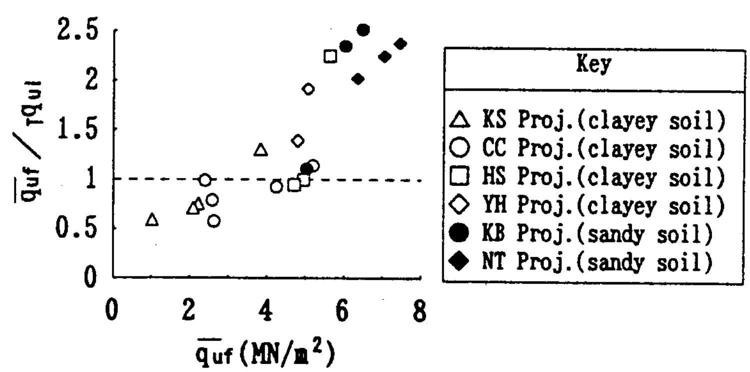


Figure 125. Data on coefficient λ (Saitoh et al., 1996).

They then experimented in the laboratory with portland cement (OPC) and Type B portland blast furnace slag cement (PSC). Figures 126 and 127 show the impact of cement type and factor, and initial water content, calculated as:

$$W\,(\%) = w_o + (1/\rho_s + w_o / 100 \cdot \rho_w) \times (w/c) \times \alpha_c \times 100$$

Where	w	=	Natural water content (%)
	ρ_s	=	Density of soil particles (g/cm^3)
	ρ_w	=	Density of water (g/cm^3)
	w/c	=	Water/cement ratio
	α_c	=	Cement factor (kg/m^3).

They regard humus content as a most important controlling factor on strength. When the color of the soil is black (and so is considered to contain large amounts of humus) there is a possibility that improving the soil with OPC and Type B PSC will be extremely difficult. Tests of such parameters as pH, ignition loss, organic matter content, and humus content are used to determine this possibility. Figure 128 compares the sensitivity of these factors. Humus content appears to be the most reliable test, and soil with a humus content of greater than 0.8% cannot be economically improved with OPC or PSC. Soils with greater than 6% of organic matter, or having a pH less than 5, also proved very difficult to treat.

They also demonstrated a 28-day prediction relationship based on 1-day laboratory strengths (Figure 129). In the field, they take core samples after 5 to 6 days to permit 7-day testing. This is used to predict 28-day strength (Figure 130). The in situ treated ground is cured at a higher temperature than laboratory samples, and so the gain of strength is more rapid. The impact of water content, especially on the slag cement mixes, is also well illustrated.

Chen et al. (1996) reported on the results of laboratory tests on "several hundred" soft clay samples from different locations in Shanghai. These clays typically have 45 to 50% moisture content, Plasticity Indices of approximately 20%, and c_u = 10 to 30 kPa. Using the equivalent of Type 2 portland cement, at cement factors of 160 to 260 kg/m^3, and a

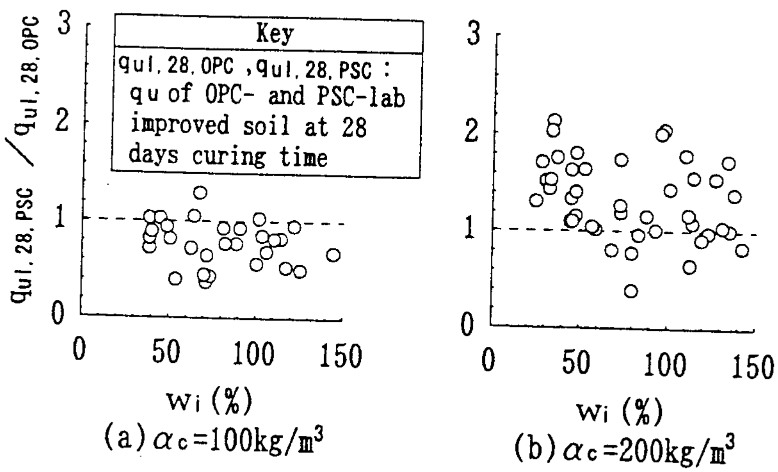


Figure 126. Relationship between initial water content, w_i, and $q_{ul.28.PSC}/q_{ul\,28.OPC}$ for improved soil (soil: 17-28 types) (Saitoh et al., 1996).

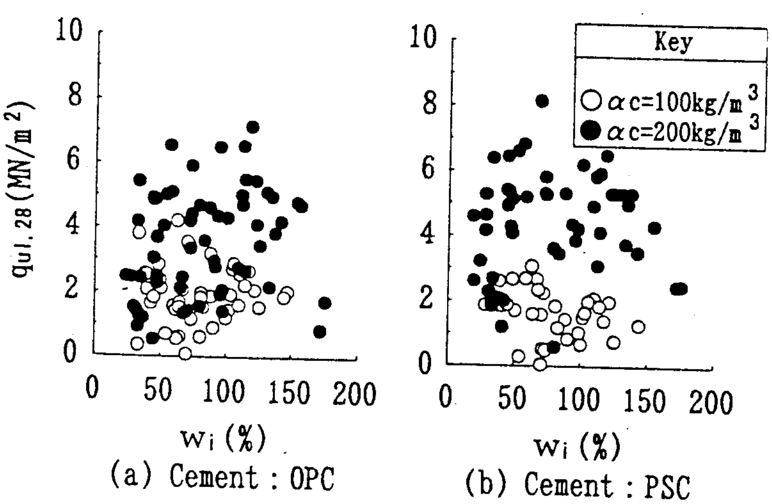


Figure 127. Relationship between initial water content, w_i, and $q_{ul.28}$ of improved soil (soil: 32-41 types) (Saitoh et al., 1996).

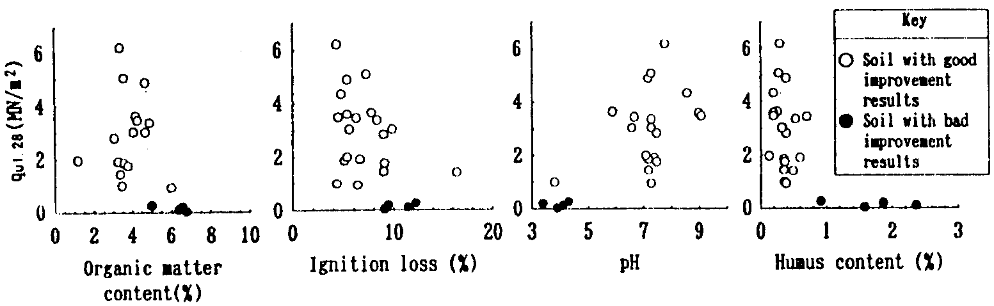


Figure 128. Relationship between chemical properties of soil and unconfined compressive strength (cement factor: 150 kg/m^3) for laboratory improved soil (soil: 23 types) (Saitoh et al., 1996).

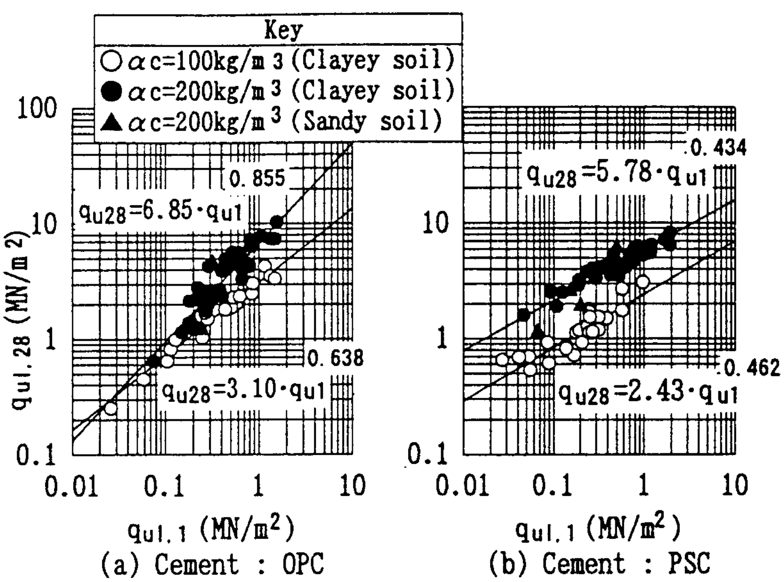


Figure 129. Relationship between q_{u1} and q_{u28} for laboratory improved soil (soil: 22 types) (Saitoh et al., 1996).

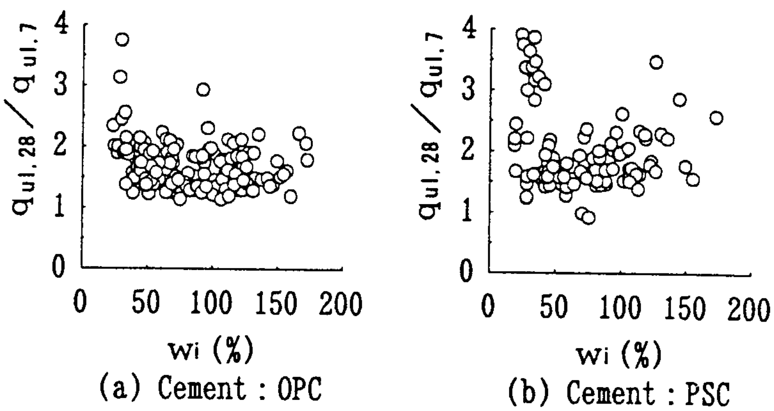


Figure 130. Relationship between initial water content, w_i, and q_{u28}/q_{u7} for laboratory improved soil (soil: 39-21 types) (Saitoh et al., 1996).

water/cement ratio of 0.5, the data in Figure 131 were generated. Admixtures were added to improve early strength (more than 0.8 MPa in 28 days). Reported permeabilities were 10^{-8} to 10^{-9} m/s.

Gotoh (1996) reported on a series of laboratory tests on samples of alluvial clayey and sandy soils, volcanic ash cohesive soils, and volcanic ash cohesive soils mixed artificially with mountain sand, obtained from two Japanese project sites. Four parameters, namely pH value (pH), ignition loss (L_i), natural water content (w_n), and fines content (F_c), were predicted to be the main factors that could affect the strength of cement-treated soils. The influence of actual cement content on the treated soils considering the effect of the parameters pH, L_i, w_n, and F_c was defined in terms of modified cement content (F), and 45 different relationships were obtained correlating the F-value and the soil strength. The results revealed that the relations between the strength of the improved soils and F-value based on the combination of pH and L_i values were very good. The relationships based on pH value only were not as good as for the volcanic ash cohesive soils, and the relationships based on w_n only, or F_c only, were found to be very poor.

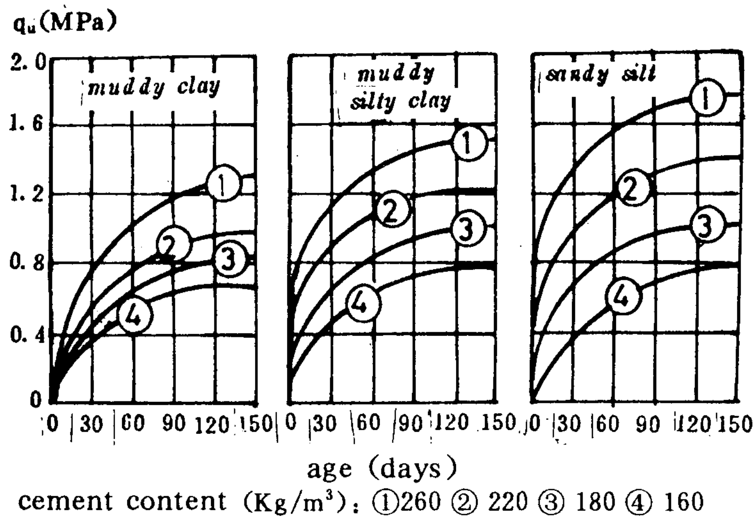


Figure 131. Strength of treated soil (Chen et al., 1996).

Asano et al. (1996) reported that the Japanese industry is looking for ways to use surplus coal ash, and so they began experiments with ash (lagoon), gypsum, and cement to produce the so-called FGC deep mixing material used in a modified CDM technique. This slurry produces only low strengths (less than 0.5 MPa) even with high volume ratios (25 to 40%), but (due to pozzolanic reaction) provides higher than expected long-term strengths. The addition of gypsum in "little" amounts to flyash and cement was found to increase strength without generating the deleterious effects noted in Chapter 2.

Laboratory data for the silty clay soil and grouts are shown in Tables 19 through 22. The optimum slurry mix (for strength) was Flyash 10: Gypsum 1: Cement x (Figure 132). Cement factor x clearly controlled strength (Figure 133) of treated soil. Figure 134 shows 40 to 60% continuing strength increase for normal CDM grouts from 28 to 730 days (presumably due to the pozzolanic reactions involving the flyash and gypsum components), but no increase after 28 days for FGC and low strength CDM (70 kg/m^3 cement factor). Figure 135 shows the strong inverse relationship between water/cement ratio and strength of treated ground (total water).

A field test was then conducted and cored, and the authors found (Figure 136) that the U.C.S. of the treated soil was proportional to its calcium content. Figure 137 shows a comparison of U.C.S. from laboratory and field tests. This ratio was about 20 to 50 percent for FGC and cement slurry, – "well within the range of past CDM executions," but about 75% for CDM itself, and 33 to 75% for "cement mortar." It appeared to decrease with increasing laboratory strengths. Figure 138 compares E-values, being generally 50 to 300 times U.C.S. for FGC; 140 to 500 times U.C.S. for normal CDM.

Dong et al. (1996) described a laboratory DM test "to clarify the effects of several factors, the shape of mixing blade, revolution speed, velocity of penetration-withdraw of shaft, etc. on the degree of mixing and strength properties of cement treated soil column…" In particular the degree of mixing and the unconfined compressive strength of the treated soil were addressed. The mixing apparatus is detailed in Table 23, the composition of the treated soil in Table 24, and the grout mix in Table 25. Various types of counter-rotating mixing blades are shown in Figure 139. A prepared clay-sand of U.C.S. = 29 kPa was produced and then treated by the 400-mm-diameter counter-rotating augers to a depth of 1 m. Test variables were as in Table 26. The general conclusions were

- The efficiency of mixing improves with higher rotary speeds.
- The U.C.S. improves with rotary speed (Figure 140), and was higher for thinner blades.

Table 19. Physical properties of the soil to be improved (Asano et al., 1996).

Specific gravity			2.64
Texture	Clay	(%)	58.0
	Silt	(%)	35.2
	Sand	(%)	6.8
Moisture content		(%)	74.88
Wet density		(t/m^3)	1.563
Liquid limit		(%)	89.6
Plastic limit		(%)	37.8
Uniconfined compressive strength		(kgf/cm^2)	0.51
PH			8.0
Organic content			4.5

Table 20. Physical properties of the materials (Asano et al., 1996).

Material	Specific Gravity	Specific Surface	Average Grain Size D50	SiO2	Al2O3	Fe2O3	CaO	SO3	Ig. loss
		m^2/g	μm	%	%	%	%	%	%
Cement	3.16	352.0	-	-	-	-	-	2.3	0.7
Fly ash	2.18	2420	31.3	66	23.6	2.4	4.3	0.2	2.3
Gypsum	2.33	920	38.4	0.2	0.3	0.1	33.0	43	7.7

Table 21. Blends used for laboratory and in situ tests (Asano et al., 1996).

Stabilizer	FGC			FGC			CDM		
Water-stabilizer ratio	1.0			1.0			1.0		
FGC content	10:1:2			10:1:6			0:0:1		
Cement content kg/m^3	20	40	70	70	110	140	70	110	140

Table 22. Blends used for slurry pressure feeding test (Asano et al., 1996).

F : G : C	W/FGC 0.5	0.6	0.8
10:1:6 ($C=70kg/m^3$)	O	O	O
10:1:4 ($C=70kg/m^3$)	-	O	O
10:1:3 ($C=40kg/m^3$)	O	O	O
10:1:2 ($C=40kg/m^3$)	-	O	O

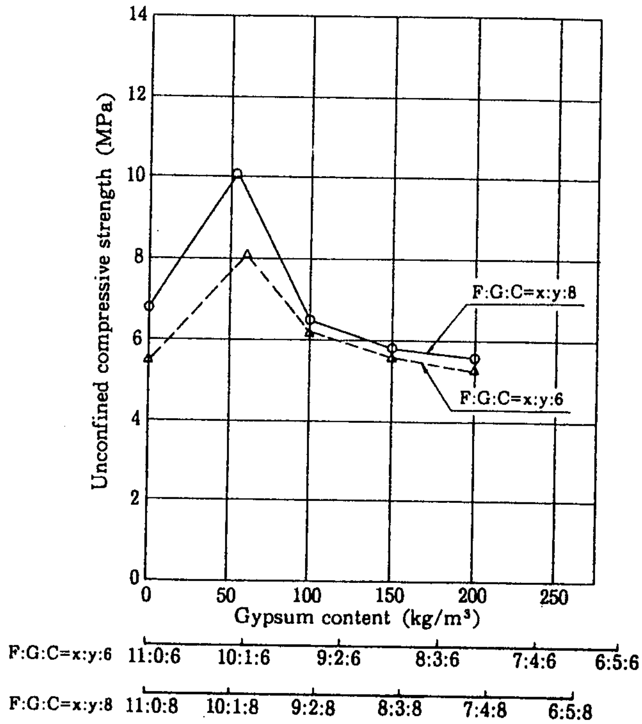


Figure 132. Results of the study on the optimum blend of gypsum (Asano et al., 1996).

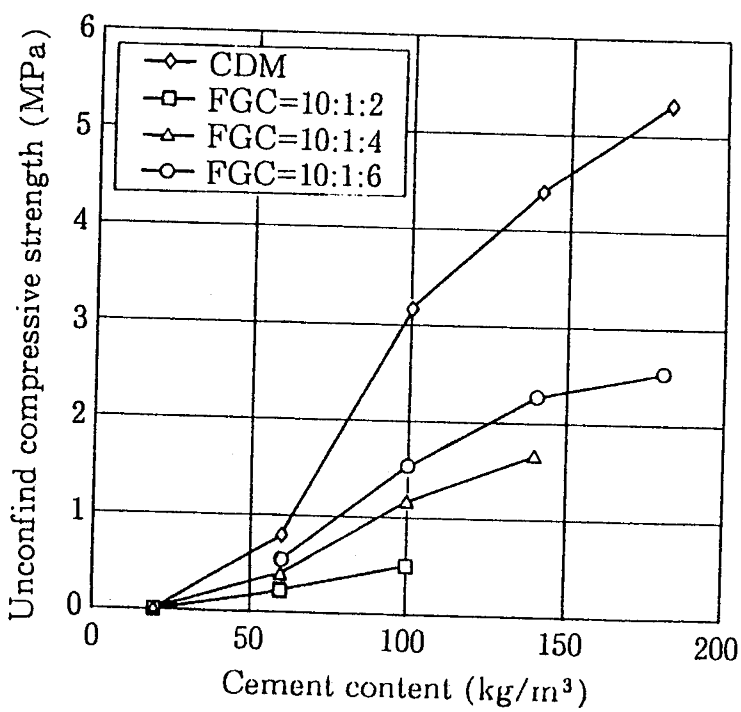


Figure 133. Relationship between cement content and unconfined compressive strength (Asano et al., 1996).

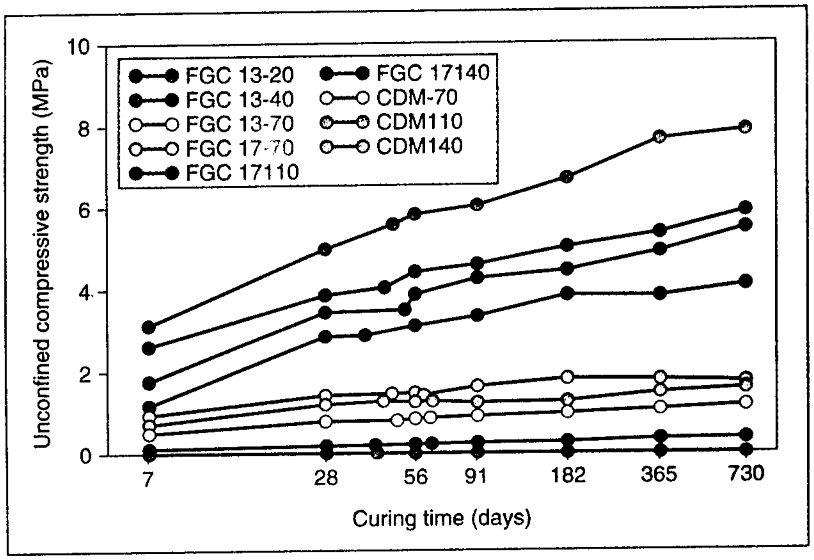


Figure 134. Relationship between curing time and unconfined compressive strength (Asano et al., 1996).

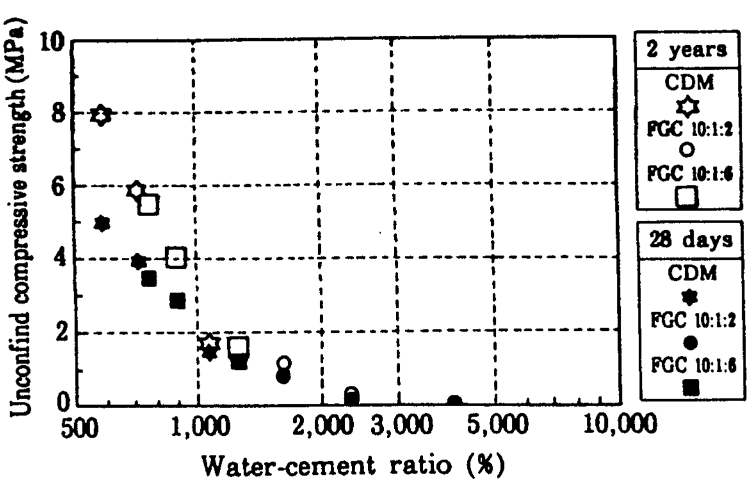


Figure 135. Relationship between water-cement ratio and unconfined compressive strength (Asano et al., 1996).

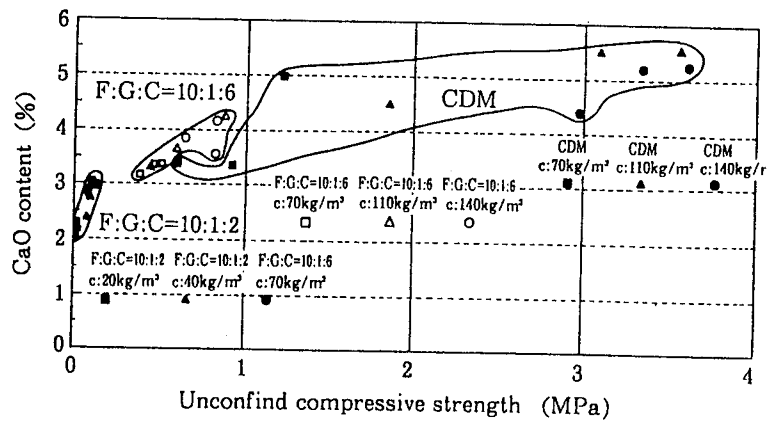


Figure 136. Relationship between unconfined compressive strength and calcium content (Asano et al., 1996).

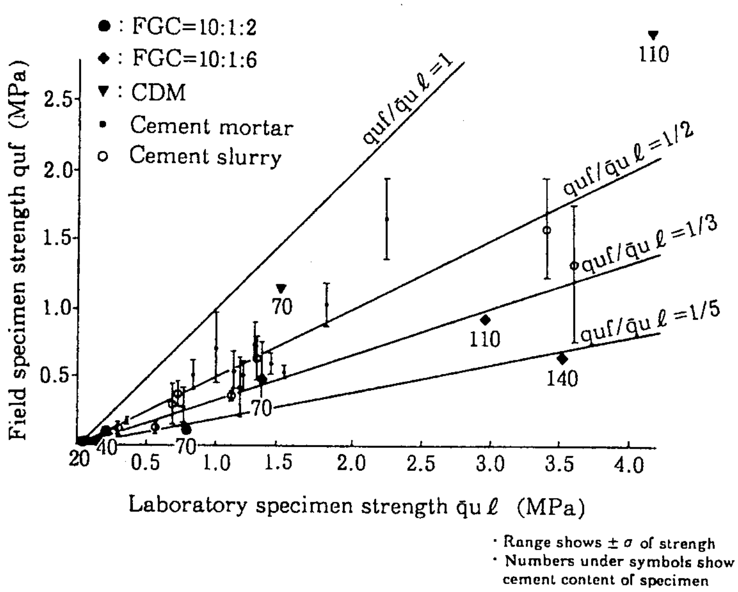


Figure 137. Relationship between strength of laboratory test specimens and strength of in situ test specimens (Asano et al., 1996).

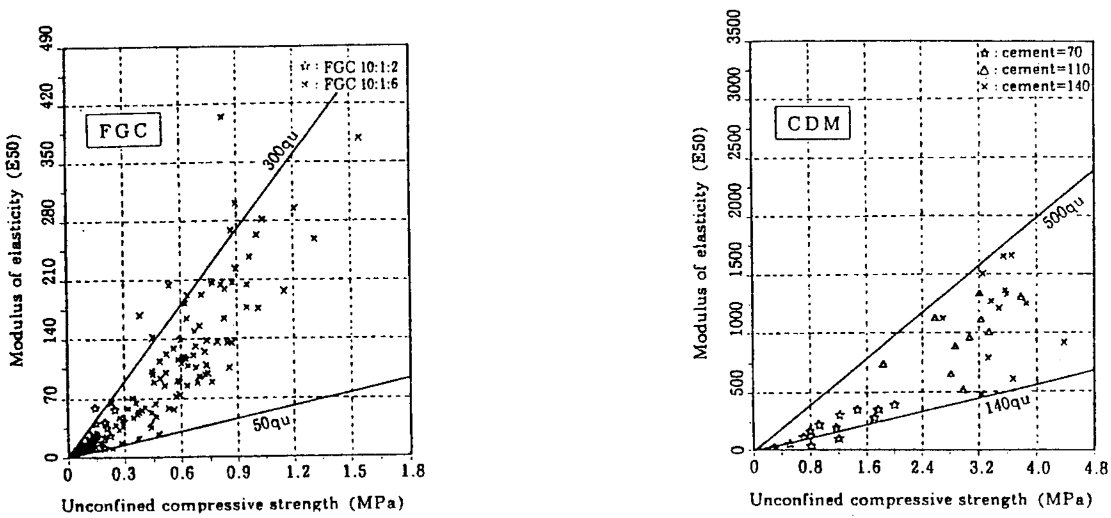


Figure 138. Relationship between unconfined compressive strength and modulus of elasticity, E_{50} (Asano et al., 1996).

Table 23. Main specifications of the mixing apparatus (Dong et al., 1996).

	Specifications
Mixing out put	7.5KW
Rotary speed	5～100rpm
Diameter of mixing	ϕ400～700mm
Lifting output	1.2KW
Lifting speed	1.25～12.5m/min
Mixing container	Diameter10,00mm, Height1,100mm
Mixing blade	4 kinds, ϕ400mm in diameter

Table 24. Mixture ratio of ground material (kg/m^3) (Dong et al., 1996).

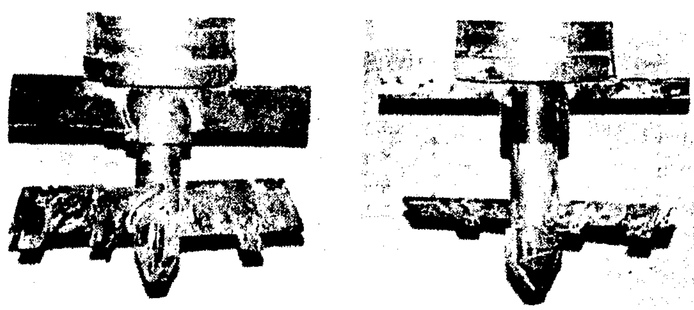
Table 2 Mixture ratio of ground material (kg/m^3)

Clay sand	Quick-hardening cement	Water
1,260	32	505

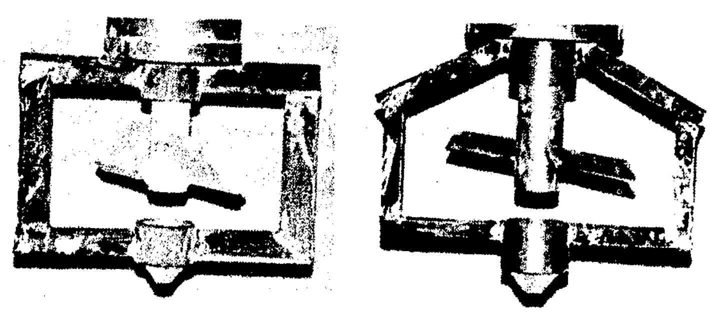
Table 25. Mixture ratio of slurry cement (kg/m^3) (Dong et al., 1996).

Table 3 Mixture ratio of slurry cement (kg/m^3)

W/C(%)	Quick-hardening	Water
80	44.704	35.763



(a) TypeA-1(thick blade) (b) TypeA-2(thin blade)



(c) TypeB-1(thick blade) (d) TypeB-2(thin blade)

Figure 139. Types of mixing blades (Dong et al., 1996).

Table 26. Test conditions (Dong et al., 1996).

Table 4 Test conditions

Test No.	Shape of Mixing blade	Rotary speed (rpm)	Lifting speed (m/min)	Total number of blade revolution (cycles/min)	Injecting velocity of slurry (l/min)	Injecting volume of slurry (l/one column)
1	TypeA-1	30	1.0	60	28	28
2	TypeA-2	30	1.0	60	28	28
3	TypeA-2	15	0.5	60	14	28
4	TypeA-2	30	0.5	120	14	28
5	TypeA-2	45	0.5	180	14	28
6	TypeA-2	60	0.5	240	14	28
7	TypeB-1	30	1.0	90	28	28
8	TypeB-1	30	0.5	180	14	28
9	TypeB-1	60	1.0	180	14	28
10	TypeB-2	15	0.5	90	14	28
11	TypeB-2	30	0.5	180	14	28
12	TypeB-2	45	0.5	270	14	28
13	TypeB-2	60	0.5	360	14	28

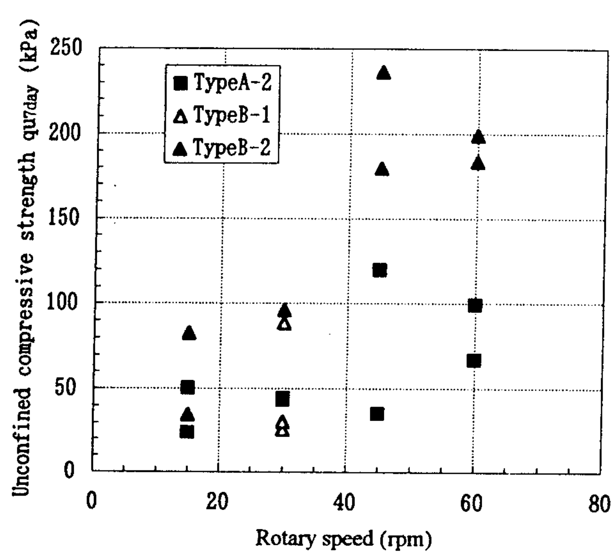


Figure 140. Relationship between rotary speed and improved strength (Dong et al., 1996).

- The U.C.S. improves with the number of blade revolutions per given length of penetration (Figure 141) (and so is inversely proportional to penetration rate).
- 7-day tests showed tensile strengths to average 29% U.C.S.
- The Type B-2 blade (Figure 139) seemed to be the most efficient.

Master Builders Technologies (1998) found that by using their clay dispersant, U.C.S. could be doubled, or twice the amount of clay could be treated using only 60% of the initial cement factor.

During 1998 and 1999, Al-Tabbaa and various coworkers at Cambridge University, England, reported on a long series of laboratory and field tests, conducted jointly with a specialty contractor, May Guerney, Ltd. (Al-Tabbaa and Evans, 1998; Al-Tabbaa et al., 1998; Al-Tabbaa and Evans, 1999a and 1999b). The initial "treatability study" was reported by Al-Tabbaa and Evans (1998). Site soils (Figure 142) were mixed in the laboratory with a variety of grout mixes (Table 27). The grout w/c ratio (0.42) and volume ratio (20%) were constant.

Regarding U.C.S. and dry density data, data are shown in Table 28 (28 days). Regarding durability, both wet-dry and freeze-thaw tests were conducted at 28 days (Table 29). All mixes survived all 12 cycles of wet-dry (maximum loss of 3.0%) but all failed the freeze-thaw test (ASTM D4843 and D4842, 1988 and 1990, respectively). Permeabilities of 0.48×10^{-9} to 1.90×10^{-9} m/s were recorded, with coefficient of compressibility of 1.06×10^{-6} to 2.49×10^{-6} m^2/kN.

Based on these tests, three further mixes (Table 30) were developed to improve freeze-thaw behavior and reduce permeability. (Mixing is assumed to be manual.)

Al-Tabbaa (1999) reported on laboratory auger mixing tests in stratified sand. Uniform fine-medium sand, and coarse sand, were mixed at moisture contents of 10% and 30% (saturated). Data are provided in Figure 143. Strengths in coarse sand were appreciably higher, while all strengths increased with increasing cement:ash ratio and decreasing water:solids ratio.

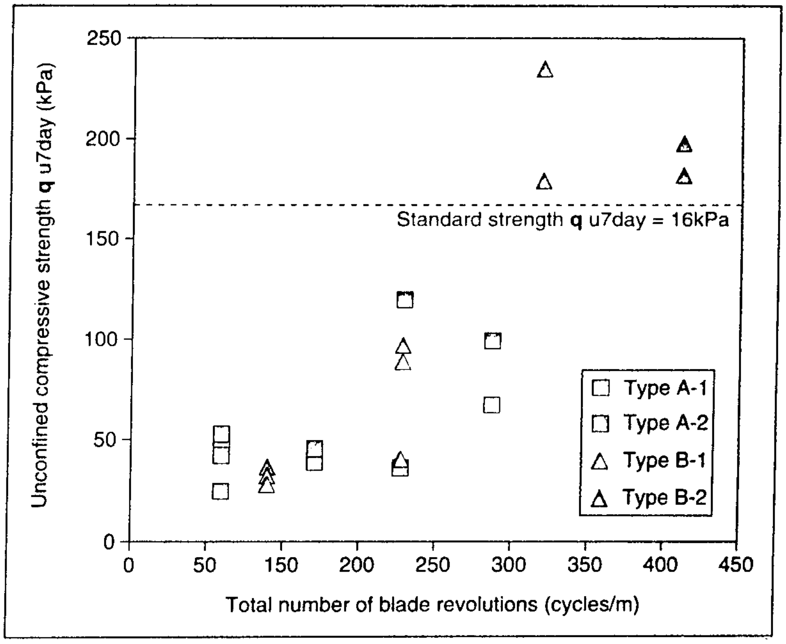


Figure 141. Relationship between unconfined compressive strength and blade revolutions (Dong et al., 1996).

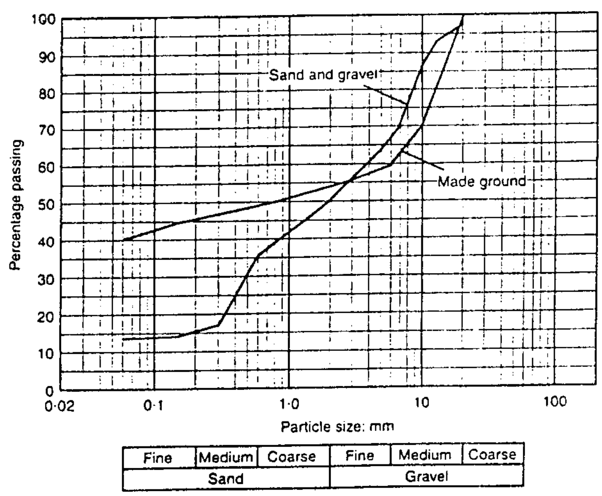


Figure 142. Particle size distribution of the two main soil types in the trial pit (Al-Tabbaa and Evans, 1998a).

Table 27. Details of selected mixes (Al-Tabbaa and Evans, 1999b).

Mix	Cement:pfa:lime	Water:dry grout	Soil:grout
A	2:8:0	0·42:1	5:1
B	3:8:0	0·42:1	5:1
C	2·5:8:0·4	0·42:1	5:1
D	3:8:0·1	0·42:1	5:1

pfa = pulverized fuel ash.

Table 28. U.C.S., dry density, and leachate pH values of soil-grout mixes (Al Tabbaa and Evans, 1999b).

	UCS: kPa		Dry Density: kg/m^3		Leachate pH	
Mix	Made ground	Sand and gravel	Made ground	Sand and gravel	Made ground	Sand and gravel
A	362	585	1584	1813	6·5	9·2
B	510	1042	1513	1816	7·7	9·3
C	418	495	1534	1824	9·0	10·5
D	502	815	1549	1844	8·6	9·6

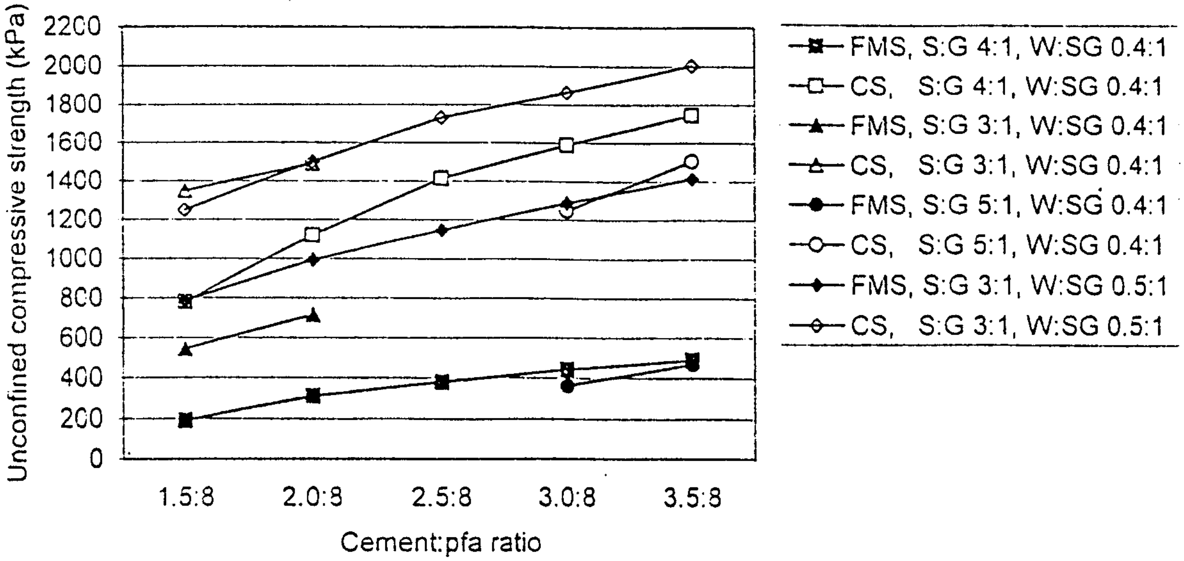
Table 29. Percentage cumulative mass loss in wet-dry durability tests on soil-grout mixes (Al Tabbaa and Evans, 1999b).

Mix	Made ground	Sand and gravel
A	3·9	0·4
B	0·0	0·1
C	1·0	0·1
D	1·2	0·2

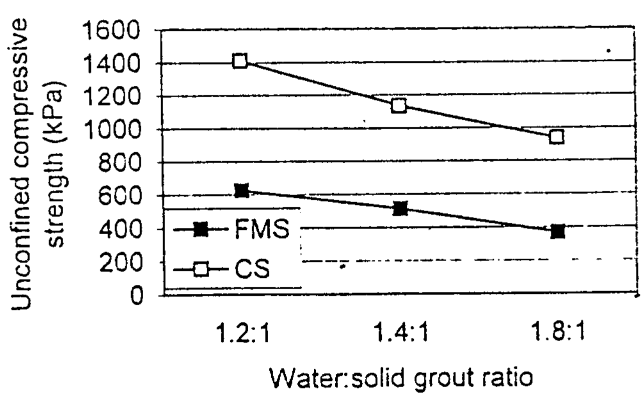
Table 30. Additional soil-grout mixes selected for the site trial (Al Tabbaa and Evans, 1999b).

Mix	Cement:pfa:lime:bentonite	Water:dry grout	Soil:grout
E	2·5 : 8 : 0·4 : 0	0·42 : 1	3·5 : 1
F	2·5 : 8 : 0·4 : 0	0·3 : 1	3·9 : 1
G	8 : 0 : 0 : 0·8	1·6 : 1	2·8 : 1

pfa = pulverized fuel ash.



(a)



(b)

Figure 143. 28-day U.C.S. of uncontaminated soil-grout mixes of a) cement-pfa grout, and b) cement-bentonite grout (Al Tabbaa, 1999).

4.4 Special Field Tests

Ryan and Jasperse (1989) and Taki and Yang (1991) reported on the use of SMW equipment to create "honeycombs" of treated soil columns, and a hydraulic cut-off, in fluviolacustrine and lacustrine alluvium and outwash sands and gravels under Jackson Lake Dam, WY (Figure 144). The project featured 0.91-m-diameter columns to 33 m depth. A minimum shear strength of 1.4 MPa was specified. A pre-construction test section, and an extensive quality control program were carried out, particularly related to the strength of the treated soil. Two thousand wet samples were made and 70 core holes were drilled to provide 150 samples for testing.

The major conclusions were:

- Samples continued to increase in strength for at least 112 days (Figure 101).
- Water/cement ratio is the key determining factor in controlling strength, even more than cement factor (Figure 94).
- Laboratory samples conservatively predicted field results from wet grabs (Figure 145) and wet grab samples generally have lower strengths than cores (Figure 146) (Ryan and Japsperse concluded this was "probably because of excellent in situ curing conditions..., a cool, moist environment").

The production slurry mix had a water/cement ratio of 1.25, and a cement factor of 300 kg/m^3 (down from 400 kg/m^3 at a water/cement ratio of 1.35).

Shear strength was determined from triaxial and direct shear tests to be about 33% U.C.S. Taki and Yang (1991) showed the data against the SMW international "database" assembled to that point. This would appear to show an average wet grab strength of 3 to 4 MPa and a core strength of 5 MPa (Figure 147).

Taki and Yang (1991) also described the SMW treatment conducted at the EBMUD Wet Weather Storage Basin Project, Oakland, CA. A 12- to 14-m deep excavation was required through soft to medium highly plastic silty clay (San Francisco Bay Mud) (Figure 148), and a

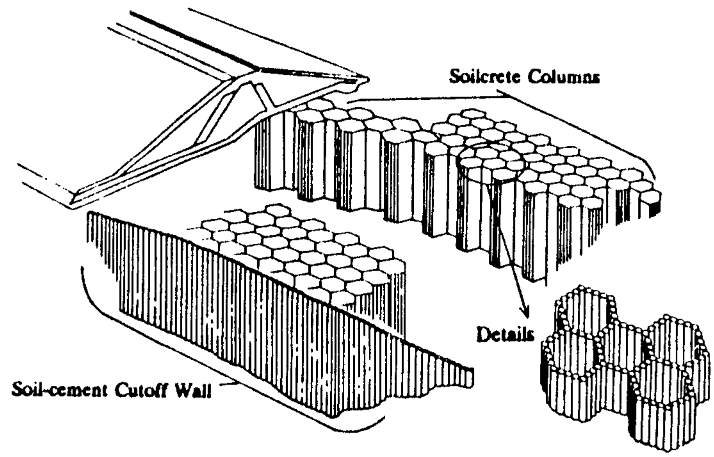


Figure 144. SMW work for Jackson Lake Dam Modification Project, WY (Taki and Yang, 1991).

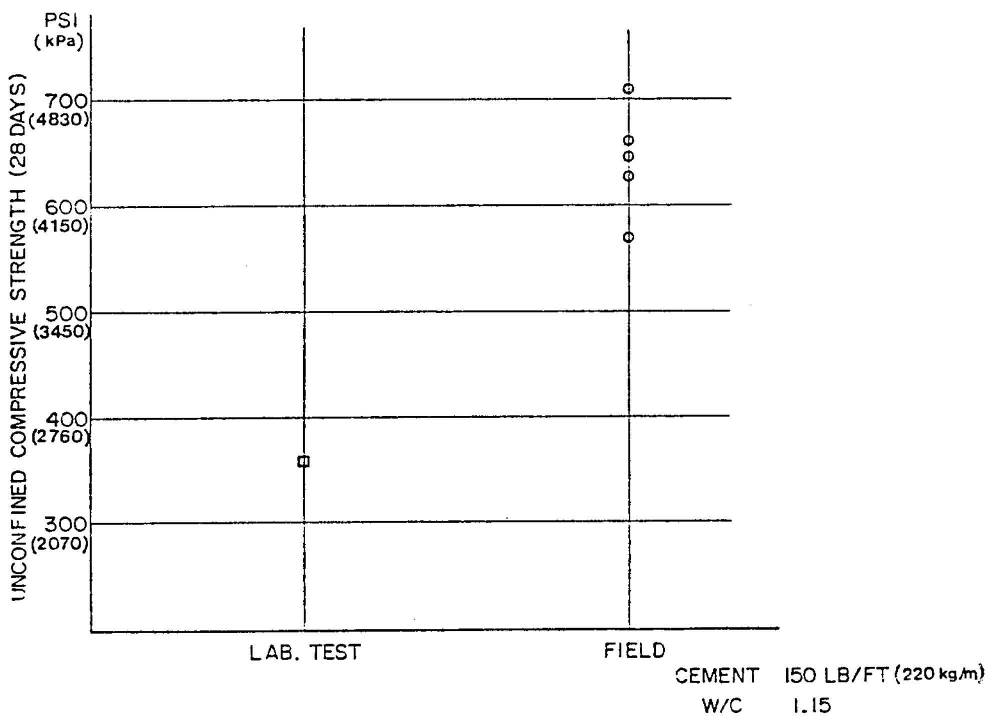


Figure 145. Laboratory strength vs. field strength of wet samples (Ryan and Jasperse, 1989).

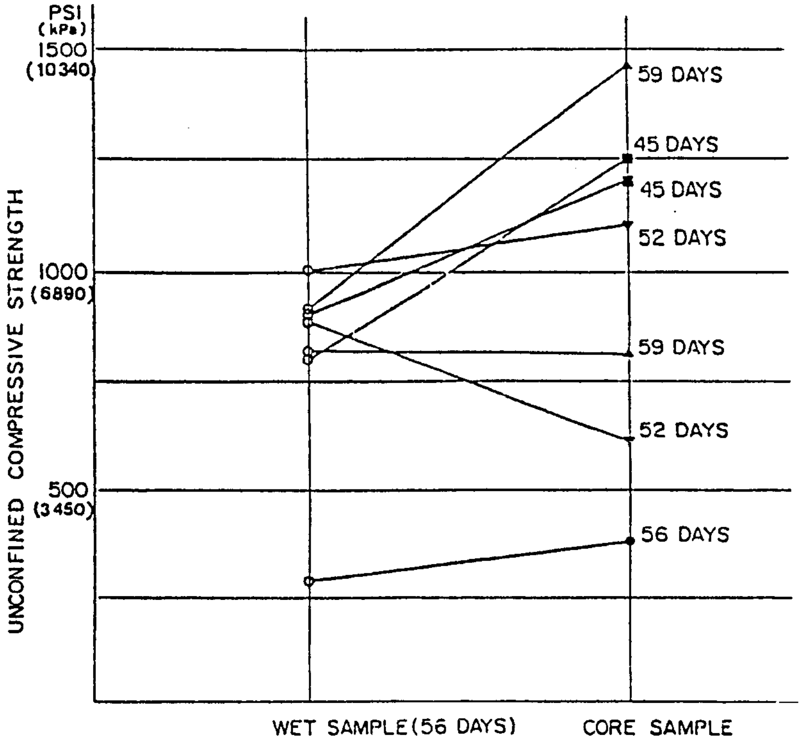


Figure 146. Wet sample strength vs. core sample strength (Ryan and Jasperse, 1989).

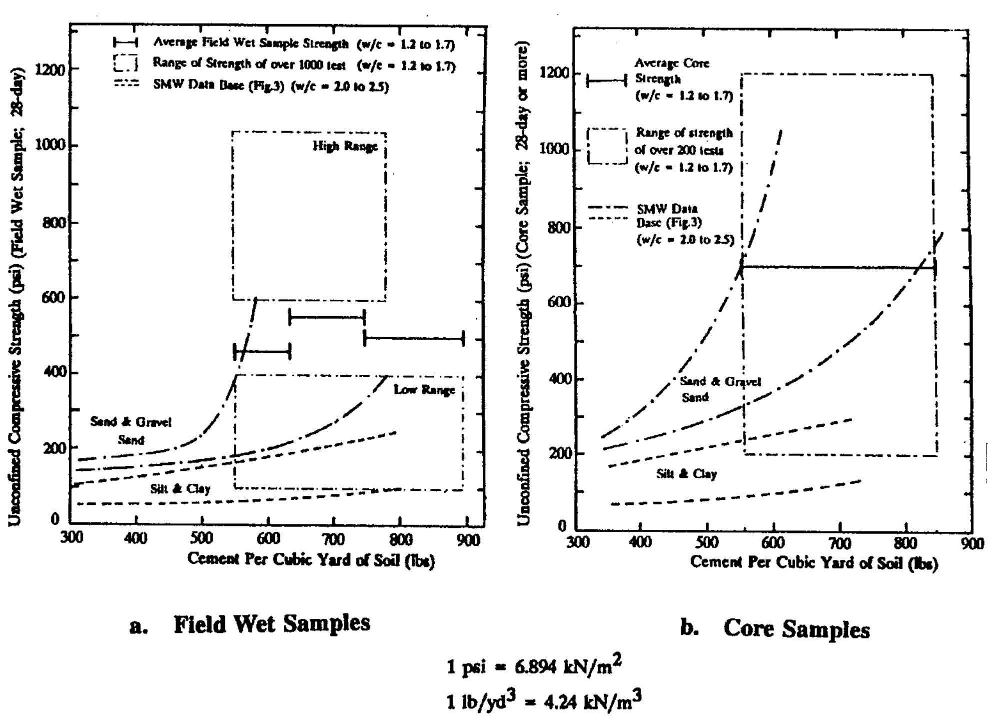


Figure 147. Unconfined compressive strength of SMW treated ground at Jackson Lake Dam, WY (Taki and Yang, 1991).

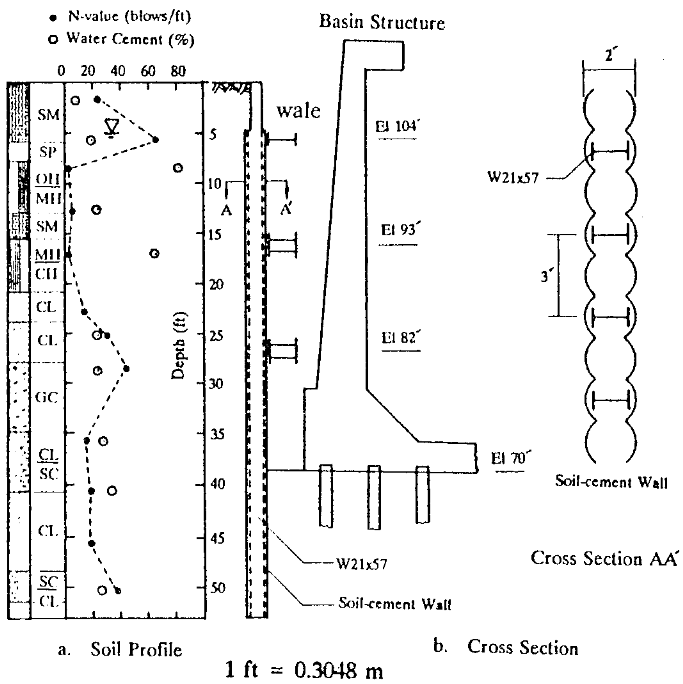
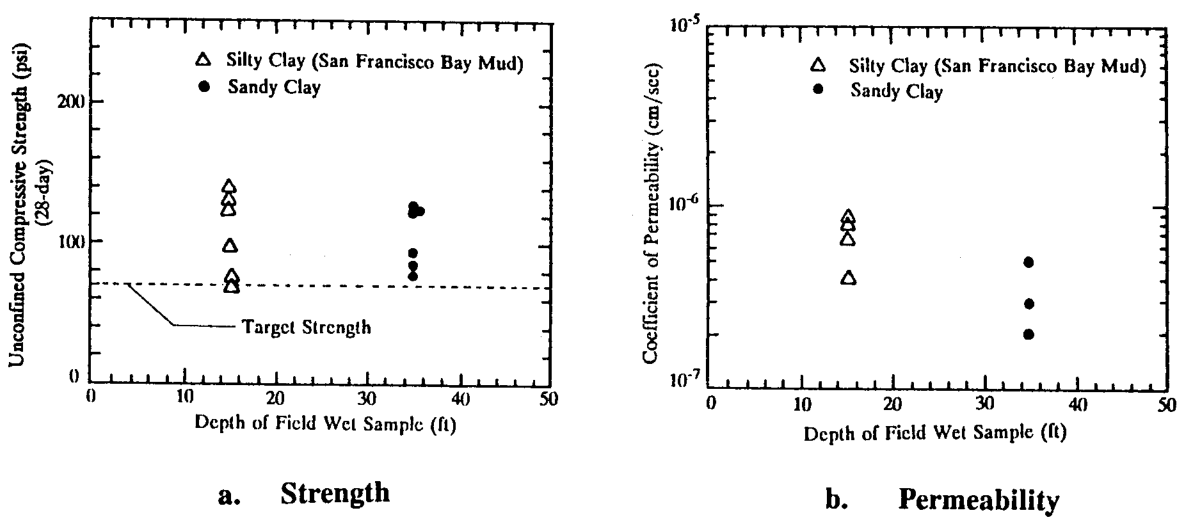


Figure 148. SMW excavation support wall for EBMUD project, Oakland, CA (Taki and Yang, 1991).

relatively impermeable earth retention system was needed. A cement factor of 290 kg/m^3 (plus 5 kg bentonite) was used at a water/cement ratio of 2.3 to provide the target U.C.S. of 0.5 MPa. Wet grab samples provided the strength and permeability data of Figure 149.

Yang and Takeshima (1994) described SMW treatment through glacial deposits comprising recessional outwash, lacustrine deposits, and lodgment till (Figure 150) at Lake Cushman Dam, WA. The outwash was mainly dense to very dense fine to coarse sand, and the lacustrines were stiff to very stiff clayey silt, silt, and medium dense to very dense sand. The till was similarly very dense with a permeability ranging from 10^{-4} to 10^{-7} m/s. A total length of 116 m of 0.61-m-wide wall to a depth of 43 m was created. Initial tests on treated soil gave U.C.S. = 0.7 to 1.2 MPa and based on these data, mixes with cement factor 350 to 550 kg/m^3 were selected. This provided treated soil U.C.S. values of 0.6 to 4.8 MPa and permeabilities of 2 x 10^{-7} to 6 x 10^{-9} m/s.



1 psi = 6.894 kN/m^2

Figure 149. Engineering properties of SMW treated soil for EBMUD project, Oakland, CA (Taki and Yang, 1991).

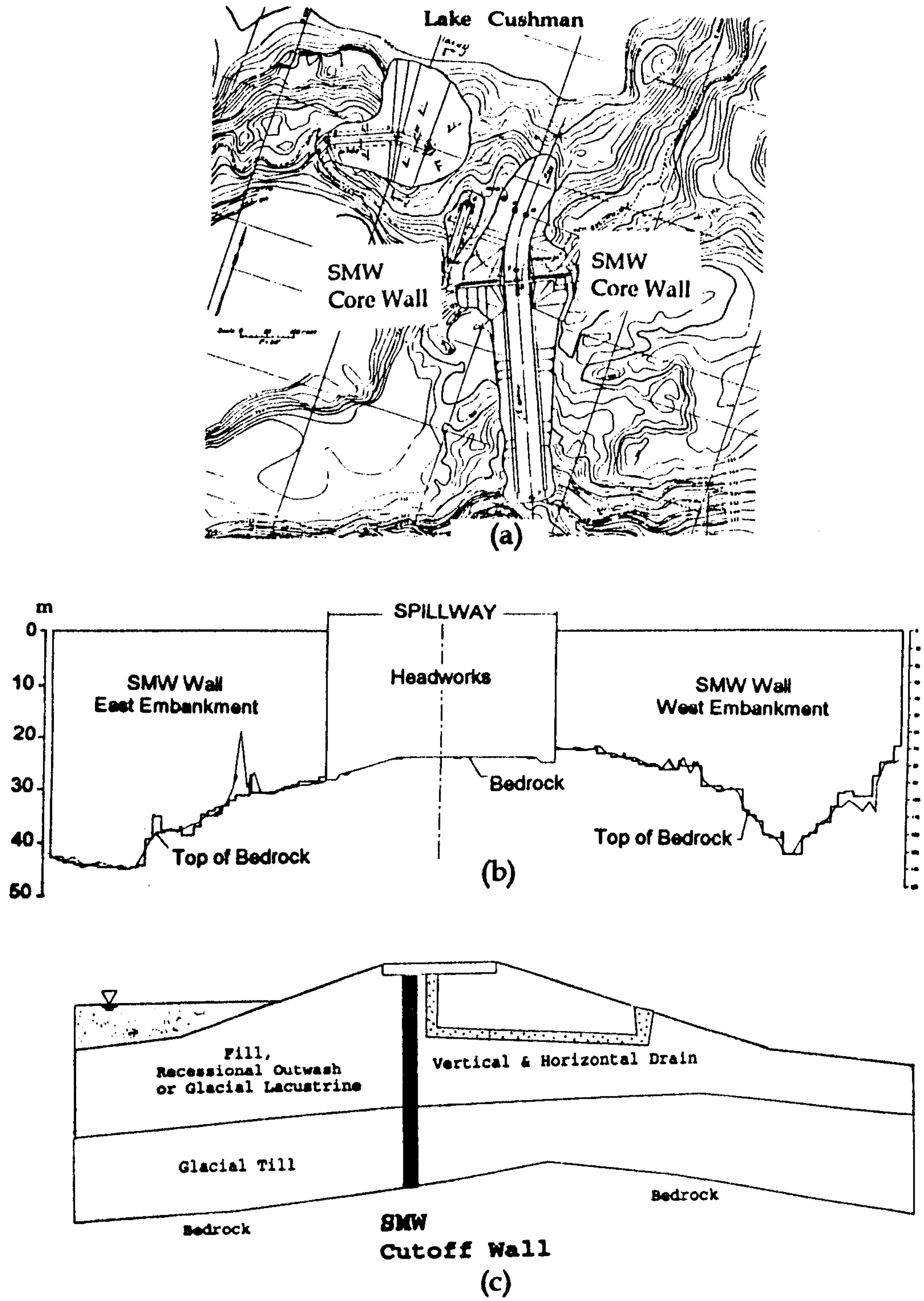


Figure 150. Cushman Spillway Project, WA a) site plan, b) cut-off wall profile, and c) simplified general cross section of embankment (Yang and Takeshima, 1994).

Futaki et al. (1996) installed 1-m diameter test columns at two different sites (Figure 151) exhumed them, and tested the full size columns with a prototype unconfined compression test. Twenty-five cores were also tested for comparison. The full-scale columns were trimmed to 2 m length and gave peak strengths of up to 2.3 MPa (Area A) and 4 MPa in Area B. Both exhibited brittle failure characteristics. They also found that the strength decreased as specimen size increased. Figure 152 shows the variation of U.C.S. with depth, and Figure 153 shows the derived relationship of U.C.S. and E_{50} (144 to 209 times U.C.S.).

Walker (1994) described tests on DSM treated soil at Lockington Dam, OH. The DSM method was used to create a cut-off through 6 m of an existing dam core. The material consisted of medium-dense gravelly sand to sand gravel to 1.5 to 4.3 m depths overlying loose silty sands, sandy silts, and silty clays. The specifications called for a residual in situ permeability of 10^{-8} m/s and a cement factor in place of at least 6% (about 100 kg/m^3).

A laboratory program was established to determine an appropriate mix design (Table 31). Testing was based on ASTM 1633 for U.C.S., and ASTM 5084 for permeability (triaxial [flexible wall] permeameters).

Desiccation tests were also run on Mixes 1A and 4A from Table 31: over 2 weeks moisture contents reduced 4 times to 7 to 8% but without sample deterioration. Mix 1A was selected for production on the basis of its higher strength. Field tests on wet grab samples gave the results shown in Table 5.

Takenaka (1995) presented data (Figure 154) on the relationships between apparent cement factor, strength, age, and soil conditions (Table 32). These data refer to only the "normal" soils, and not those with high organic contents described in Chapter 2. Amid the great variability, it may be noted that the impact of raising the cement content may only be apparent beyond a 7-day cure period. Strength gain from pozzolanic reactions may be expected long after the 60 days shown in Figure 154. The variability in soil properties underlines the need to thoroughly investigate each site individually prior to predicting likely mixed soil parameters.

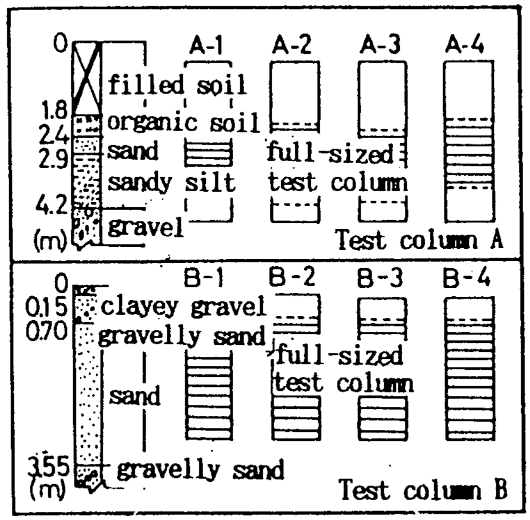


Figure 151. Sampling of full-sized columns (Futaki et al., 1996).

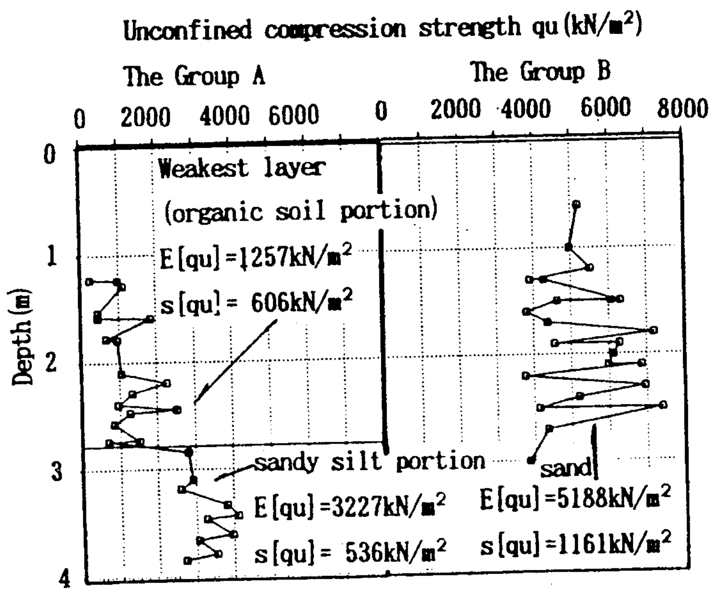


Figure 152. Vertical distribution of q_u within soil-cement column (Futaki et al., 1996).

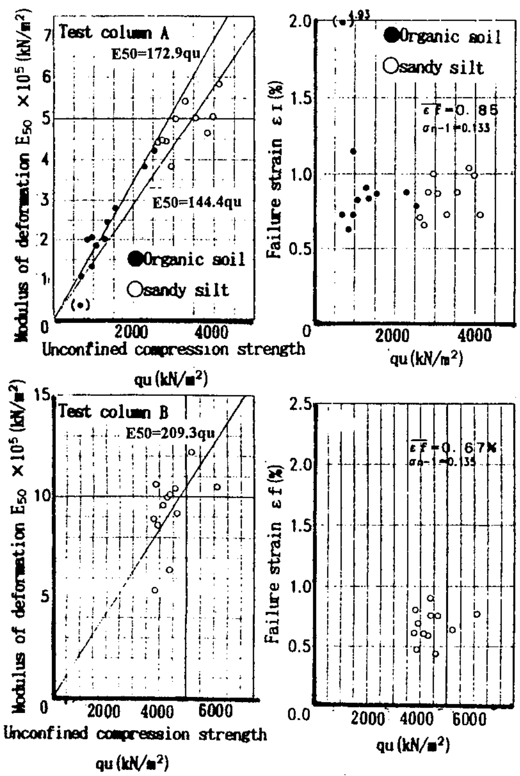


Figure 153. Relationship between q_u and E_{50} (Futaki et al., 1996).

Table 31. Trial soil-grout mix proportions and properties (Walker, 1994).

SLURRY AND GROUT PROPERTIES	1A	2A	3A	4A	5A	6A	1B	2B	3B	4B	5B	6B
Soil Sample	SP-SM	SP-SM	SP-SM	SP-SM	SP-SM	SP-SM	SP-SC	SP-SC	SP-SC	SP-SC	SP-SC	SP-SC
Grout Ratios C/W B/W G/S	33 4 32	33 4 37	25 6 31	19 6 30	19 6 34	14 7 29	33 4 32	33 4 37	25 6 31	19 6 30	19 6 34	14 7 29
VISC (CP @ 600 rpm)	99	103	140	153	167	186	99	103	140	153	167	186
UNIT WEIGHT (t/m^3)	1.92	1.92	2.00	1.97	1.84	1.86		1.94		1.94		
USC (KN/m^2) 3 days 7 days 28 days 60 days	136 358 411	150 353 406	140 212 276	66 75 103 330	94 138 241	53 57 44 121	132 76 576	132 240 92 217	138 203 220	96 129 191	114 136 178	98 82 147
CEMENT CONTENT % BY WT. OF SOILCRETE	5.9	6.5	4.5	3.5	3.9	2.6		6.5		3.5		
PERMEABILITY (cm/sec) x 10^4	0.091	0.195	0.290	0.310	0.691	0.192	-	1.02	-	0.175	-	-

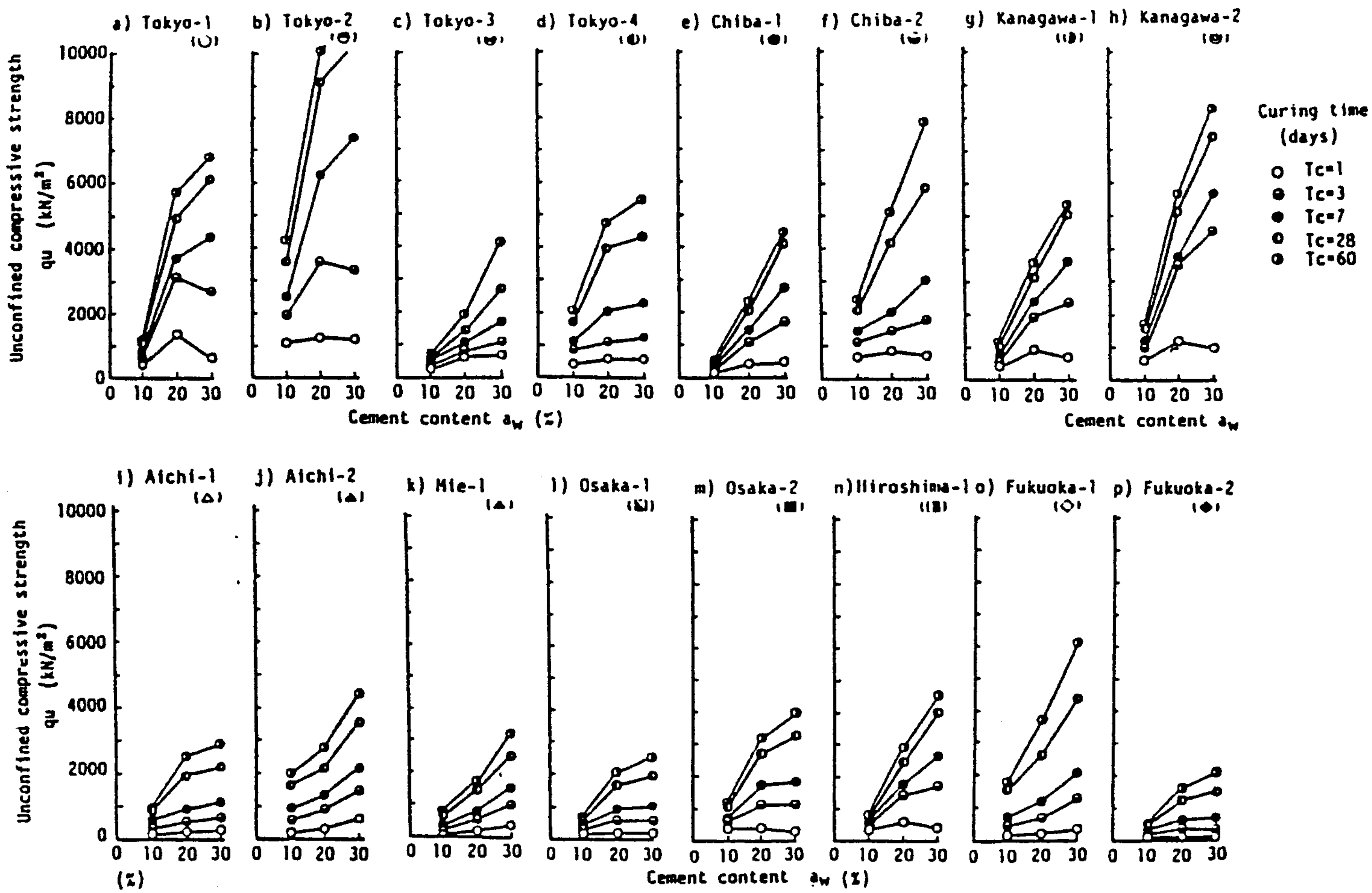


Figure 154. Relationship between unconfined compressive strength, q_u, cement content, and curing time, T_C, for improved soil (Takenaka, 1995).

Table 32. Physical and chemical properties of pre-improved soil (Takenaka,1995).

Name	Symbol	Soil Type	Physical Properties									Chemical Properties							
			Grain Size Distribution			Atterberg Limit						Organic Matter Content							
			Sand (74-2000μ)(%)	Silt (5-74μ)(%)	Clay (5μ>)(%)	Liquid Limit W_l(%)	Plastic Limit W_p(%)	Plasticity Index I_p	Specific Gravity G_s	Natural Water Content W_0(%)	Centrifuge Moisture Equivalent W_c(%)	Ignition Loss (%)	Dichromic Acid Method (%)	Humus Content (%)	Absorbance of Extract	Chlorides (%)	Sulfates (%)	Water Soluble Component (%)	pH-Value
Aomori -1	⊘	(L)	34.0	43.0	23.0	230.0	72.6	157.4	2.434	305.1	114.8	11.9	11.2	2.90	0.80	0.01	0.31	0.41	4.5
Akita -1	⊗	(L)	5.0	33.0	62.0	100.4	36.2	64.2	2.601	100.1	78.9	9.5	6.6	1.87	1.02	0.00	0.05	0:12	3.4
Ibaragi -1	⦶	(L)	4.0	43.0	53.0	107.8	38.8	69.0	2.625	157.7	90.1	12.3	5.0	0.92	0.79	0.01	0.04	0.10	4.3
Ibaragi -2	⊕	(L)	9.0	33.0	58.0	78.3	31.7	46.6	2.625	92.4	61.9	11.5	6.4	2.37	3.60	0.01	0.02	0.04	4.1
Tokyo -1	○	(P)	1.0	41.0	58.0	90.5	35.2	55.3	2.746	99.6	71.0	5.4	4.7	0.39	0.12	0.88	1.48	2.77	7.2
Tokyo -2	◓	(L)	14.0	55.0	31.0	52.4	28.0	24.4	2.720	62.8	49.7	4.4	3.4	0.30	0.40	0.05	0.20	0.40	7.8
Tokyo -3	●	(L)	25.0	61.0	14.0	49.3	21.5	27.8	2.728	58.4	36.9	4.4	3.5	0.37	0.25	0.01	0.50	1.00	3.8
Tokyo -4	◐	(L)	1.0	45.0	54.0	85.1	29.7	55.4	2.661	86.3	70.6	7.8	4.2	0.26	0.20	0.01	0.15	0.34	5.9
Chiba -1	●	(P)	2.0	50.0	48.0	95.7	31.7	64.0	2.749	123.0	72.4	9.0	3.1	0.39	0.08	2.33	0.30	4.58	7.5
Chiba -2	◒	(P)	44.0	32.0	24.0	55.8	24.0	31.8	2.692	61.0	44.6	9.9	4.1	0.34	0.29	0.04	0.02	0.14	6.6
Kanagawa -1	◑	(P)	16.0	54.0	30.0	91.0	31.5	59.5	2.739	109.7	87.2	6.3	4.3	0.71	0.04	1.86	0.22	3.46	6.7
Kanagawa -2	◎	(P)	8.0	45.0	47.0	85.0	33.3	51.7	2.716	97.1	63.9	7.3	3.6	0.28	0.11	1.31	0.21	2.66	7.3
Aichi -1	△	(P)	5.0	61.0	34.0	83.4	23.4	60.0	2.724	99.3	62.7	6.6	3.3	0.60	0.02	1.59	0.25	3.12	7.4
Aichi -2	▲	(P)	22.0	73.0	5.0	46.0	25.4	20.6	2.773	54.4	32.8	5.1	3.6	0.36	0.02	1.45	0.23	2.86	7.2
Mie -1	▲	(P)	29.0	59.5	11.5	46.9	27.1	19.8	2.741	56.7	28.9	6.4	6.0	0.39	0.03	0.92	0.20	1.87	7.3
Mie -2	▽	(L)	15.0	61.0	24.0	80.0	32.5	47.5	2.608	122.2	60.4	9.1	6.8	1.58	0.76	0.14	0.00	0.27	3.9
Osaka -1	◪	(P)	2.0	62.0	36.0	61.2	24.3	36.9	2.695	55.8	49.3	5.6	4.7	0.33	0.06	0.08	0.21	0.58	7.3
Osaka -2	■	(P)	5.0	41.0	54.0	95.0	29.9	65.1	2.693	113.5	76.5	9.1	3.8	0.37	0.07	1.94	0.21	3.79	7.5
Hyogo -1	□	(P)	3.0	33.0	64.0	116.1	32.5	83.6	2.731	118.0	87.6	9.3	4.5	0.45	0.10	2.25	0.26	4.40	7.4
Hiroshima -1	◨	(P)	3.0	61.0	36.0	121.0	36.1	84.9	2.677	136.3	89.9	8.4	4.9	0.54	0.01	1.85	0.35	3.64	7.3
Fukuoka -1	◇	(P)	24.0	64.5	11.5	41.5	25.5	16.0	2.705	55.8	28.3	5.4	1.2	0.14	0.01	0.92	0.25	2.04	7.1
Fukuoka -2	◆	(P)	2.0	65.0	33.0	93.1	25.2	67.9	2.664	149.2	87.2	9.0	3.4	0.36	0.02	0.01	0.04	5.75	7.2

Note: (P)...Soils below sea bottom
(L)...Originally marine soils now on land

Uchida et al. (1996) described the massive CDM treatments for the Trans Tokyo Bay Highway Project (Figure 155). The maximum depth of water is 30 m. The work was completed in 1994. DMM was used in three areas, in two styles, for a total of 1.837 million m^3 of treated soil (Table 33).

At Ukishima, 1,248,000 m^3 of "low strength" DMM was used in the soft clays to facilitate subsequent shield tunneling. The target strength was 1 to 3 MPa, the cement factor was 69 kg/m^3, and the water/cement ratio = 1.0, to promote uniform mixing. Verification was provided by UCT, and CUTC testing on "RCT" samples (assumed to be cores). PS logging was also used to confirm treatment homogeneity.

At Kawasaki Island, 132,000 m^3 of clays were treated to provide "ordinary type DMM" (i.e., cement factor = 137 kg/m^3, water/cement ratio = 1.0). Subsequent excavation allowed the surprisingly good homogeneity of the treated mass to be demonstrated. A series of TC tests was conducted, as were field seismic surveys (refraction). A further 168,000 m^3 of soils were treated using the "low strength" DMM.

At Kisarazu Island, 289,000 m^3 of "low strength" DMM was conducted to treat soft clays.

Unami and Shima (1996) provided further details of the low-strength CDM as used at the Ukishima site. The work was conducted by an 8-shaft, barge-mounted machine through a water depth of over 21 m (Figures 156 through 158). Mix design was as shown in Table 34. "Undisturbed" 86-mm triple tube cores were sampled "continuously" (Figure 159) to provide U.C.S., E, and strain data to supplement natural water content and wet density data.

They concluded, inter alia.,

$$E_{max} = 107.19 \times q_{umax}^{1.553}$$
$$E_{max} = 1.43 \times E_f^{0.914}$$

where E_f = dynamic Young's modulus.

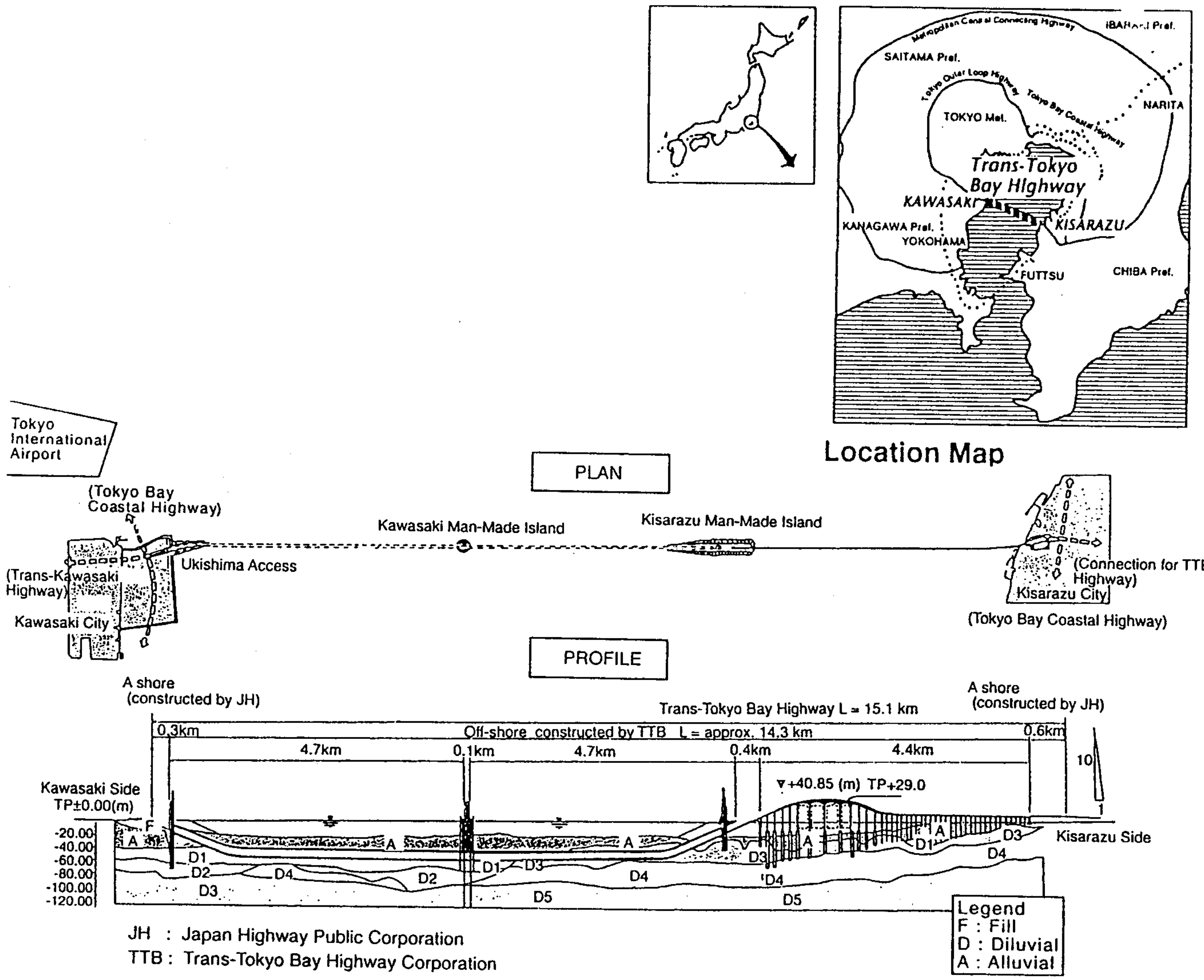


Figure 155. General plan of the Trans-Tokyo Bay Highway (Uchida et al., 1996).

Table 33. Ground improvement techniques by cement-treatment used for the TTB highway project ($1.0\ kgf/cm^2 = 98$ MPa) (Uchida et al., 1996)

Cement-treatment method	Mixing proportion	Construction site	Volume ($\times 1{,}000m^3$)
Ordinary type DMM	cement: $1.372kN/m^3$ w/c ratio: 100%	Kawasaki man-made island	132
Low strength type DMM	cement: $0.686kN/m^3$ w/c ratio: 100%	Ukishima m-m isl. Kisarazu m-m isl. Kawasaki m-m isl.	1,248 289 168
Slurry mixture cement-treated sand	sand: $11.535kN/m^3$ cement: $0.980kN/m^3$ clay: $1.078kN/m^3$ sea water: $0.495kN/m^3$	Kisarazu m-m isl. Ukishima m-m isl. Kawasaki m-m isl.	1,028 351 118
Dry mixture cement-treated sand	sand: $13.034kN/m^3$ cement: $0.980kN/m^3$ anti-segregation adhesive: $1.078kN/m^3$	Kisarazu m-m isl.	435

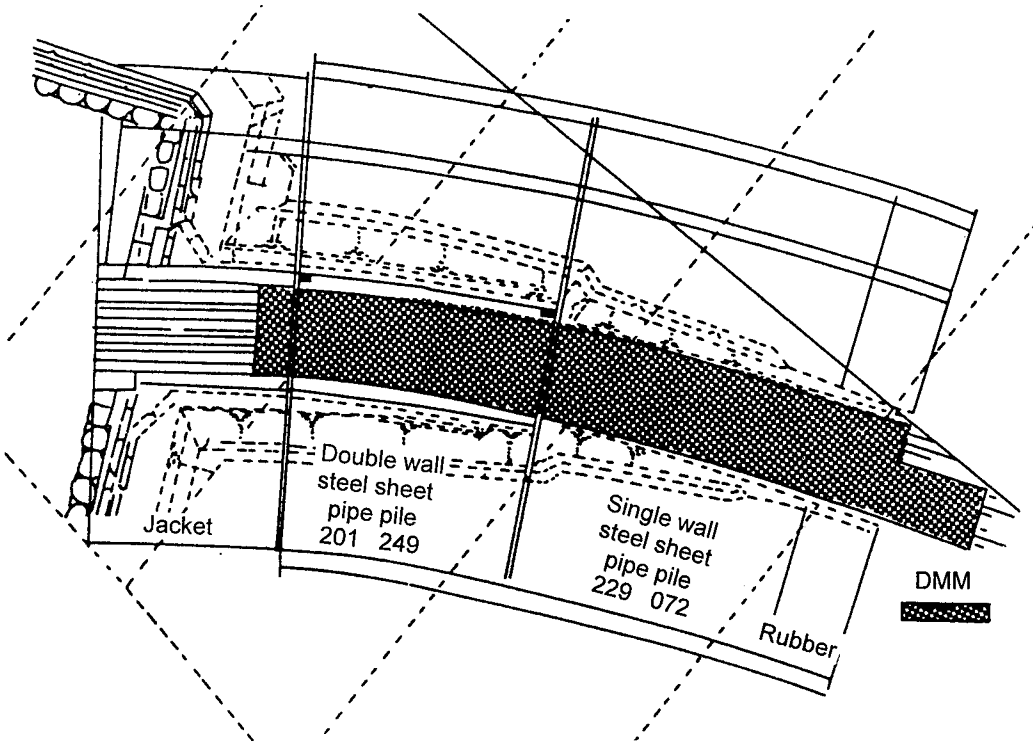


Figure 156. Plan of DMM execution area (Unami and Shima, 1996).

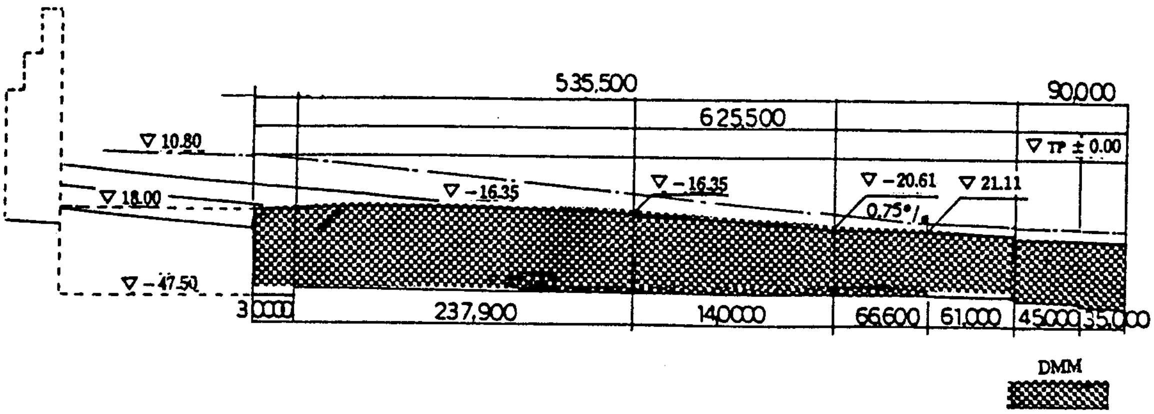
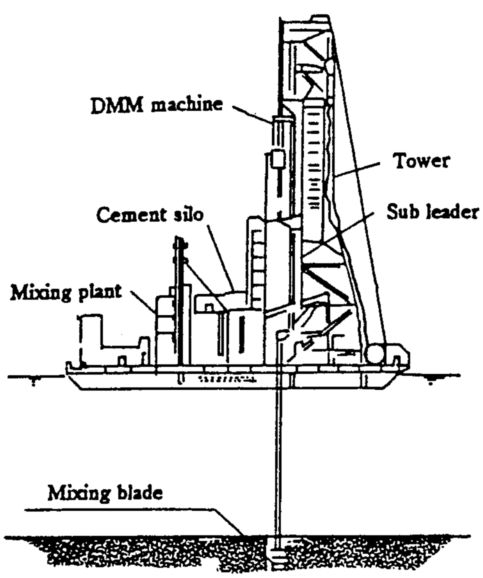


Figure 157. Elevation view of DMM execution area (Unami and Shima, 1996).



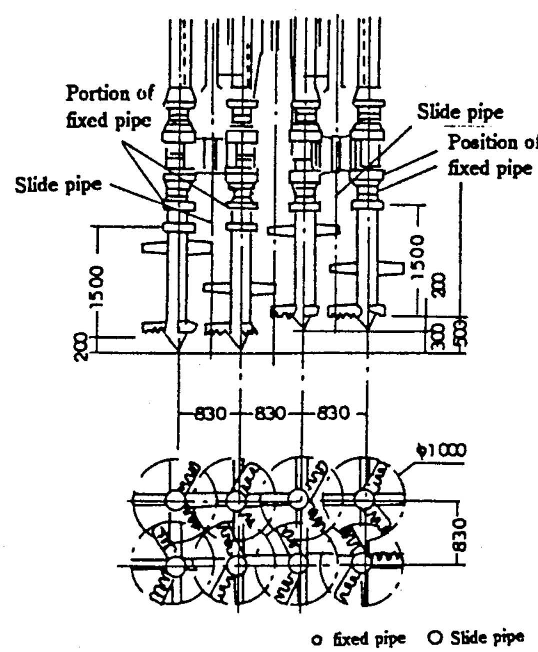


Figure 158. Detail of mixing blades (Unami and Shima, 1996).

Table 34. Design of mix proportion for permanent DMM piles (Unami and Shima, 1996)

Material \ Layer	Deposit file layer	Original ground layer	Appendix
Cement	70 kg	110 kg	
Water	70 kg	110 kg	
Additive agent	C x 0.25%	C x 0.25%	C equals to the quantity of cement

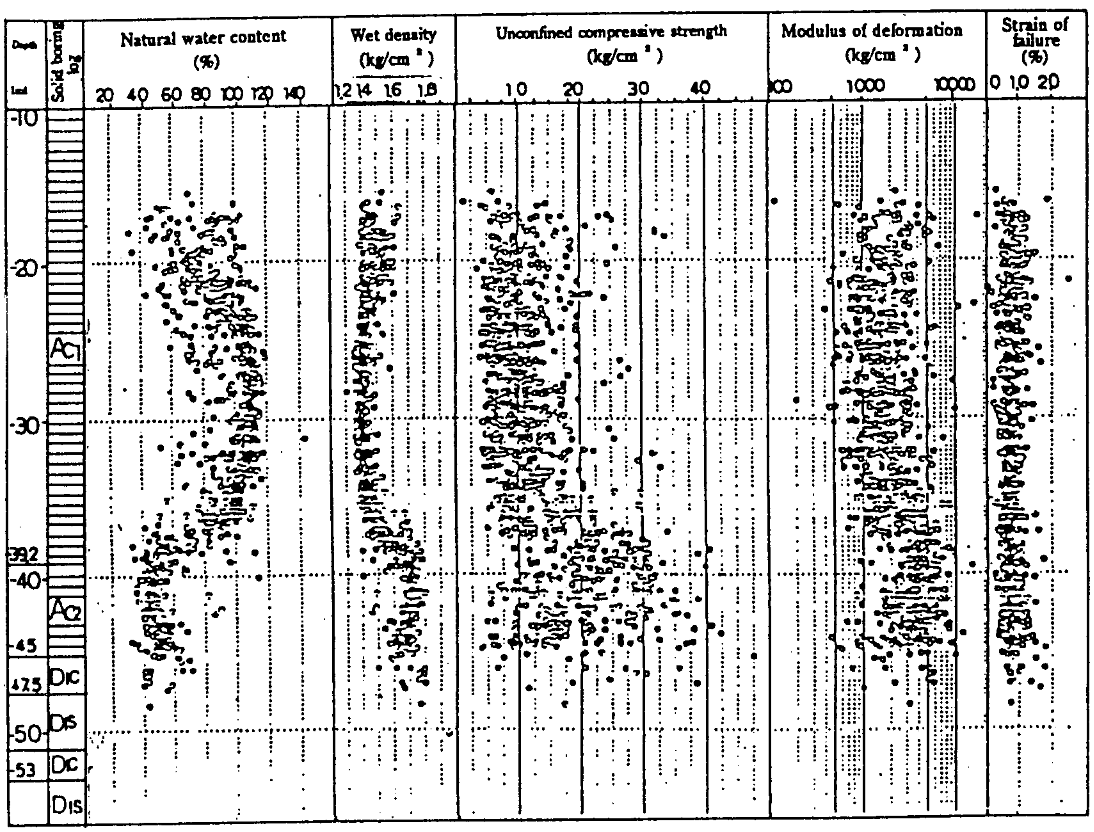


Figure 159. Diagram of post execution laboratory test for DMM improved soil (Unami and Shima, 1996).

For the evaluation of in situ U.C.S. (q_f)

$$q_f = (0.013 \times E_f^{0.914})^{0.644}$$

and also reflects RQD (Figures 160 and 161). The authors concluded that "the evaluation of quality of the improved soil with the elastic wave velocity seems to be a useful tool."

Min (1996) described extensive field testing prior to using CDM for 60,000 m^3 of treatment in soft silts, with sandy layers, at Yantai Port, PRC. Laboratory tests were first conducted on 384 samples. They found that the 90:28-day U.C.S. ratio was 1.5 for silt and 1.4 for low plasticity silt (Table 35) and recommended that the mix comprise portland cement Grade 425, a cement factor of 170 to 190 kg/m^3, and a water/cement ratio of 1.3.

In subsequent production, the parameters were revised three times, as shown in Table 36 and Table 37 (Double Tube Core Barrel) in order to optimize the process and achieve the target U.C.S. of 2.5 MPa. For the low water/cement mixes, 3% calcium lignosulfonate was used as a plasticizer. This was an excellent demonstration of the influence of cement factor and water/cement ratio on strength development and of the value of a meticulous, well-conceived, and responsive rolling test program.

Mizutani et al. (1996) described the results of a field test designed to demonstrate a relatively new DMM variant (Method 12). Six columns were installed in the clayey soils detailed in Figure 162, each 1-m square and 5.8-m deep, using the installation parameters of Table 38. Major conclusions were:

- Cores were taken both vertically and horizontally (Figures 163 through 165), which led to the conclusion that the columns were of uniform strength throughout.
- Core recovery was everywhere in excess of 92%.
- The variation in U.C.S. values was within the usual 20 to 40% range for DMM.

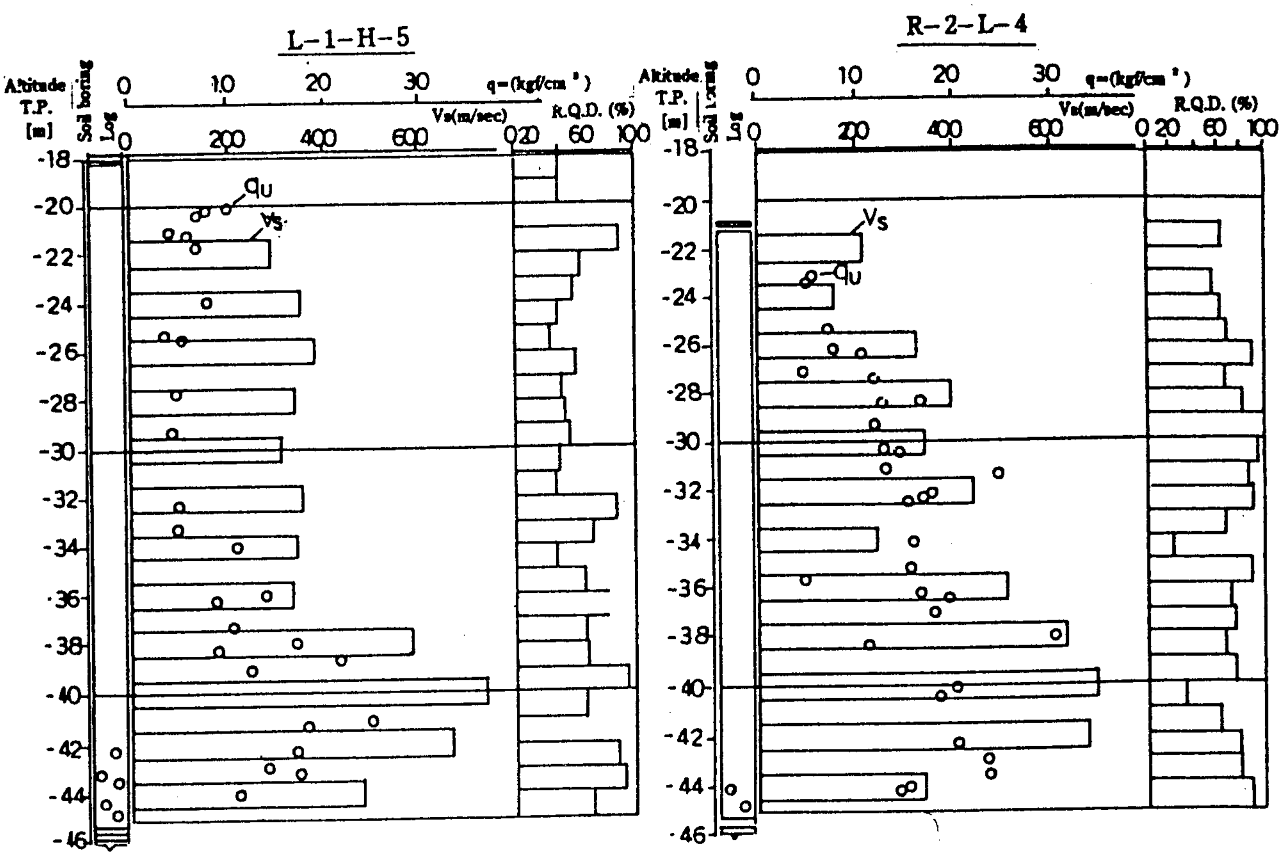


Figure 160. Diagram of RQD, q_u, and v_s (Unami and Shima, 1996).

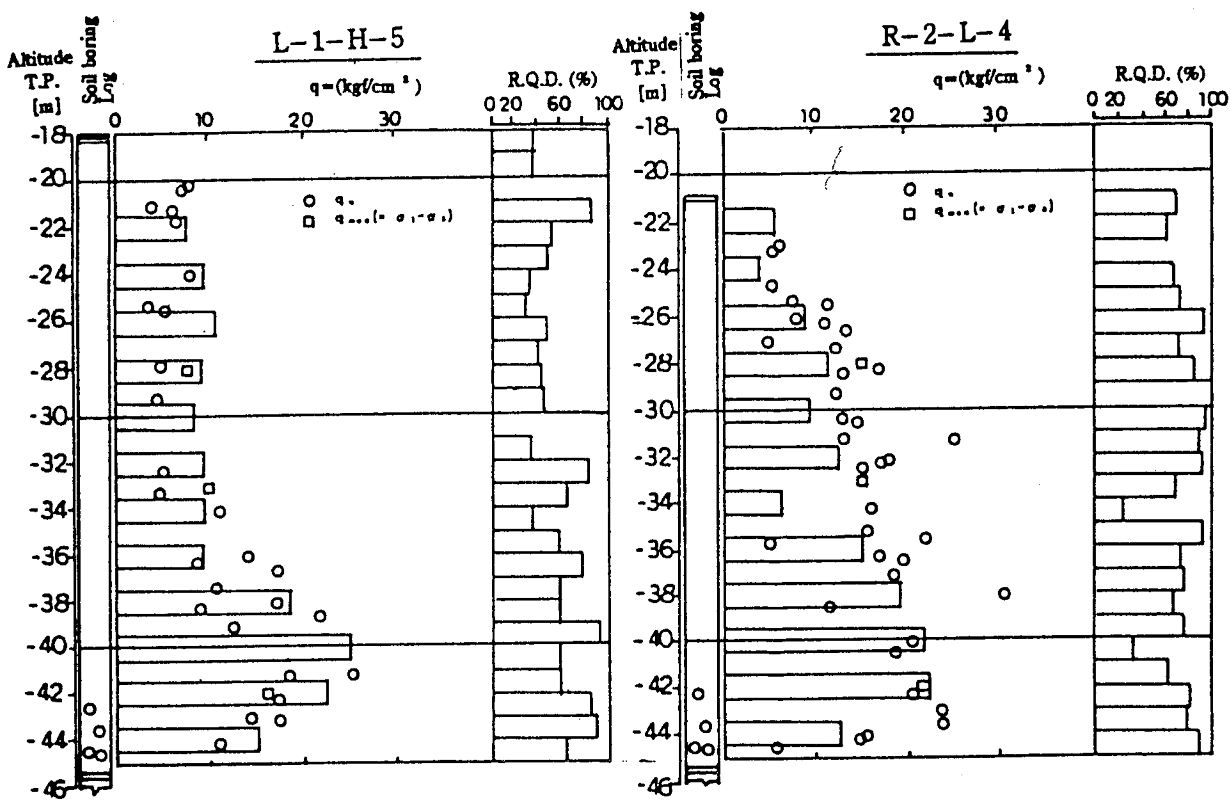


Figure 161. Diagram of q_u, q_{max}, and RQD (Unami and Shima, 1996).

Table 35. Properties of submarine soils at Yantai Port (Min, 1996).

Description of soils	Water content (%)	Specific gravity	* Activity (mS/cm)	Mineralogical composition		Organic content (%)	Humus content (%)	pH value	Soluble salt content (%)
				main	secondary				
Silt	32	2.68	0.75	quartz	grundite, chlorite	1.51	0.063	7.45	0.434
Silt of low plasticity	28	2.67	0.77	quartz	chlorite	0.77	/	7.70	0.210

NOTE: * The activity of a soil is evaluated by its conductivity.
Conductivity < 0.4 mS/cm = non-pozzolana; 0.4mS/cm < conductivity < 1.2mS/cm = ordinary pozzolanicity; conductivity > 1.2mS/cm = good pozzolanicity

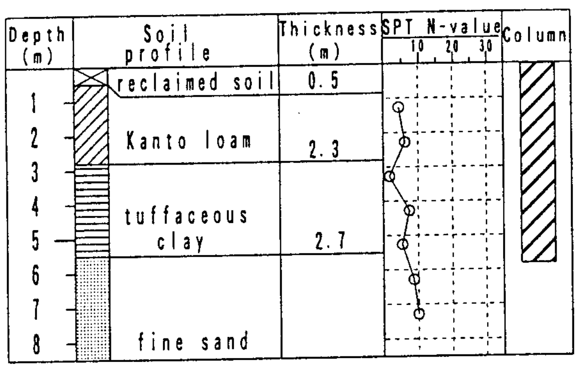
Table 36. Readjustments of mixing ratio and mixing procedure (Min, 1996).

Time	Amount of cement (kg/m3)	Water con-tent (W/C)	Admix-ture (%)	Penetration		Withdrawal		Reasons of readjustment
				speed (m/min)	revolution (r.p.m.)	speed (m/min)	revolution (r.p.m.)	
Dec. 1992	190/170	1.3	/	0.7, 1.0	25.4	1.0	25.4	Based on laboratory mixing ratio test
Apr. - June 1993	200/180	1.0	/	0.5	25.4	1.0	50.8	Strength low and poor uniformity according to the first test on field samples
July - Sept 1993	230/170	0.9	3	0.5	25.4	1.0	50.8	Strength still lower than required according to the second test on field samples
After Sept 1993	190/160	0.9	3	0.5	25.4	1.0	50.8	Strength a bit higher than required according to the third test on field samples

Table 37. Field sampling and results of laboratory strength tests (Min, 1996)

Sampling number	Borehole number	Amount of cement (kg/m3)	Water/ cement ratio	Amount of admixture (%)	Time of CDM operation (d/m/yr)	Time of sampling (d/m/yr)	Age day	Sampling rate (%)	Number of samples	Average strength (MPa)
I	1	170/160	1.3	/	7.12.1992	3.4.1993	88	78	22	0.21-21.62*
II	2	200/180	1.0	/	17. 4.1993	18. 6.1993	62	100	24	1.81
	3	200/180	1.0	/	13. 4.1993	20. 6.1993	68	100	24	2.26
III	4	230/170	0.9	3	20. 7.1993	18. 9.1993	60	100	18	3.02
	5	230/170	0.9	3	15. 7.1993	18. 9.1993	65	100	22	2.45
	6	230/170	0.8	3	16. 7.1993	20. 9.1993	66	100	21	3.87
	7	230/170	1.0	/	15. 7.1993	22. 9.1993	69	100	17	3.38
IV	8	190/160	0.9	3	25. 9.1993	15.12.1993	82	100	19	2.27
	9	190/160	0.9	3	26. 9.1993	16.12.1993	82	100	18	2.72

NOTE: * Strength differs greatly as the mixture is not uniformly mixed.



soil type	w (%)	G_s	w_l (%)	w_p (%)	I_p	grain size(%) sand	silt	clay
loam	129.9	2.67	179.6	89.4	89.4	6	56	38
clay	54.2	2.67	90.8	47.2	49.6	14	56	30

NOTE: w : water content
G_s : specific gravity
w_l : liquid limit
w_p : plastic limit
I_p : plasticity index

Figure 162. Soil profile and parameters (clayey soil) (Mitzutani et al., 1996).

Table 38. Installation records (Mizutani et al., 1996)

execution rate (m/min) penet-ration	execution rate (m/min) pull -out	r.p.m.	stabilizer amount (kg/m^3)	stabilizer type
0.5	1.0	30	100,200,300	cement-type

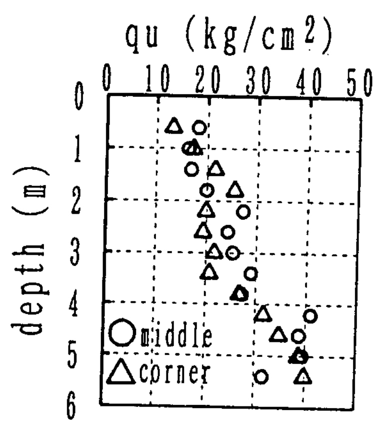


Figure 163. q_u distribution (Mizutani et al., 1996).

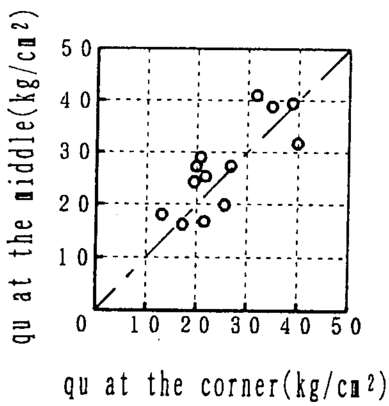


Figure 164. q_u comparison (Mizutani et al., 1996).

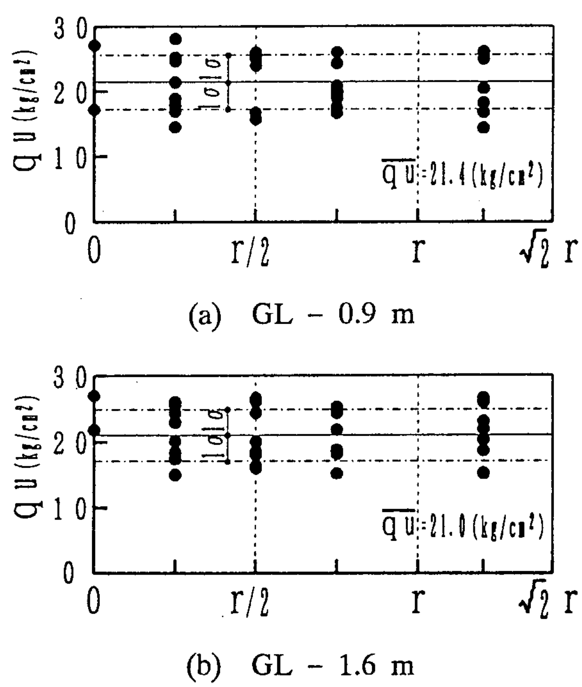


Figure 165. q_u distribution in section (Mizutani et al., 1996).

- Figure 166 shows the relationship between U.C.S. and cement factor for laboratory specimens. The field core strengths were 75% (loam) and 60% (tuffaceous clay) of these values compared with the 20 to 33% ratio often reported elsewhere for other DMM techniques.

A further 20 columns, each 1-m-square and 40-m deep, were installed in loose, alluvial sand (Figure 167). The data (Table 39) from cores taken axially and laterally indicated:

- Reducing the cement factor from 200 to 150 kg/m^3 led to a major strength reduction.
- Silty sand gives noticeably lower strengths than cleaner sand.
- Raising the water/cement ratio leads to lower strength (in the range of 1.0 to 1.2). (Volume ratios varied from 25 to 44 percent.)

Takenaka and Takenaka (1995) described a test of the distribution of direct shear strength under "controlled" field conditions (Figure 168). Given that all the data showed a great scatter, there is some, slight indication of an average reduction in strength toward the perimeter. (This very much reflects the soil and the mixing characteristics: Lambrechts (1999) believes that this contrast in strength would be much more pronounced in the SMW work conducted in the clays in Boston.)

Yoshida (1996) reported on direct field experiments to investigate the interface shear strength between adjacent 1-m-diameter columns (Figure 169). The evidence suggests that interlock shear strength was only 70% of column shear strength up to a 4-day overlap interval. At an interval of 6 days the shear strength was zero. This conclusion appears to be supported by data from Saitoh et al. (1996), who also highlighted the lower U.C.S. values obtained at interfaces in four different soils (Figure 170). This is important where designs feature mechanical shear between adjacent columns that may be separated in construction by considerable time.

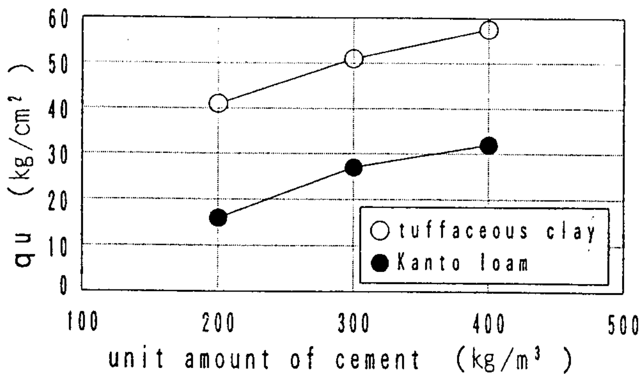


Figure 166. q_u by laboratory mixing test (Mizutani et al., 1996).

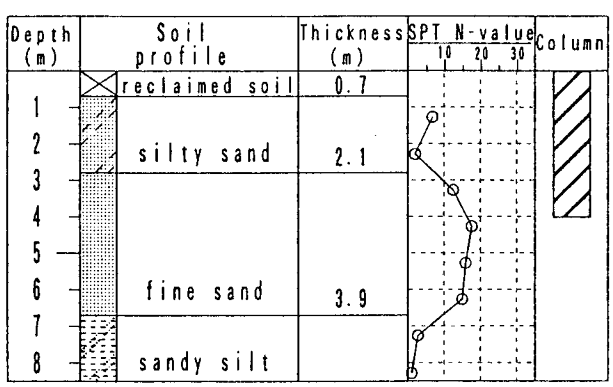


Figure 167. Soil profile (sandy soil) (Mizutani et al., 1996).

Table 39. Comparison of q_u value (Mizutani et al., 1996)

cement factor	soil	location middle	location corner	W/C 100(%)	W/C 120(%)
	silty sand	8.0	---	7.6	8.3
150	sand	15.5	---	15.3	---
(kg/cm^3)	average	8.9	---	9.5	8.3
	silty sand	13.8	16.9	18.7	12.1
200	sand	30.7	32.2	35.5	29.2
(kg/cm^3)	average	22.2	26.4	27.1	21.9
	silty sand	19.1	33.8	28.3	15.4
300	sand	26.6	27.4	32.5	26.3
(kg/cm^3)	average	23.6	29.5	29.1	24.3

NOTE: All units are in kg/cm^2

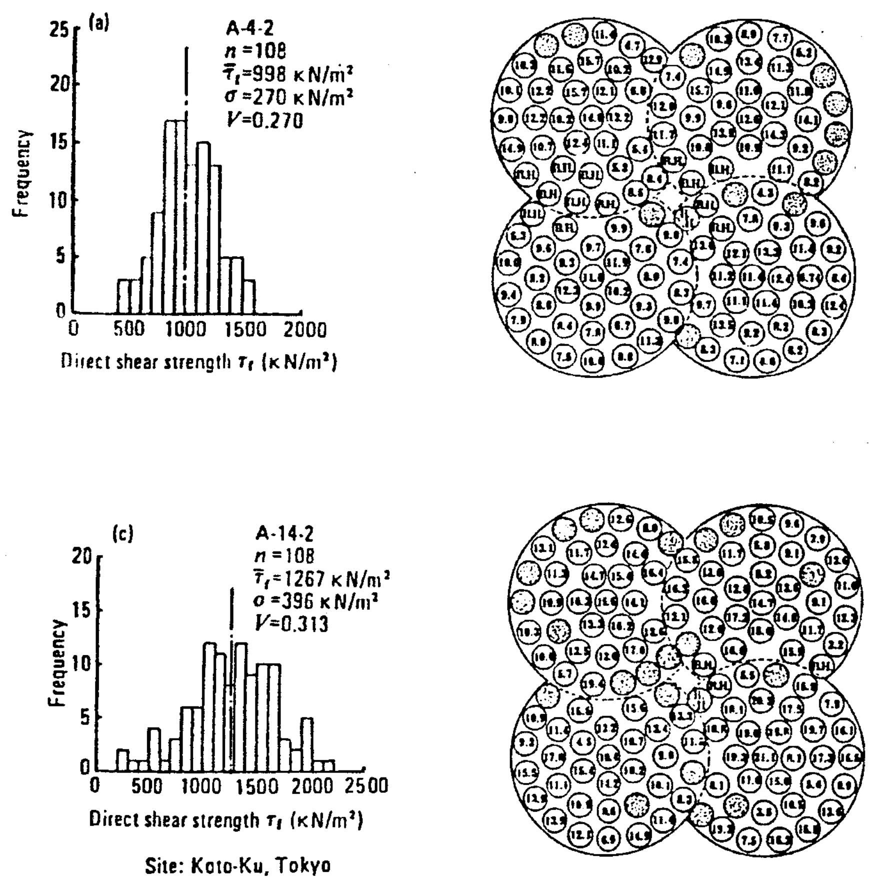


Figure 168. Distribution of shear strength – a trial study (Takenaka and Takenaka, 1995).

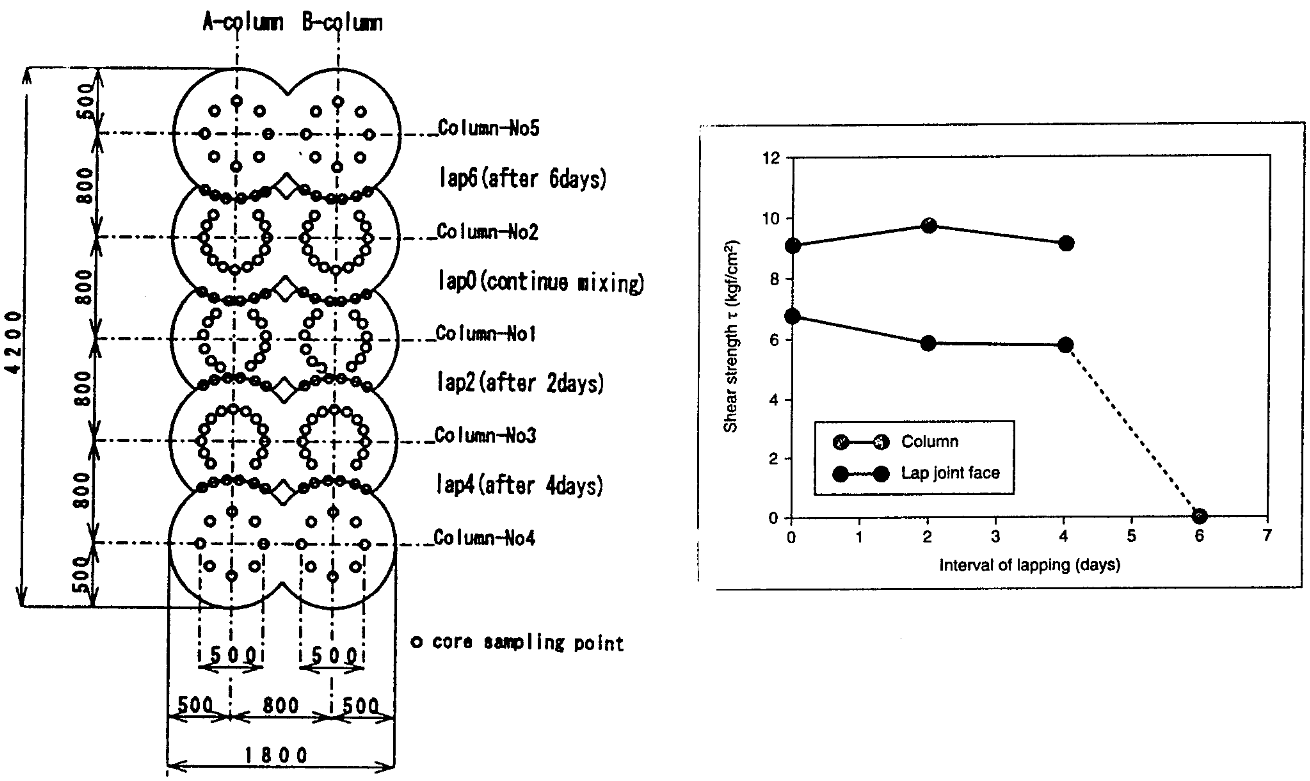


Figure 169. Relationship between shear strength and interval of lapping (Yoshida, 1996).

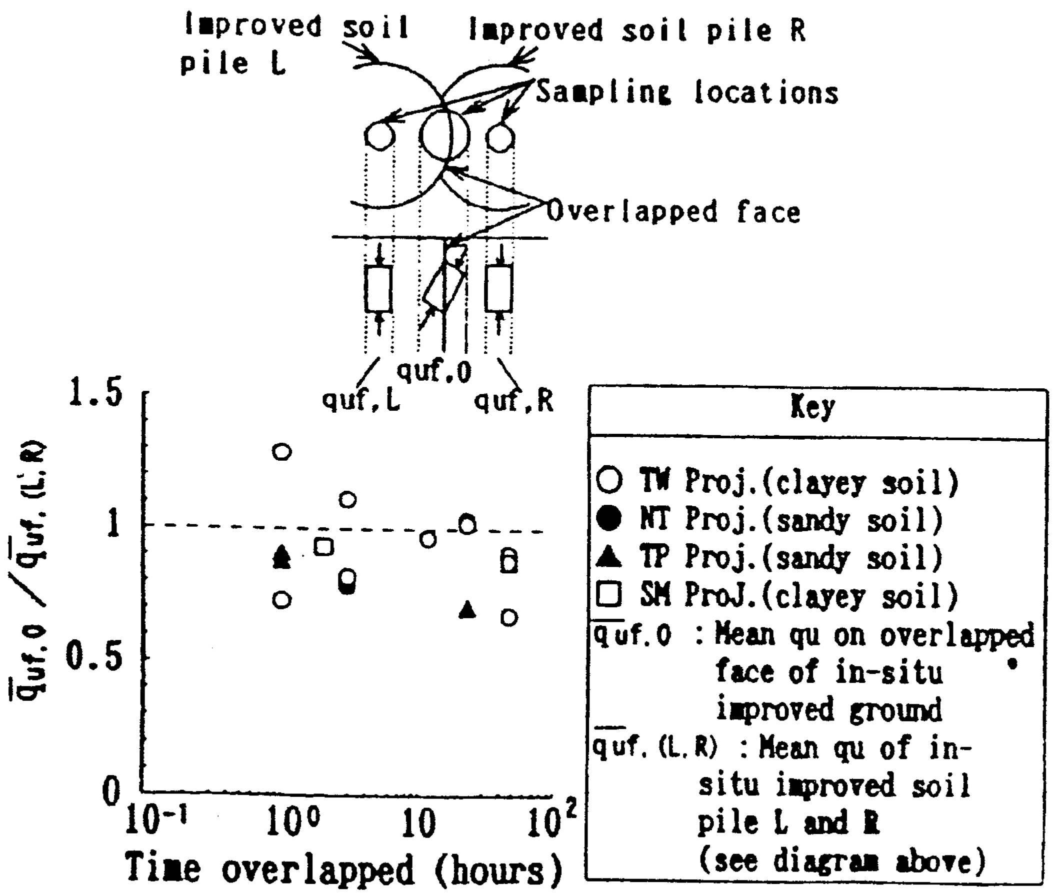


Figure 170. Interface shear strength (Saitoh et al., 1996).

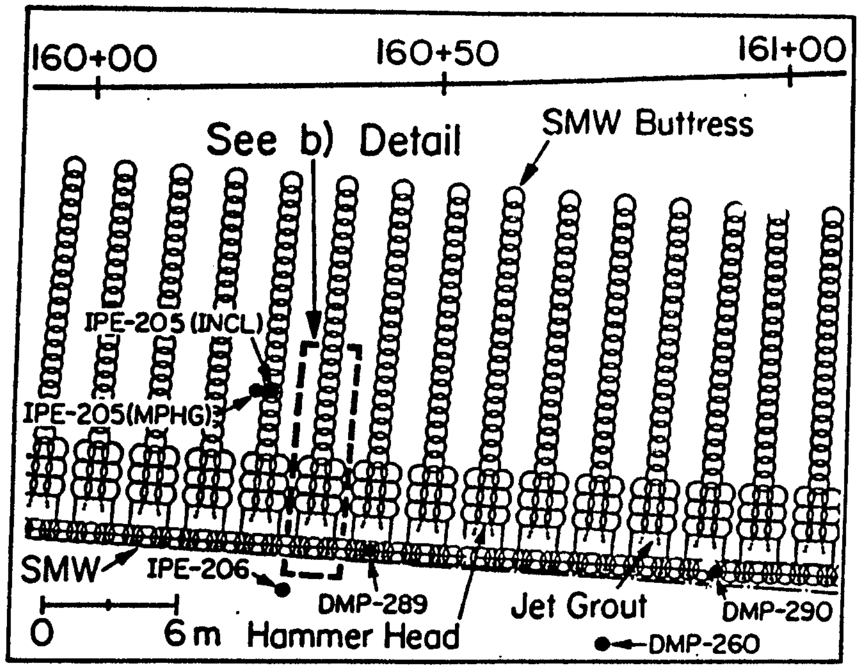
Various authors, including Nicholson and Chu (1994), Yang and Takeshima (1994), and O'Rourke et al. (1997) have described the construction of an SMW wall for earth retention at the Boston Central Artery C07A1 Contract at Logan Airport. This cut and cover tunnel excavation was about 1130 m long, and from 12.5 to 26.5 m deep. "Buttresses" of treated soil were also designed to provide remedial in situ support to the base of the excavation (Figure 171). Figure 172 shows a typical geological sequence, and Table 40 provides additional data. The target minimum design strength was 0.6 MPa, and cement factors were varied along the wall in response to site conditions from 200 to 350 kg/m^3 as shown in Figure 173.

O'Rourke et al. (1997) summarized that field strength data were obtained from unconfined compressive tests on 75- and 150-mm-diameter specimens made from wet grab samples. There was no significant difference in the mean and variance of the two data sets, indicating that 75-mm-diameter specimens were equally as effective as 150-mm-diameter specimens in evaluating field strength characteristics. Little increase in strength was recorded after 7 days.

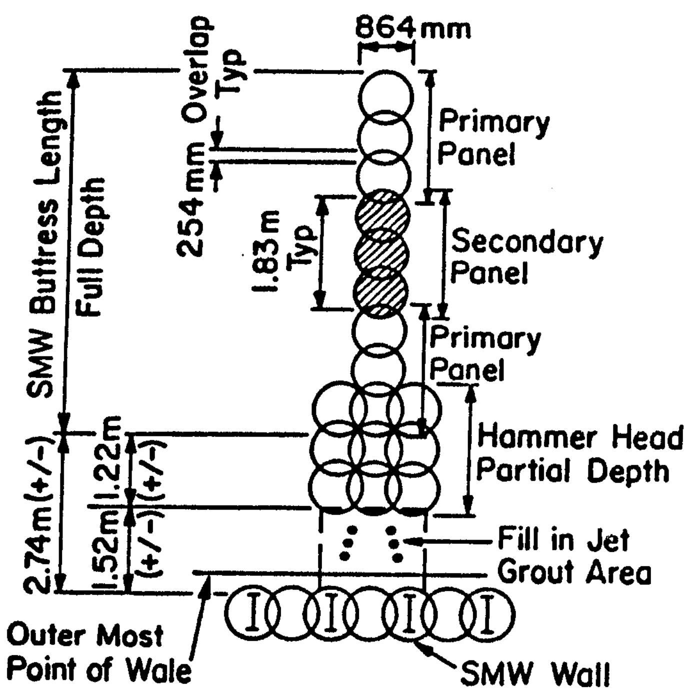
The average strength of all specimens prepared from field samples was on the order of 3.8 MPa, with a coefficient of variation of roughly 0.48. The average q_u value from specimens prepared from columns in which grout was injected only on the upstroke was approximately 6.2 MPa, which is 60% larger than the overall average from all field samples. The coefficient of variation for the upstroke samples was 0.41, only slightly less than that for all field samples.

The volumes of the DMM elements and the records of grout pumped during installation were reviewed to estimate the amounts of cement that could be in place. The calculated apparent cement factor did not correlate strongly with the measured strengths, probably due to construction and recording variations. However, when apparent cement factors were between 250 and 350 kg/m^3, the DMM unconfined compressive strengths were between 2 and 9 MPa, which was satisfactory.

There was a limited correlation between strength and total unit weight (Figures 174 and 175) of field samples, with variations in unit weight accounting for roughly 50% of the strength



a) Plan View



b) Detail

Figure 171. SMW buttresses near MLT station 160+30 (O'Rourke et al., 1997).

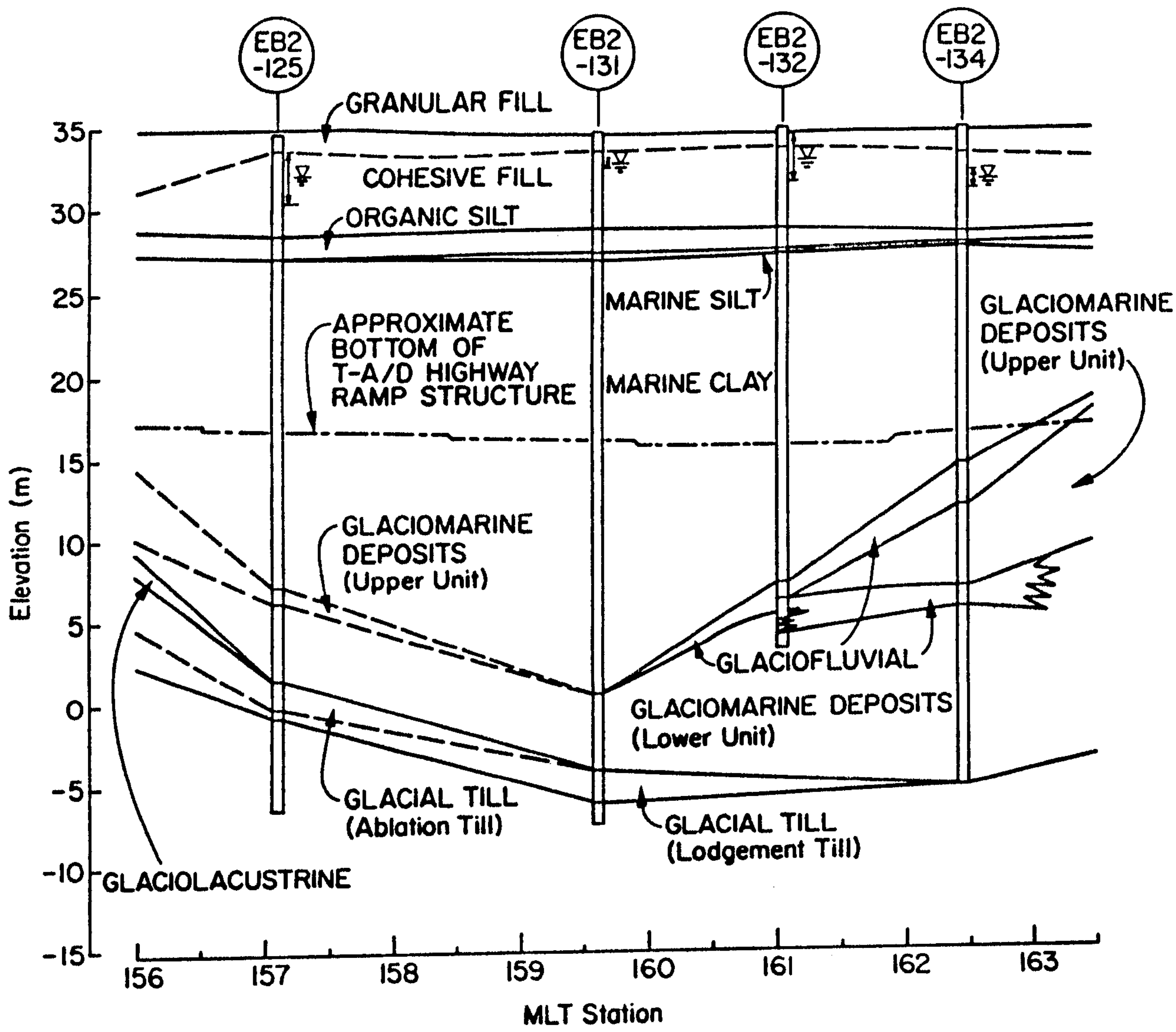


Figure 172. Longitudinal soil profile for MLT stations 156+00 to 163+00- east wall (O'Rourke et al., 1997, after Haley & Aldrich, Inc., 1991a).

Table 40. Soil properties for excavation between MLT station 156+00 and 163+00 (O'Rourke, et al., 1997).

Soil Type	Description	Unit Weight (kN/m^3)[f]	Drained Angle of Shearing Resistance	Undrained Shear Strength[a] (kN/m^2)
Granular Fill	Brown coarse to fine sand, trace of silt and cinders, with fragments of brick, wood and concrete.	18.9	30°	-
Cohesive Fill	Pebble to head-size clay fragments, with infilling of subangular to angular coarse sand and gravel.	18.0	30°	0.3-0.4 σ'_{vo}[b]
Organic Silt	Slightly to moderately overconsolidated medium stiff dark brown to black organic silt, little fine sand, trace of shells and clay.	17.3	30°	33-53[c]
Marine Clay	Gray clay and silt with seams and partings of fine sand			
	Overconsolidated (OC) Crust	18.5	-	119
	Upper Zone	18.5	-	119 - 41[d]
	Middle Zone	18.5	-	46
	Lower Zone	18.5	-	48 - 56
Glaciofluvial Deposits	Gray coarse to fine sand, little coarse to fine gravel, trace silt with cobbles	21.2	42°	-
Glaciomarine Deposits	Gray silt, little coarse to fine sand, clay, trace of fine gravel with cobbles	23.1	-	96-383[e]
Glacial Till	Ablation Till: gray coarse to fine sand and silt, little coarse to fine gravel, trace clay with cobbles and boulders Lodgement Till: gray silt, little coarse to fine gravel, coarse to fine sand, trace of clay, with cobbles and boulders	22.8	45°	-

a - Strengths of marine clay are corrected from VST values (see Figure 2.4) according to Bjerrum (1972, 1973)

b - s_u expressed as a function of vertical effective stress, σ'_{vo}.

c - Average strength ± one standard deviation.

d - Linearly decreasing with depth.

e - Assumed to increase linearly with depth from El. 27.4 m to -3.1 m.

f - 1 kN/m^3 = 102 kg/m^3

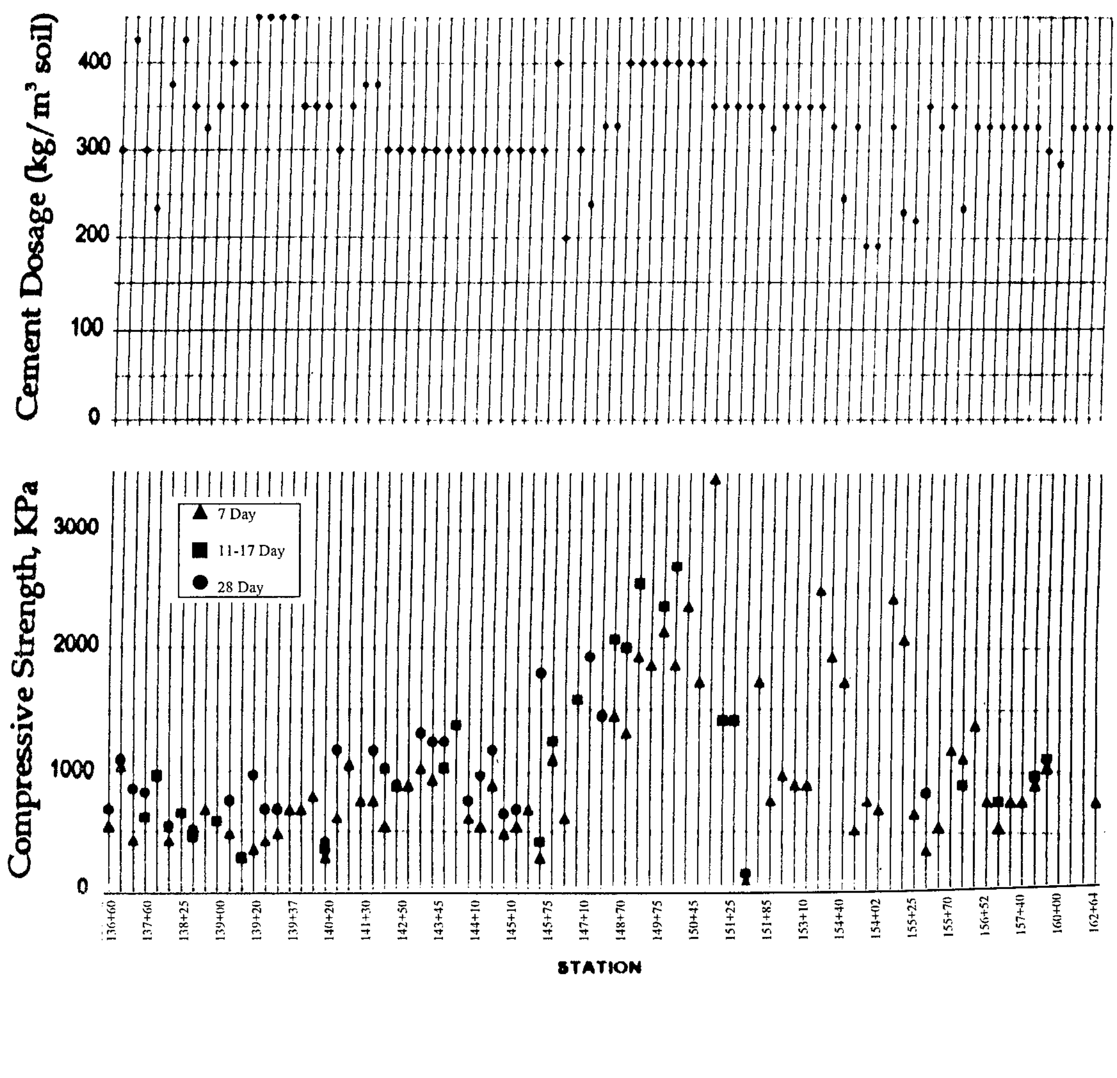


Figure 173. SMW strength and cement factor (Yang and Takeshima, 1994).

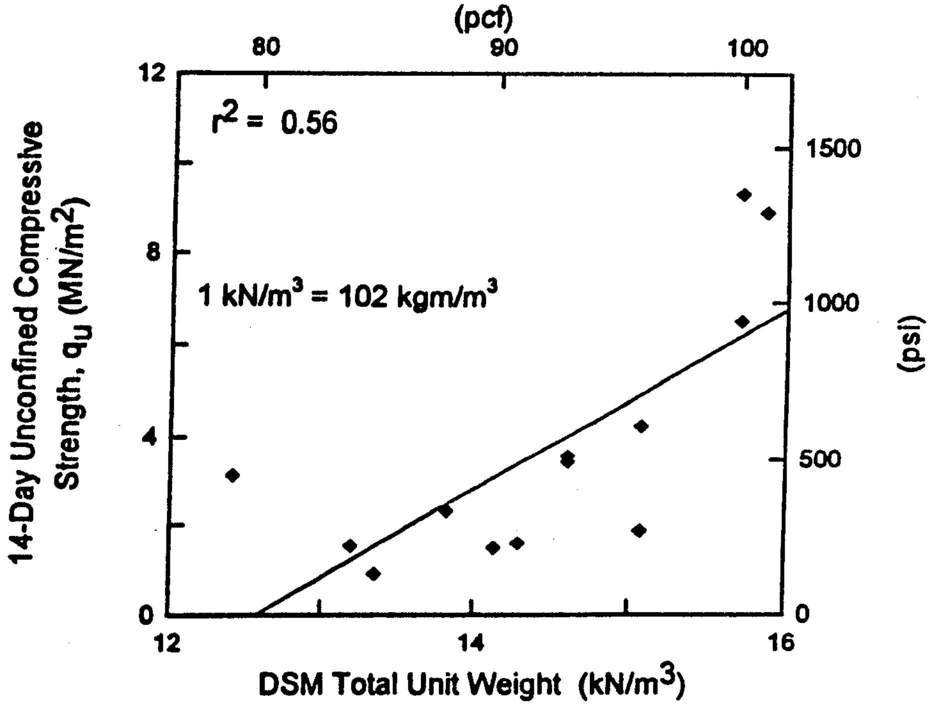


Figure 174. Unconfined compressive strength as a function of total unit weight for data involving different installation procedures (O'Rourke et al., 1997).

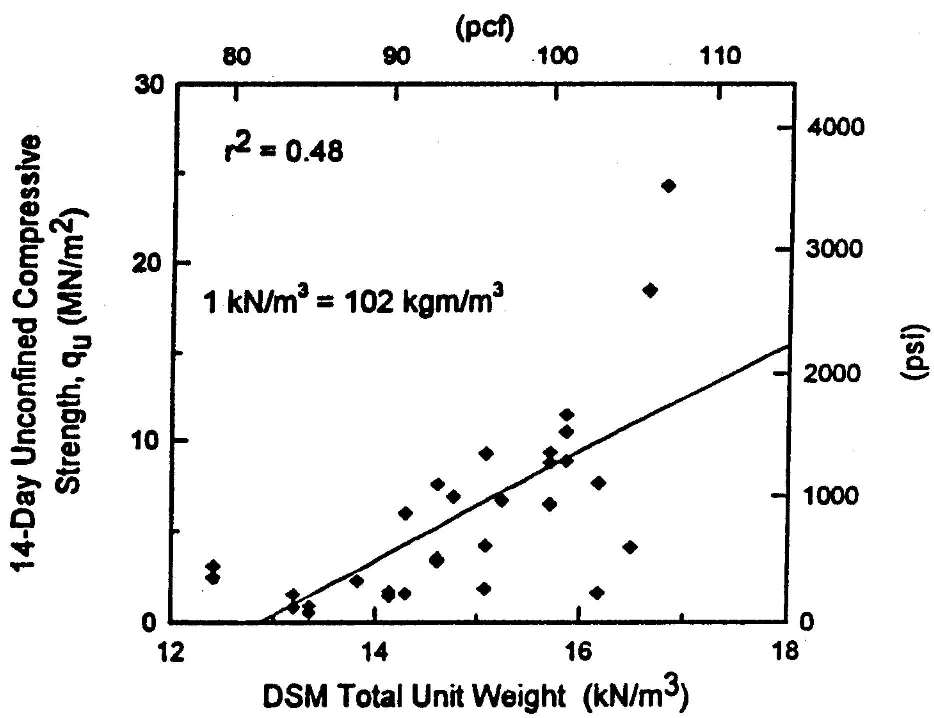


Figure 175. Unconfined compressive strength as a function of total unit weight for screened data set (w/c=1.0:grout on upstroke only)(O'Rourke et al., 1997).

variation. The average field DMM total unit weight was 14.7 ± 1.3 kN/m^3. This is nearly 20% less than the typical 18.5 kN/m^3 total unit weight of marine clay.

Undrained shear test results are provided in Table 41. E-value/U.C.S. relationships for laboratory prepared and cured samples are shown in Figure 176 (E_{50} = 50 to 150 x U.C.S.).

Data from laboratory-prepared soil-cement samples were reviewed to evaluate the effects of mix properties on strength (Figures 177 through 182 and Table 42). The test specimens were prepared at a range of cement factors, which included cement factors comparable to the apparent cement factors determined from field records. Laboratory strengths were strongly dependent on water content. The laboratory data indicate that cement factors in the range of 200 to 250 kg/m^3 at water/cement ratios of 1.25 resulted in U.C.S. of 1 to 2.2 MPa. Strengths of 1.5 to 3.3 MPa were attained at cement factors of 300 to 400 kg/m^3 at water/cement ratios of 0.8 to 1.0.

Two important comparisons were made between the unconfined compressive strengths obtained from laboratory and field test programs. First, the laboratory specimens showed that strength increased with cure time. This is consistent with well-established trends reported from several other experimental programs, and is consistent with general concepts. In contrast, the field data show no significant difference between strengths for different curing times. Secondly, the variability of the laboratory strengths was less than that of the field samples. Coefficients of variation for the laboratory data, determined for each cement factor and water/cement ratio subset (with varying cure times), consistently were lower than the coefficients of variation of the field samples.

The large variability in field strengths may be a result of a) variability in the field mix characteristics, b) variability in field specimen preparation, curing, and handling, or c) a combination of mix and field sample control. Close quality control of field sampling, specimen preparation, and specimen storage is necessary to reduce strength variability.

Table 41. Comparison of shear strengths from linear and nonlinear strength envelopes (O'Rourke et al., 1997).

Confining Stress kN/m^2 σ_3	Shear Strength on Failure Plane from Nonlinear Envelope kN/m^2 [a]	Shear Strength on Failure Plane from Linear Envelope kN/m^2 [b]
0	758 ± 103	758
34.5	896 ± 103	806
172	1034 ± 103	1000

a - Range given by GeoTesting Express (1994)

b - Determined with Equation 5.2 and c = 461 kN/m^2, ϕ = 40°

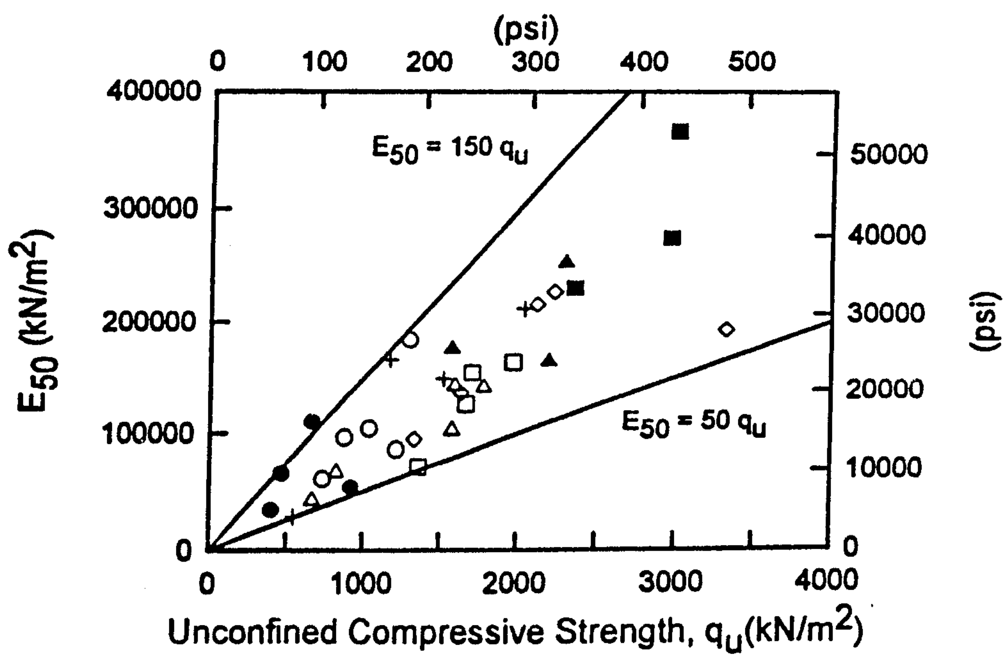


Figure 176. E_{50} versus q_u laboratory prepared and cured soil-cement tested by Geotesting Express (O'Rourke et al., 1997).

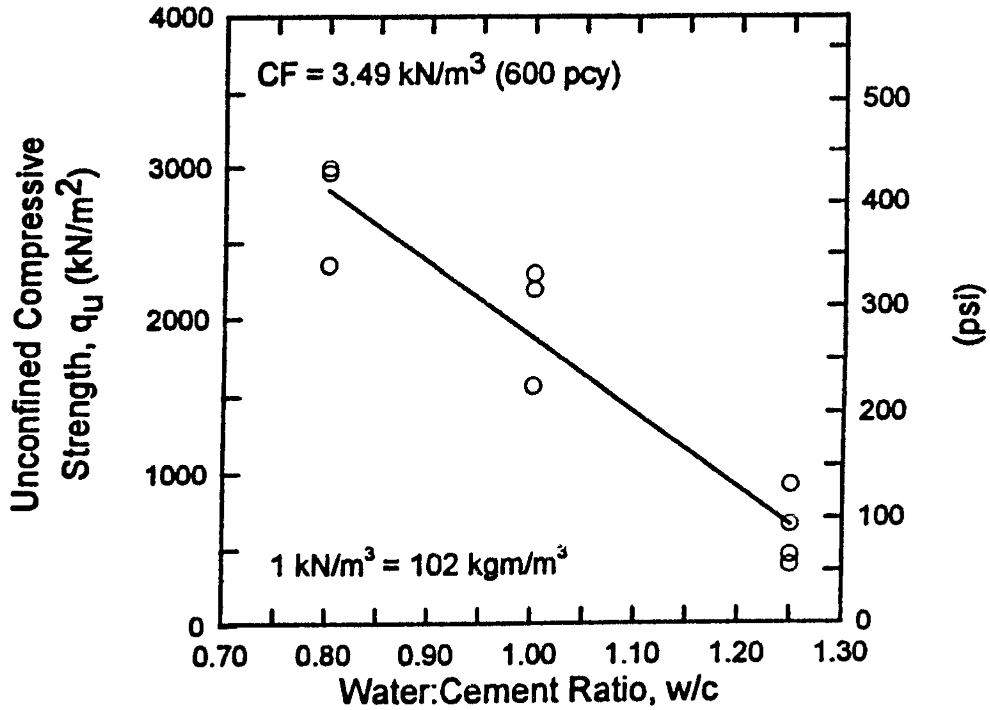


Figure 177. Unconfined compressive strength versus water: cement ratio for a cement factor of 3.49 kN/m^3 (O'Rourke et al., 1997).

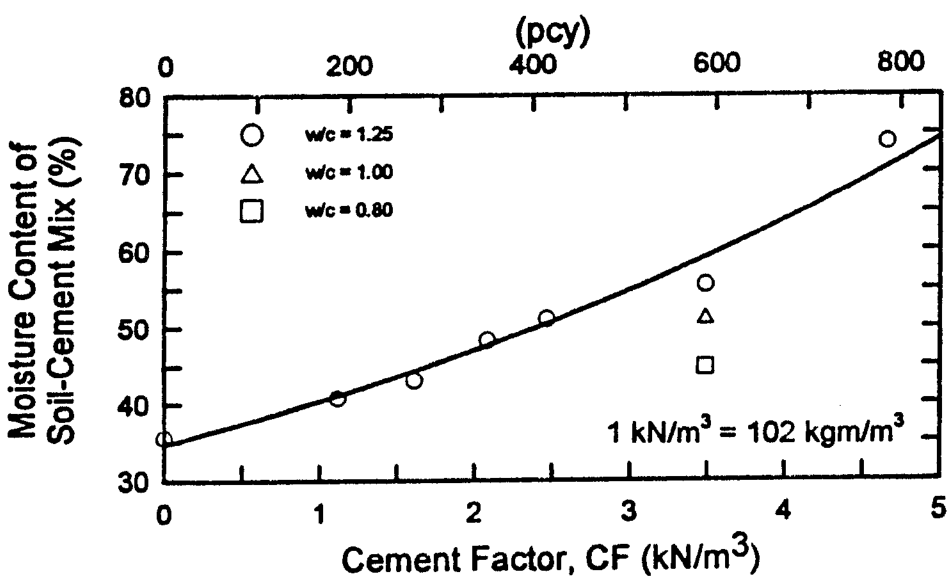


Figure 178. As-mixed moisture content versus cement factor (O'Rourke et al., 1997).

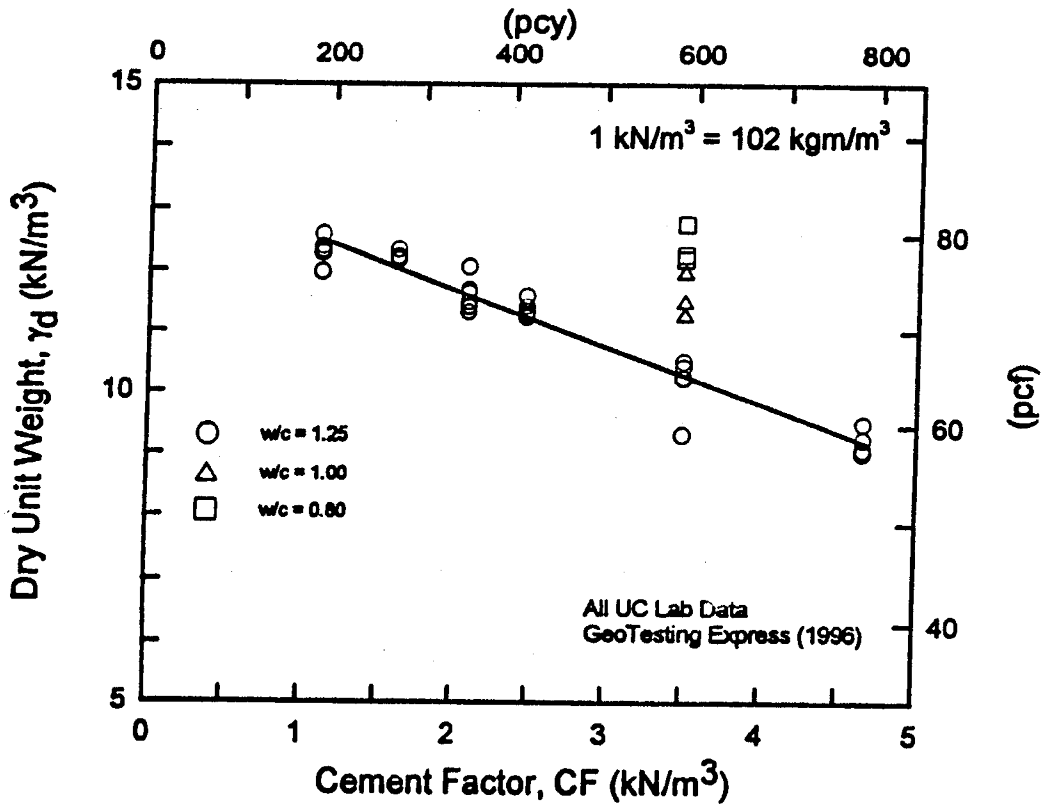


Figure 179. Dry unit weight versus cement factor (O'Rourke et al., 1997).

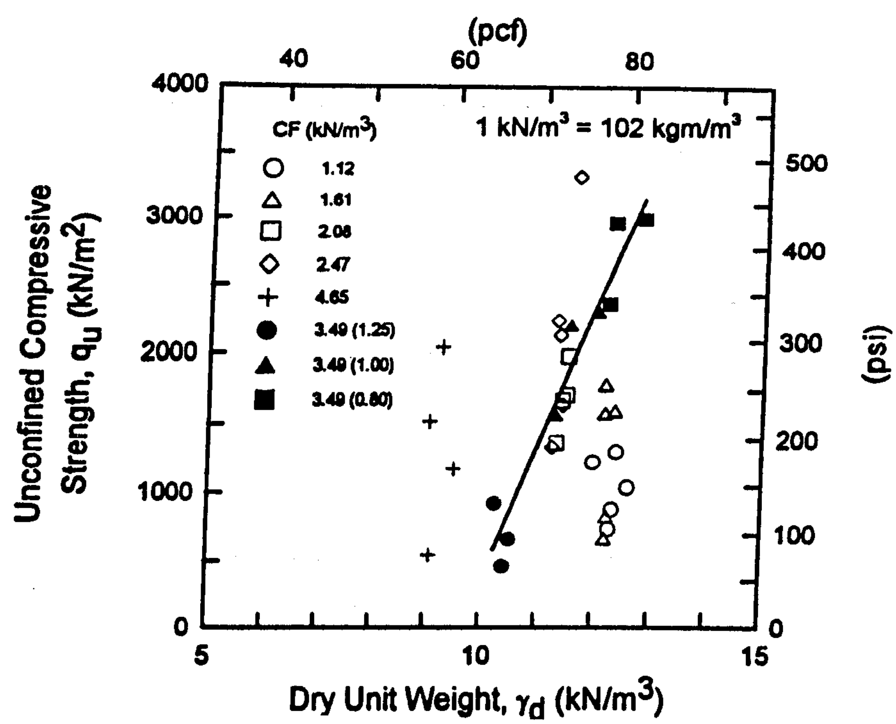


Figure 180. Unconfined compressive strength versus dry unit weight (O'Rourke et al., 1997).

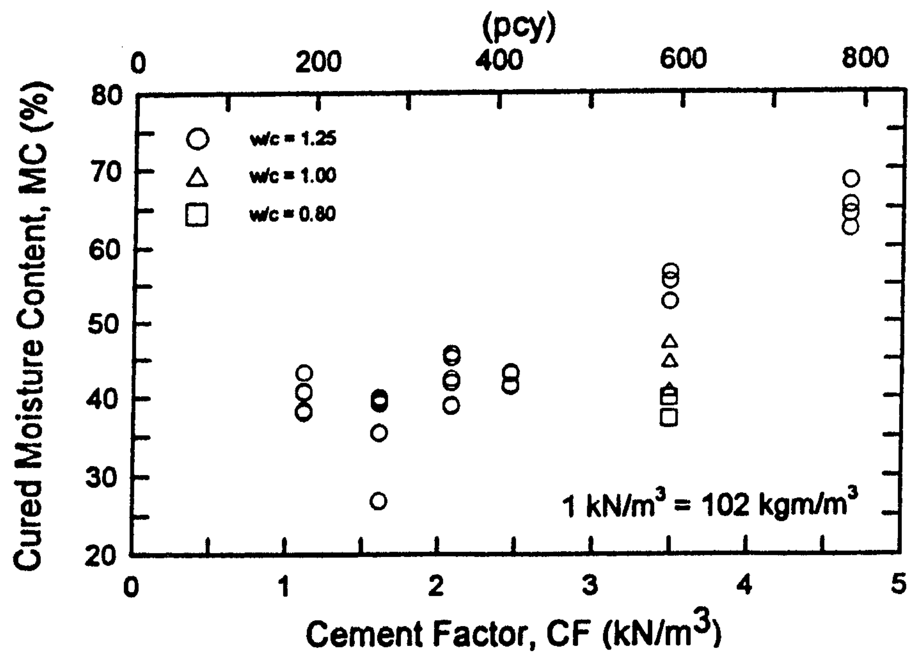


Figure 181. Cured moisture content from unconfined compression tests versus cement factor (O'Rourke et al., 1997).

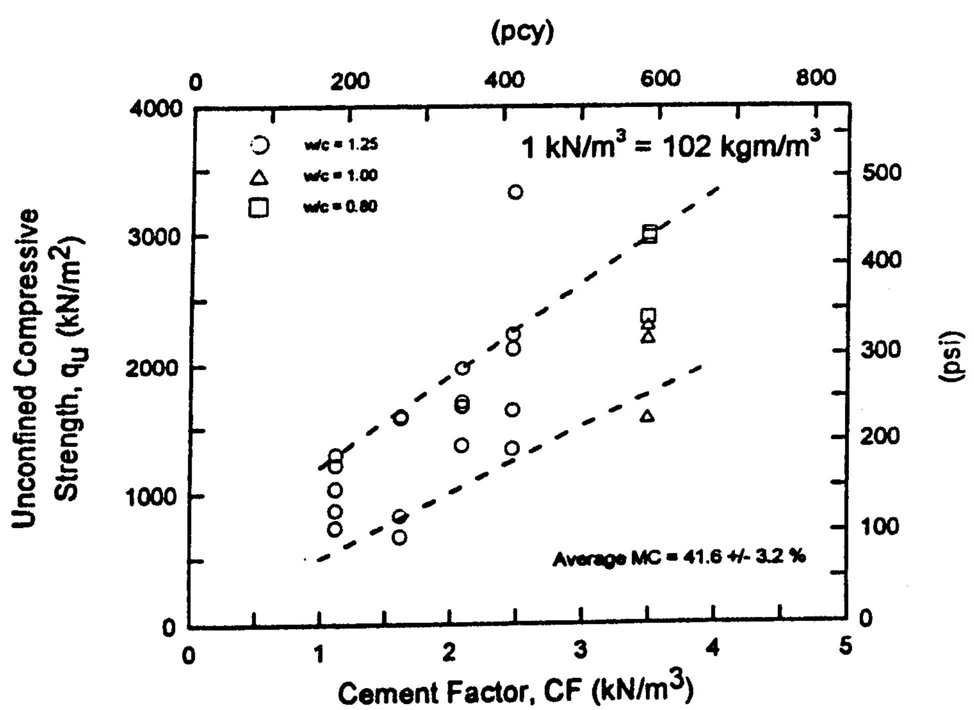


Figure 182. Unconfined compressive strength versus cement factor for average cured moisture content of approximately 40% (O'Rourke et al., 1997).

Table 42. Estimated unconfined compressive strengths for cured moisture contents of approximately 40% (O'Rourke et al., 1997).

CF			q_u	
(kN/m^3)	(pcy)	w/c	(kN/m^2)	(psi)
1.0	172	1.25	500 - 1200	75 - 175
2.0	344	1.25	1000 - 1900	145 - 275
2.5	430	1.25	1250 - 2250	180 - 325
3.0	515	0.80 - 1.00	1500 - 2600	215 - 375
4.0	687	0.80 - 1.00	2000 - 3300	290 - 475

Even with close controls, significant field variability is most probable because of variations of in-place DMM composition. Quality control programs need sufficient flexibility to respond to variable characteristics and avoid restrictive penalties for occasional low specimen strengths.

Laboratory data show reasonable relationships between early strength (3- to 7-day U.C.S.) and strengths at cure times of 2 to 4 weeks. Field testing specimens at relatively short cure times (approximately 3 days) appeared to be feasible so that rapid assessment of the mix characteristics in-place can be made.

Al-Tabbaa and coworkers (Al-Tabbaa et al., 1998; Al-Tabbaa and Evans, 1999a) described a series of field tests related to their laboratory work as described in Section 4.3 above. First, a prototype auger, 0.6-m-diameter and 2.4-m-long was developed (WRS) to mix the 7 grouts developed in the laboratory test program (Section 4.3). A total of 23 overlapping soil-grout columns were constructed treating 14 m^3 of soil. Seven sets of small paddles were located above a single auger turn. Time for penetration was 10 minutes at 30 rpm. Data are shown in Figure 183 and Table 43. Cores of 75 to 150 mm in diameter were obtained at 28 and 45 days (single tube probably). The best cores were found with Mix G.

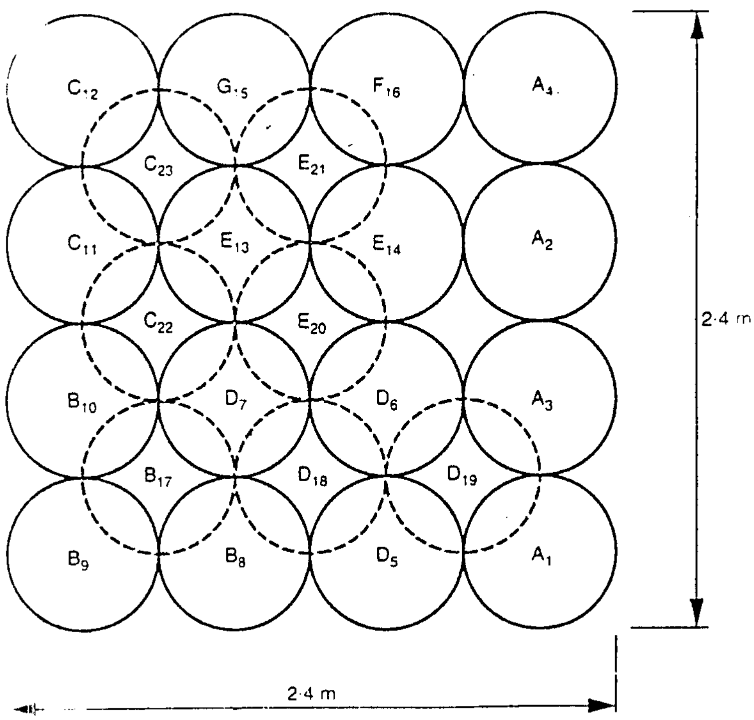


Figure 183. Plan of the constructed columns (Al-Tabbaa and Evans, 1999b).

Table 43. Details of the soil-grout mixes applied in the site trial (Al-Tabbaa and Evans, 1998).

Mix	Cement:PFA:Lime: Bentonite	Water: Dry Grout	Soil:Grout	Soil: Dry Grout
A	2 : 8 : 0 : 0	0.42 : 1	5 : 1	7 : 1
B	3 : 8 : 0 : 0	0.42 : 1	5 : 1	7 : 1
C	2.5 : 8 : 0.4 : 0	0.42 : 1	5 : 1	7 : 1
D	3 : 8 : 0.1 : 0	0.42 : 1	5 : 1	7 : 1
E	2.5 : 8 : 0.4 : 0	0.42 : 1	3.5 : 1	5 : 1
F	2.5 : 8 : 0.4 : 0	0.30 : 1	3.9 : 1	5 : 1
G	8 : 0 : 0 : 0.8	1.6 : 1	2.8 : 1	7.3 : 1

Regarding U.C.S. and dry density (Table 44), data at 60 to 70 days were about 2.5 times higher than 28-day laboratory values. The densities were 14% higher than the laboratory values and so the greater strength was most probably due to better in situ compaction. Regarding durability (Table 45), all samples survived the wet-dry durability test with total cumulative dry mass loss of less than 2%. Freeze-thaw resistance followed increased cement factor and bentonite content. Regarding permeability (Table 46), field values were similar to laboratory values, and were lowest with highest cement factor and bentonite concentration (0.5×10^{-9} to 3×10^{-9} m/s).

Cores were also tested at 2, 14, and 28 months (treated made ground – using data from Table 47). Regarding U.C.S., data from 75- to 100-mm diameter cores are shown in Figure 184. A broadly linear increase in U.C.S. is shown with time ascribable to continued hydration of the cementitious materials (by wetting). Strengths are predictably related to cement content and volume ratio. Regarding durability (Table 48), the data indicated resistance remained consistently high for wet-dry tests (<0.6% loss). For freeze-thaw (Table 49), the susceptibility decreased with time, and mixes with high lime, high cement factor, and bentonite performed better. Regarding permeability (Figure 185), a reduction in permeability at 14 months is ascribed to continued hydration of the cementitious materials (and so a gain in U.C.S.) Regarding compressibility (Figure 186), the data show that compressibility of the samples was not affected by age.

Based on these and other tests, only Mix G satisfied all the design criteria and had the best time-related performance.

In another paper, Al-Tabbaa and Evans (1999b) described an environmental project, in which data were compared between laboratory and field studies. They noted that the treatment of waste containing organics has been far less successful than when treating inorganics because some organics interfere with the hydration and setting processes of the cement. In such cases, special additives are used to provide the link between the organic waste and the cement (e.g., natural and chemically modified –organophilic – bentonite clays). The soils contained two retarders: lead and oils, and the goal was to use a low cement factor to produce a "soft, rock-like material" for logistic and economic reasons.

Table 44. Unconfined compressive strength, dry density, and leachate pH of the in situ treated ground (Al-Tabbaa and Evans, 1998).

MIX	U.C.S. (KN/M^2)	DRY DENSITY (KN/M^3)	LEACHATE PH
A	990	1757	9.6
B	1332	1778	10.1
C	1231	1809	10.9
D	1274	1717	10.3
E	1480	1711	10.8
F	1335	1745	10.9
G	1365	1775	10.7

Table 45. Percentage dry mass loss in the durability test on the in situ treated ground (Al-Tabbaa and Evans, 1998).

MIX	WET – DRY	FREEZE – THAW AT 0°C	FREEZE – THAW AT -10°C	FREEZE – THAW AT -20°C
A	-1.5	-1.3	-1.1	28.5
B	1.8	-0.7	1.7	25.4
C	1.9	0.2	0.9	23.5
D	-1.2	-2.3	0.1	26.9
E	0.4	0.04	-0.1	11.5
F	0.5	-0.1	0.4	10.5
G	-0.4	0.9	-1.5	1.1

Table 46. Permeability and compressibility of in situ treated ground (Al-Tabbaa and Evans, 1998).

MIX	PERMEABILITY (x 10^{-9} M/S)	COEFFICIENT OF VOLUME COMPRESSIBILITY (x 10^{-6} M^2/KN)
A	2.64	3.20
B	0.69	2.85
C	2.56	2.60
D	2.21	2.63
E	1.99	3.20
F	0.64	2.30
G	0.70	2.70

Table 47. Details of the soil-ground mixes in the cored samples (Al-Tabbaa and Evans, 1999a).

mix	cement : pfa : lime : bentonite	water : dry grout	soil : grout	soil: dry grout
A	2 : 8 : 0 : 0	0.42 : 1	5 : 1	7 : 1
B	3 : 8 : 0 : 0	0.42 : 1	5 : 1	7 : 1
C	2.5 : 8 : 0.4 : 0	0.42 : 1	5 : 1	7 : 1
D	3 : 8 : 0.1 : 0	0.42 : 1	5 : 1	7 : 1
E	2.5 : 8 : 0.4 : 0	0.42 : 1	3.5 : 1	5 : 1
F	2.5 : 8 : 0.4 : 0	0.30 : 1	3.9 : 1	5 : 1
G	8 : 0 : 0 : 0.8	1.6 : 1	3.7 : 1	9.7 : 1

pfa = pulverized fuel ash.

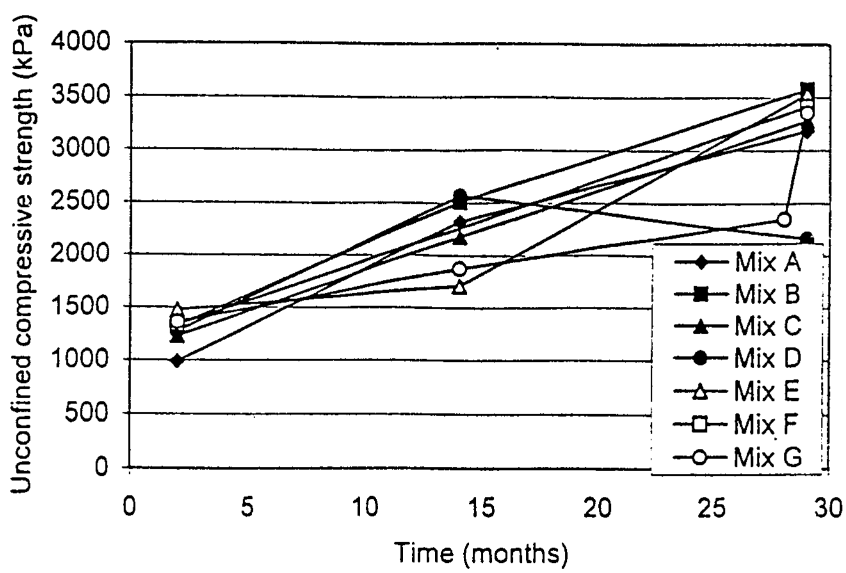


Figure 184. Time-related U.C.S. of the in situ treated ground (Al-Tabbaa, 1999).

Table 48. Development with time of wet-dry durability behavior of the in situ treated ground (Al-Tabbaa and Evans, 1999a).

	% mass loss		
Mix	2 months	14 months	28 months
A	-1.5	0.59	0.33
B	1.8	0.04	0.05
C	1.9	0.03	0.01
D	-1.2	0.05	0.06
E	0.4	0.05	0.01
F	0.5	0.03	0.02
G	-0.4	0.08	-0.03

Table 49. Development with time of freeze-thaw durability of in situ treated ground (Al-Tabbaa and Evans, 1999a).

	% mass loss					
	2 months			14 months	28 months	
mix	6 cycles at 0°C	6 cycles at −10°C	6 cycles at −20°C	6 & 6 cycles at −10°C	6 & 6 cycles at −10°C	6 & 6 cycles at − 20°C
A	-1.3	-1.1	28.5	fac 5	22.4 & fac 8	
B	-0.7	1.7	25.4	29.6 & fac 7	10.5 & fac 11	
C	0.2	0.9	23.5	fac 6	0.9 & 1.95	3.0 & 13.9
D	-2.3	0.1	26.9	27.0 & fac 7	20.1 & fac 9	
E	0	-0.1	11.5	20.5 & fac 11	0.2 & 1.48	7.3 & 15.0
F	-0.1	0.4	10.5	13.5 & fac 11	3.8 & fac 11	
G	0.9	-1.5	1.1	20.8 & fac 11	0.37 & 0.66	9.3 & 21.9

fac = failed at cycle

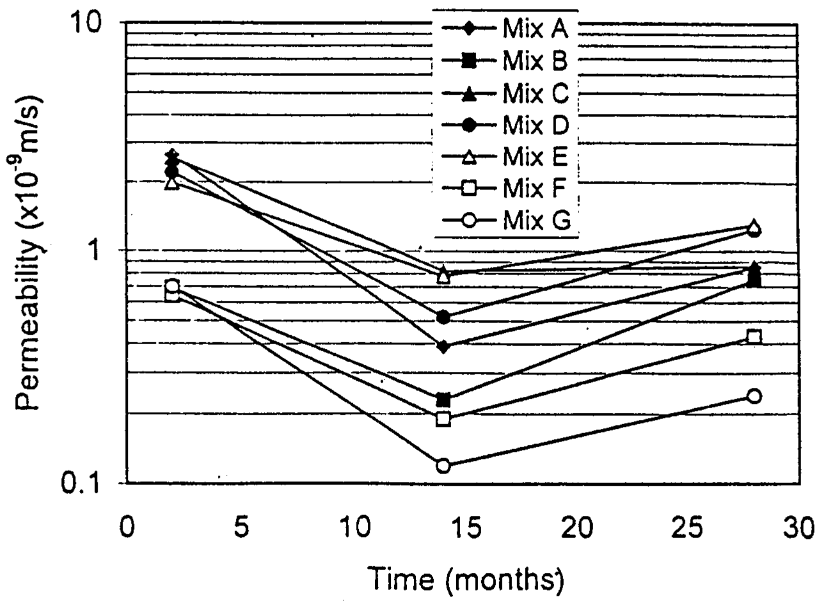


Figure 185. Time-related permeability of the in situ treated ground (Al-Tabbaa, 1999a).

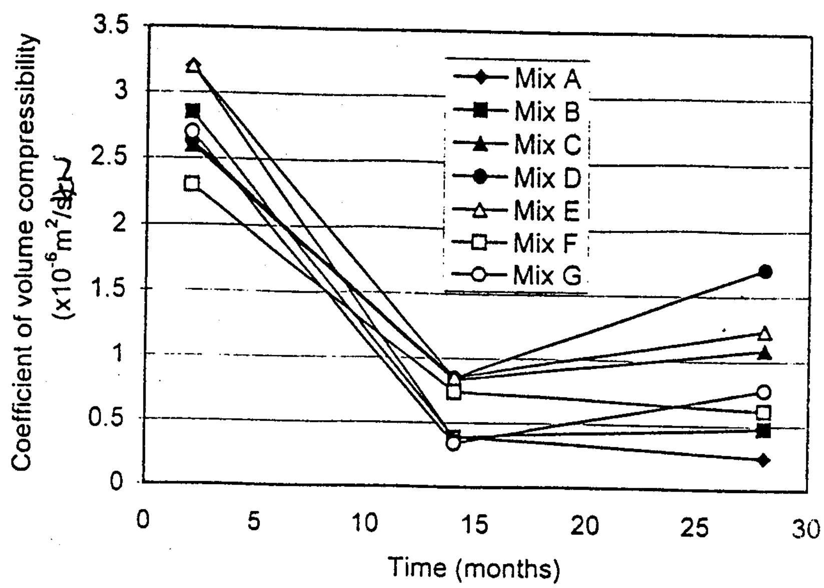


Figure 186. Time-related coefficient of volume compressibility of the in situ treated ground (Al-Tabbaa, 1999a).

Site data are shown in Table 50.

Table 50. Ground conditions in the trial pit (Al-Tabbaa and Evans, 1999b).

Soil description	Depth
Made ground: fine to coarse sand	0·10–0·40
Made ground: clayey silty fine to coarse sand	0·40–0·87
Made ground: very silty slightly sandy clay	0·87–1·62
Made ground: spongy peat	1·62–1·72
Fine to coarse sand with much fine to coarse flint gravel	1·72–2·00
Fine to coarse flint gravel with much medium and coarse sand	2·00–2·30

The gravel continue to -5 m, and are underlain by clay. The ground water level was at -2 to -2.3 m. Testing was conducted as per Table 51. Approximately 1/10 scale shafts of WRS (Auger 2) and WRE (Auger 1) type were used, and the soils were appropriately modeled for granulometry, composition, and mechanical properties (Figures 187 and 188). Grout Mix G (Table 52) was used. Manually mixed and model mixed samples were prepared for testing at 60 days to compare with the in situ program.

Regarding the findings related to geotechnical parameters, the dry densities (Tables 53 and 54) produced by both soils were similar in the laboratory and field, with manually mixed samples were somewhat lower. The U.C.S. data varied between soils and augers. The WRS (predictably) produced higher strengths than the WRE by a factor of 2. The latter being also more variable. Site data were similar, in the made ground to the WRS data and manual data. The authors believe this confirms that the smallest 25% of particle sizes control soil strength. For sand and gravel, U.C.S. for manual (around 1.8 MPa) compared with about 0.5 MPa for mixed samples (taken as due to higher moisture content of latter soils prior to mixing).

Regarding wet-dry and freeze-thaw durability (Figure 189), all manual and mixed soils survived 12 wet-dry cycles with up to 0.8% average cumulative dry mass loss, equivalent to site data. For freeze-thaw testing, the manual samples performed very poorly, while made ground

Table 51. Summary of mix design criteria used and corresponding test method (Al-Tabbaa and Evans, 1999b).

Material property	Criterion	Test method and reference	Modification to standard test procedure or comments
UCS (soaked)	350 kPa (US Environmental Protection Agency, 1986)	ASTM (American Society for Testing and Materials, 1995a)	Different-size samples used, all with height-to-diameter ratio of 2, correlated to give equivalent UCS of 100 mm diameter samples
Leachability	Leachate concentration to be 100 times drinking-water standards	Two tests were applied: TCLP (Federal Register, 1986) and NRA (Lewin *et al.*, 1994) Lead concentration (Department of the Environment, 1976) Mineral-oil concentration (Greenberg *et al.*, 1992)	In the TCLP test, extraction fluid pH was $4{\cdot}9 \pm 0{\cdot}2$ In the NRA test, extraction fluid pH of 7 was used owing to inconsistent effect of carbonation required in the test
Leachate pH	7–11 (Harris *et al.*, 1995a)	Performed at the end of leaching tests above	
Durability: wet–dry, freeze–thaw	Pass ASTM tests of 12 cycles	ASTM (American Society for Testing and Materials, 1995b, c)	Sample sizes used were 95–100 mm in diameter and 50 and 100 mm high (for comparison purposes) Freeze–thaw temperature used was −10°C instead of specified −20°C
Permeability	$< 10^{-9}$ m/s (as for clay liners).	Flow pump method (Head, 1992)	Samples of 100 mm diameter and up to 120 mm high tested. Test performed in triaxial cells. Same hydraulic gradient of 20 applied to all samples
Compressibility	Coefficient of volume compressibility similar to that of soft rock	Standard oedometer test (British Standards Institution, 1992)	Two vertical-stress increments of 1200 kPa were applied to obtain the results quoted
Microstructural analyses: XRD and SEM			For XRD, Piker Precision Powder Diffractometer was used on specimens from centre of UCS-tested samples For SEM, JEOL 5410 machine was used on gold-coated samples

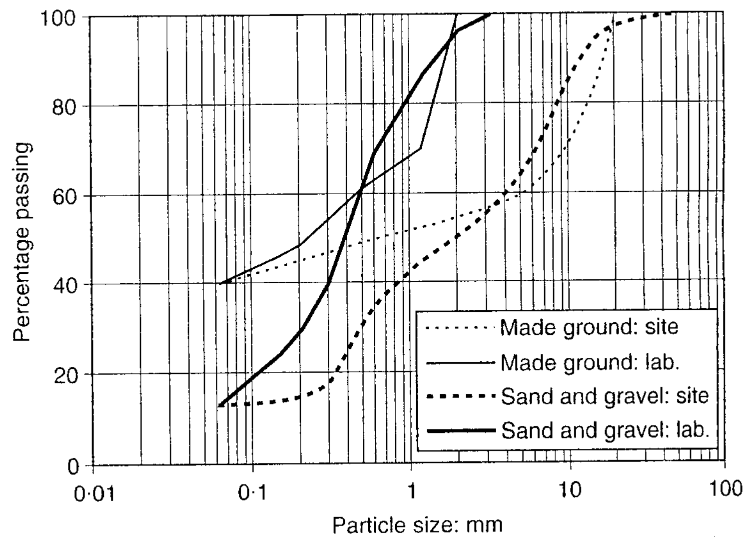


Figure 187. Particle size distribution of the two site and model soils (Al-Tabbaa and Evans, 1999b).

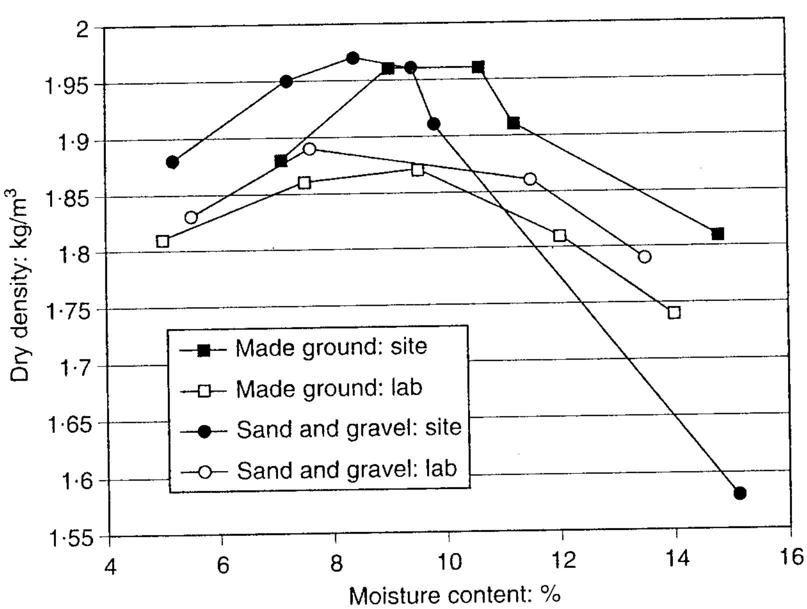


Figure 188. Compaction behavior of the uncontaminated site and model soils

Table 52. Details of in situ soil-ground mixes (Al-Tabbaa and Evans, 1999b).

Mix	Cement : PFA* : lime : bentonite	Water : dry grout	Soil : grout	Soil : dry grout
A	2 : 8 : 0 : 0	0.42 : 1	5 : 1	7 : 1
B	3 : 8 : 0 : 0	0.42 : 1	5 : 1	7 : 1
C	2.5 : 8 : 0.4 : 0	0.42 : 1	5 : 1	7 : 1
D	3 : 8 : 0.1 : 0	0.42 : 1	5 : 1	7 : 1
E	2.5 : 8 : 0.4 : 0	0.42 : 1	3.5 : 1	5 : 1
F	2.5 : 8 : 0.4 : 0	0.30 : 1	3.9 : 1	5 : 1
G	8 : 0 : 0 : 0.8	1·6 : 1	3.7 : 1	9.7 : 1

* PFA, pulverized fuel ash.

Table 53. Dry density and U.C.S. of the treated model soils (Al-Tabbaa and Evans, 1999a).

Type of sample	Made ground		Sand and gravel	
	Dry density: kg/m^3	UCS: kN/m^2	Dry density: kg/m^3	UCS: kN/m^2
Auger 1	1798 ± 9%	675 ± 40%	1763 ± 7%	300 ± 53%
Auger 2	1778 ± 8%	1125 ± 34%	1754 ± 16%	484 ± 27%
Auger 2: permeability sample	1783 ± 5%	1362 ± 21%	1769 ± 12%	579 ± 39%
Manually mixed	1682 ± 3%	1211 ± 26%	1697 ± 6%	1887 ± 32%
Manually mixed: permeability sample	1680 ± 1%	1392 ± 30%	1680 ± 1%	1755 ± 42%

Table 54. Properties of site trial soil-grout Mix G at 2 months after treatment (Al-Tabbaa and Evans, 1999b).

Dry density: kg/m^3	1775 ± 3%
UCS: kN/m^2	1365 ± 16%
Wet-dry durability: % mass loss	-0.4
Freeze-thaw durability: % mass loss	
6 cycles at −10°C	-1.5
6 cycles at −20°C	1.1
Permeability: 10^{-9} m/s	0.7 ± 14%
Compressibility: 10^{-6} m^2/kN	2.7 ± 5%
TCLP leachate pH	10.7 ± 0.2

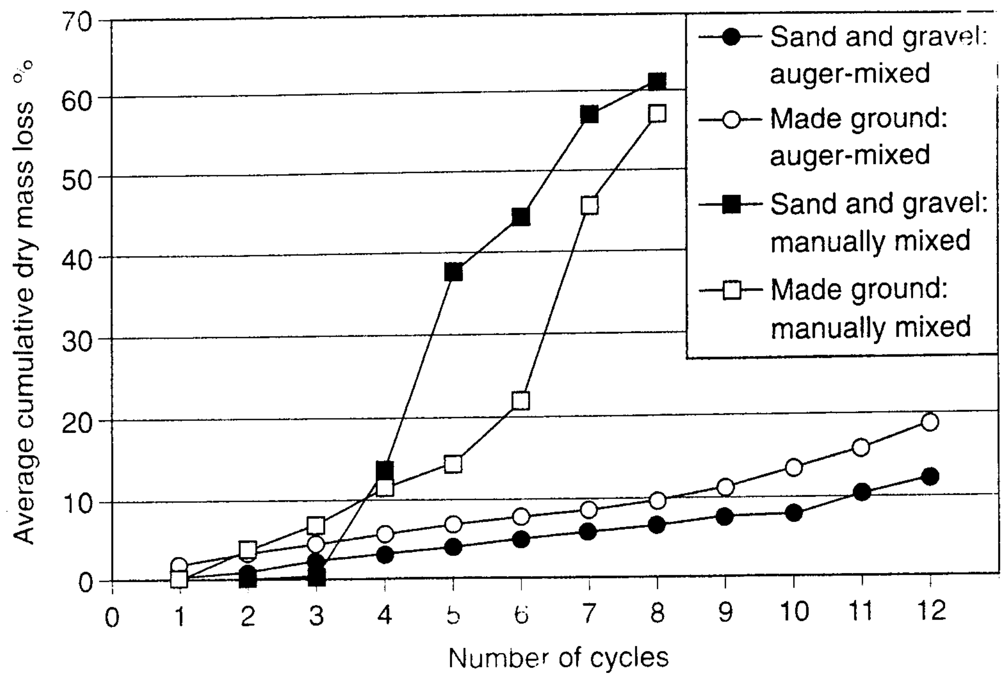


Figure 189. Freeze-thaw test results of the model soils (Al-Tabbaa and Evans, 1999b).

Table 55. Permeability and coefficient of volume compressibility of the treated soils (Al-Tabbaa and Evans, 1999b).

Sample Type	Permeability (10^{-9} m/s)		Compressibility (10^{-6} m^2/kN)	
	Made ground	Sand and gravel	Made ground	Sand and gravel
Auger 2	0.82 ± 20%	4.60 ± 33%	4.13 ± 18%	3.02 ± 22%
Manually mixed	0.38 ± 31%	1.24 ± 46%	2.44 ± 35%	1.93 ± 48%

performed better than sand and gravel. This is ascribed to a combination of contributing factors including curing conditions, mixing method, and different water contents.

Regarding permeability and compressibility (Table 55), the made ground samples had lower permeability, predictably, and the manual samples were lower than the mixed samples. Site made ground cores gave very similar data to the laboratory mixed samples.

Coefficients of volume compressibility achieved by both soils were similar, with slightly lower values achieved in both cases by manual mixed samples. Site data gave a slightly lower compressibility (2.7×10^{-6} m^2/s).

The authors concluded that laboratory "auger mixing can simulate in situ conditions in a way which is not possible by manual mixing, such as the ground water condition and the mixing process."

4.5 General Overview

This chapter illustrates the huge – and growing – volume of data that exists regarding the properties of soils treated by "wet" methods. The main goal of the chapter is to present and compare data on specific aspects from different sources. However, it must be recognized that details presented from each data source are particular to the exact circumstances enjoyed or created by that source, including the particular soil type, mix design, installation method, testing methodologies, and so on. Often key details, especially on testing procedures, are omitted from the published account, for brevity, even though they may have had a direct impact on the data produced. When reviewing the following general observations, therefore, the reader must bear these caveats in mind, and when searching for a *specific* piece of information, should revert to the published details of the original source.

The data cited in this chapter permit the following general observations to be made.

a) The most common materials used to provide slurries for geotechnical applications are (in addition to water – both fresh and salt) portland cement, bentonite, slag cement, clay, flyash, lime, gypsum, sand, and kiln dust. Small (say, less than 1.5%) amounts of additives may be used to enhance fluid and set properties whereas the use of various "industrial byproducts," more common in "dry" methods, is very rare in wet mixing. Careful experimentation has been conducted with various proportions of these materials to satisfy economic and technical goals. A particular case in point is the use of FGC methodologies, knowing that excessive amounts of gypsum can be deleterious to treated soil properties.

b) Water/cement ratios are typically in the range of 0.8 to 1.2 (by weight) although the band extends to 0.5 to 2.5 to satisfy more extreme circumstances. Ratios tend to be higher in cohesionless soils, and during the penetration phase of certain mixing techniques.

c) Cement factors generally range from 100 to 400 kg/m^3 or, in terms of apparent cement factor, 10 to 30%.

d) Volume ratios vary greatly with mixing method type but are typically in the range of 15 to 50%.

e) Unconfined compressive strengths are dependent on many factors, including:

- Soil type (proportional to initial stiffness or density and grain size; inversely proportional to moisture and fines contents, humic content, pH). Regarding soils of high organic content, it has been found that humus content is a most significant factor on strength development: little to no improvement was gained at a content over 0.8% (with either portland or slag cement and cement factor of 150 kg/m^3), organic contents over 6%, pH values less than 5, and loss ignitions greater than 10%. Such conditions interfere chemically with the hydration processes of the treated soil, and are often not, from the physical viewpoint, amenable to efficient mixing, or contributing appropriate mixed soil constituents.
- Cement factor (proportional) perhaps apparent only after 7 days of curing.
- Water/cement ratio (inversely proportional).
- Air entrainment (inversely proportional).
- Homogeneity of treatment. Laboratory mixing tests have confirmed that the efficiency of mixing (i.e., blade revolutions per unit length of column and type of blade) strongly influences U.C.S. This is simply a reflection of the work done on the soil to improve the soil/binder dispersion.

- Choice of binder "substitutes" for cement will lower strength (e.g., ash, bentonite) although they may satisfy other performance goals: the calcium content of the slurry appears to be a strong control over strength. Thus, 28-day U.C.S. strengths (from cores) can vary (or be varied) from well under 1 MPa to 10 MPa or more although it remains challenging to produce a given "designer soil" (Silvester, 1999) given all the contributory variables. Considerable strength variation (coefficient of variability of 40%) may be expected on any one site in apparently constant or uniform circumstances with even the best degrees of quality control.

f) Permeabilities in the range of 10^{-7} to 10^{-8} m/s are routinely achievable but values as low as 10^{-10} m/s are attainable with appropriate mix designs (and higher cost). Increasing the cement factor, bentonite content, volume ratio, and time decrease permeability to greater or lesser extents, as does the initial fines content of the soil.

g) Rate of gain of unconfined of unconfined compressive strength is also dependent on a number of interactive factors, but:

- Is slower in cohesive soils.
- Is slower with higher w/c ratios (and may stop in low-strength samples (less than 1 MPa) after 28 days, depending on the slurry formulation.
- 28-day strengths are about 1 to 2 times 7-day values and at least 1.6 times 3- or 4-day values.
- 56- or 60-day strengths can be 1.4 to 2.5 times 28-day values.
- Long-term strength, due to pozzolanic reactions, can continue well after 56 days (to 730 days measured), although rate of gain of strength is more rapid in in situ samples (i.e., reflects curing conditions).

h) Tensile strengths are reported as 10 to 20% U.C.S. (direct uniaxial test) and 8 to 14% U.C.S. (indirect splitting test). Reportedly, 15% is regularly used for CDM design. Ratios appear to decrease with increasing moisture content.

i) Undrained shear strengths appear to be 40 to 50% U.C.S. (U.C.S. values less than 1 MPa) and 30 to 35% in stronger ground (decreasing to 20% above U.C.S. of 4 MPa).

j) Many authors have reported on (or repeated earlier findings on) the relationship between elastic modulus (typically recorded as E_{50}) and U.C.S. Broadly, the ratios vary from 50 to 1350 with most reports being in the 100 to 1000 range. It may be broadly concluded that

- The actual E_{50} value is influenced by the same factors that control strength (especially water content), while increasing sand content also increases the ratio.
- The ratio is lower (say 50 to 350) in material of lower strengths (say, up to 2 MPa) but higher (200+) in higher strength material.

No researchers appear to have reported on possible changes in the E-value itself, or its ratio to U.C.S., with time or curing conditions – both potentially important influences.

k) Poisson's Ratio may be as high as 0.49 (undrained) but is typically 0.19 to 0.45 otherwise. A value of 0.26 is routinely used in practice by one designer in particular.

l) Durability is generally linked to the same factors that control strength and permeability, with the overall system moisture content being particularly important. The rate of deterioration under exposure to marine conditions reportedly increases with the log of time. The addition of lime, bentonite, and higher cement factors each can improve durability.

m) Treated soil properties can vary considerably over relatively small distances, depending on installation methodologies and even subtle variations in soil characteristics. Therefore, "point" sampling techniques (such as coring) may not accurately reflect overall in situ conditions, especially as related to overall design and performance concepts.

n) The ratio between U.C.S. from field cores and laboratory samples can vary substantially, being 20 to 100% for clayey soils to 4 MPa and more than 250% for sandy soils over 4 MPa (large terrestrial projects). The ratio for marine projects is reportedly 100%. A typical range for treated clayey soils of strengths up to 3.5 MPa is 20 to 50%. Core strengths are generally higher than wet grab samples by up to 50%.

5. PROPERTIES OF GROUND TREATED BY "DRY" METHODS

This chapter describes data relating to ground treated by the three "dry" or DRE methods referred to in Figure 2, namely DJM, Lime Cement Columns, and Trevimix. A great deal of published data are available on the Lime Cement Column Method, having been virtually a national research project in Sweden and Finland for well over 20 years, somewhat less (in English) on DJM, and relatively little on Trevimix, reflecting its less active application in its particular country of origin (Italy).

5.1 General Observations

Table 1 summarizes the main features of these three methods. Traditionally, the DRE methods have been used to treat relatively soft, compressible, or liquefiable materials, often (in the case of cohesives) containing high organic contents. These methods are usually feasible in soils with over 60 percent moisture content (either natural or preconditioned), with records of successful treatment of soils in Scandinavia and Japan of over 200% moisture content. Whereas the Japanese and Italian trends have been toward the use of large-scale equipment capable of producing relatively large diameter columns (1 m and over) to considerable depths (33 m) and providing unconfined compressive strengths in excess of 0.5 MPa (well in excess of that in sands), the Scandinavians have generally operated with smaller and lighter equipment producing shorter, smaller diameter columns of unconfined strengths below 0.4 MPa. Each approach, of course, has been driven by the soils, binders, and project goals particular to the respective country.

Dry methods typically use a lower cement factor than wet methods and naturally produce far less spoil, and so less wasted binder.

The newest of the dry methods has been developed in Italy (Trevimix). To date it has seen the fewest applications, with limited service in Italy and Thailand. Pagliacci and Pagotto (1994) compared this method with their corresponding "wet" technique, and stated:

- Better "mechanical characteristics" are obtained by using dry binders especially in cohesive soil. Wet methods lead to increased water content in the system, and so treated soil "characteristics" are 30 to 50% lower.
- Wet mixing is mechanically simpler (equipment and QA/QC controls) and so is preferable in "difficult" geographic areas.
- The use of dry methods produces minimal spoil or heave.

Originally, Trevimix was designed to treat very soft to soft cohesive soils (undrained shear strength less than 25 kPa), or very loose to loose cohesionless soils (N less than 4 to 6). However, later works (Pagliacci and Pagotto, 1994) indicate that fine-grained soils with undrained shear strength values of up to 200 kPa and cohesionless soils with N-values up to 25 can be successfully and economically treated.

Pagliacci and Pagotto did, however, encounter significant problems in the maintenance of the binder delivery plant, such as high levels of mechanical abrasion, the effects of humidity, and drifting calibration of the QA/QC equipment. Portland cement is the most popular choice as binder, with quick-lime (powdered or in 5-mm granules) only being added for treating very soft clays: cement has been found to give faster set, higher strength, lower deformability, greater ease in operation, and more consistent results than lime.

Cement factors typically vary from 150 to 300 kg/m^3 and strongly influence strength (Figure 190). This figure also indicates that, for equivalent apparent cement factors, dry cement produced significantly higher strength than cement slurry, or dry lime in the cohesive soils in question.

Although treated ground strengths of over 6 MPa have been recorded, more typical values of compressive strength for very soft clays are 0.36 to 2.5 MPa. Pavianni and Pagotto (1991) compared the results of cores taken at different distances from column axes, and also perpendicular to the axes (Figure 191). Predictably, samples closer to the axes were stronger (generally) while the perpendicular cores gave higher than average strengths (Section 5.3).

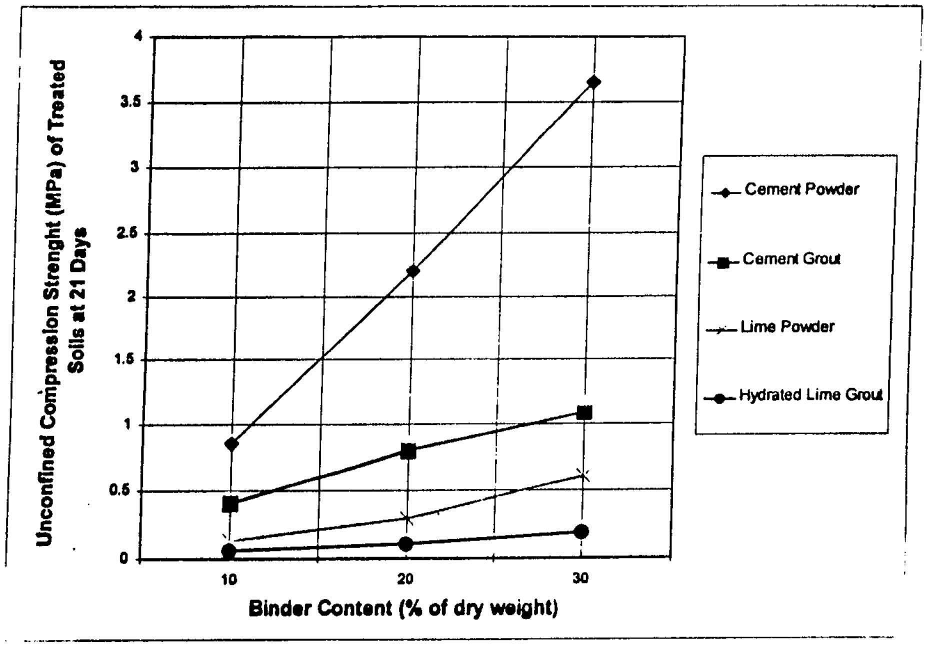


Figure 190. Unconfined compressive strength vs. binder content in cohesive soil (Catalano, 1998).

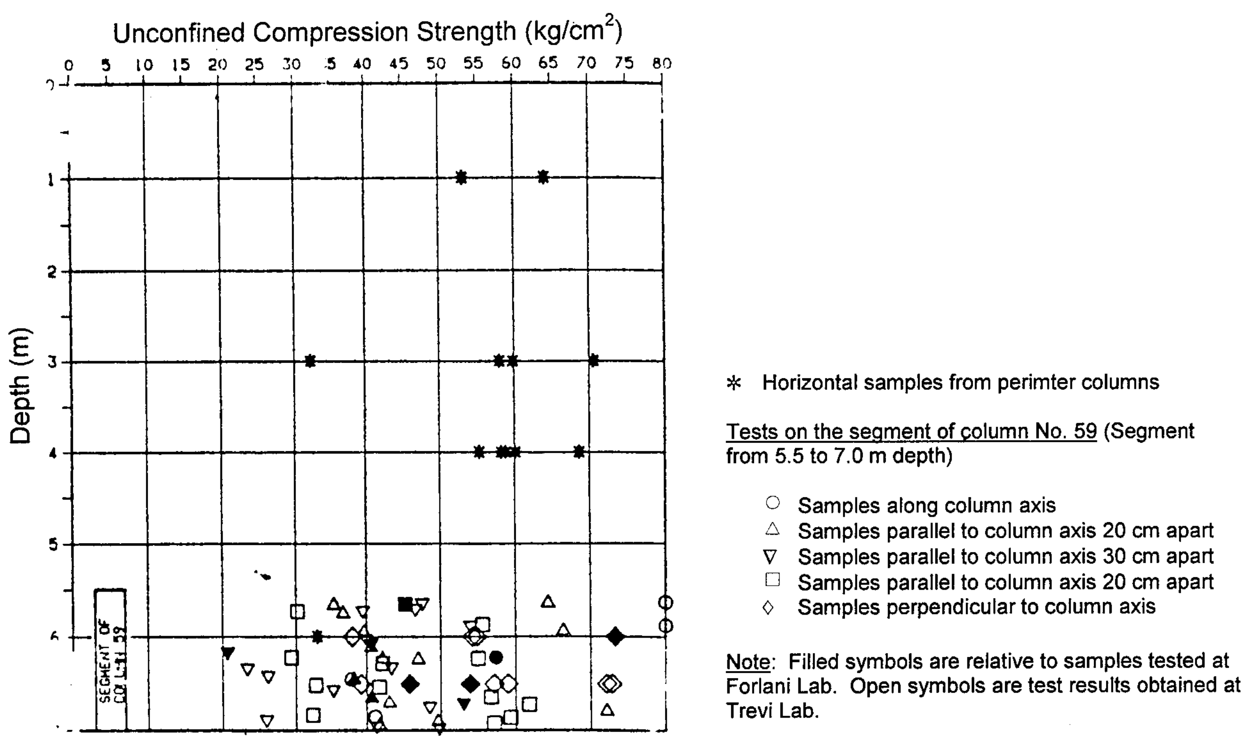


Figure 191. Strength characteristics from laboratory tests on samples recovered in directions parallel and perpendicular to the axis of the columns (Pavianni and Pagotto, 1991).

In Japan, the dry method is offered by the 70+ members of the DJM (Dry Jet Mixing) Association. The binder to be used depends on the nature of the soil and the design requirements, but in all cases has a 5-mm maximum particle size. Figures 192 and 193 show data on types of binders in use up to 1993, and 1996, respectively. Ordinary portland cement, slag cement, and "cement based reagent" constitute about 90% of the binders used (Yang et al., 1998). Flyash and gypsum are included in the small "others" category. Quicklime is used in marine clays with exceptionally high water contents. "Cement based reagents" have been found to be more effective than others for treating peats and organics (Yang et al., 1998).

Cement factors can be readily adjusted between different strata in the injection process, and are typically in the following ranges:

Cement binders	100 to 400 kg/m^3 200 to 600 kg/m^3	Sands and fine fills Peats and organics
Lime binder	50 to 300 kg/m^3	Soft marine clays

Unconfined compressive strengths of different types of treated soils are shown in Figure 194 and vary greatly (but in a controllable fashion) from 0.3 to 7 MPa in most cases. The DJM Association claims the rate of gain of strength regardless of soil type is faster than for wet methods. Figure 194 also indicates that 28-day strength is about 1.5 times 7-day strength for all types of soil. They also found from tests on a bridge abutment treatment that the field strengths of cored samples were only 25% of laboratory values.

No data have been found on permeability, but Yang et al. (1998) note that the permeability of treated ground is higher than that produced by wet methods and that such treatments may be "considered semi permeable."

As noted above, there is a wealth of experimental data available from Scandinavian sources, more recently supplemented by complementary information from U.S. projects such as I-15, Utah, and BART, San Francisco.

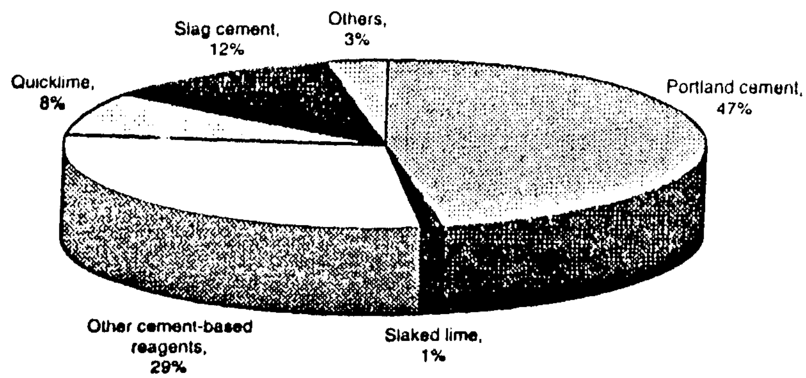


Figure 192. Types of binders used in Japan (DJM Association, 1993).

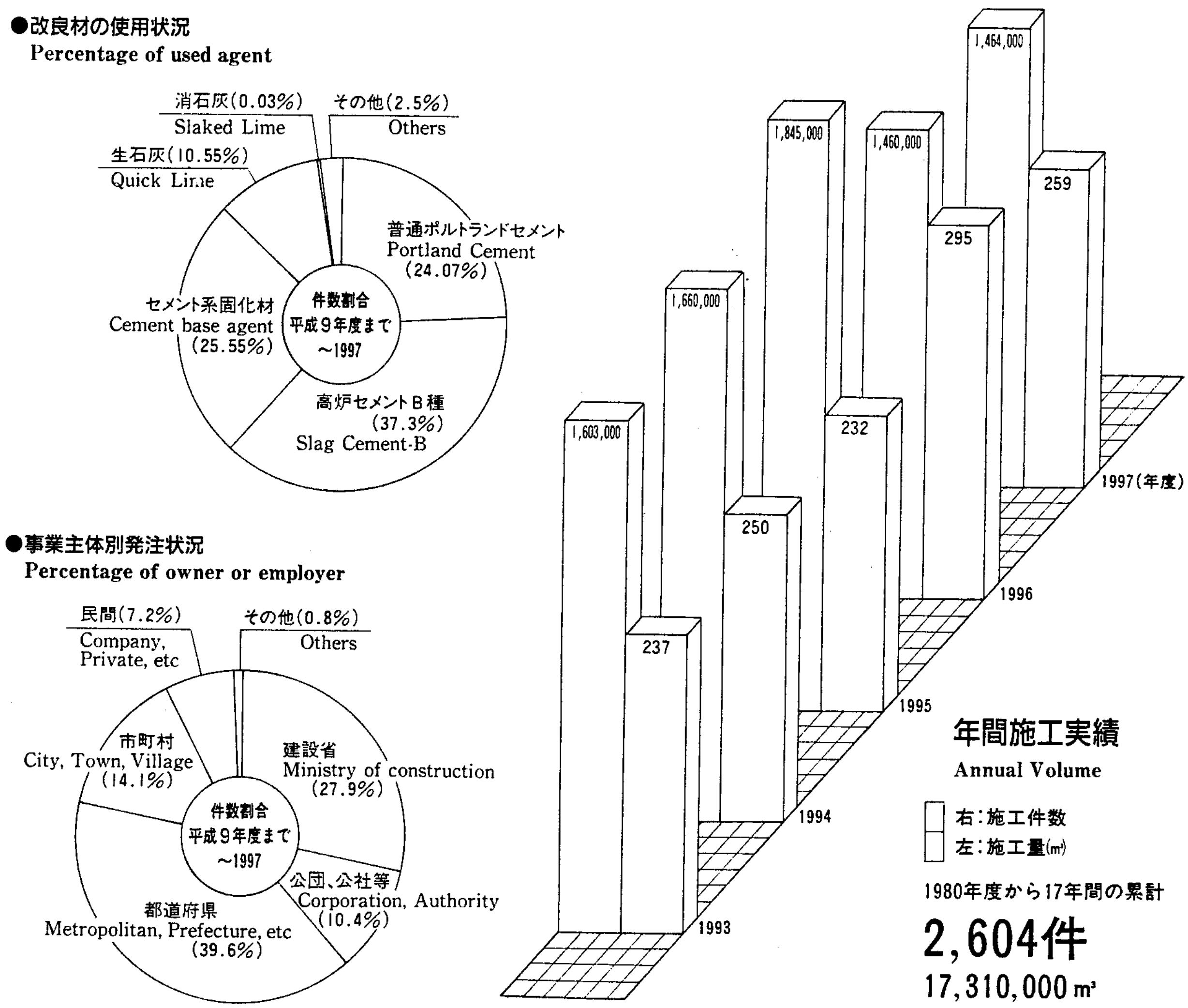
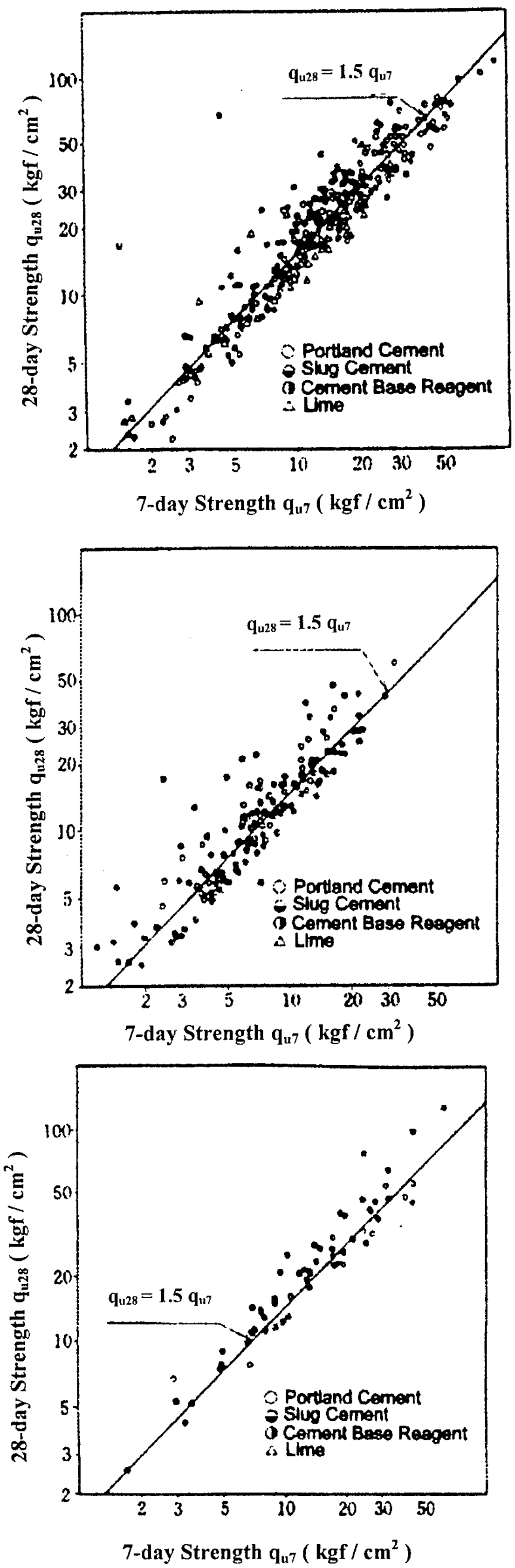


Figure 193. Data on DJM usage in Japan (1992-1996). [Left bar represents volume (m^3) and right bar represents number of projects] (DJM Association, 1996).



a) Cohesive Soils

b) Organic Soils

c) Sandy Soils

Figure 194. Relationship between 7-day and 28-day strengths (DJM Association, 1993).

Lime and cement are readily and economically available in Sweden. As shown in Figures 195 and 196, current practice uses almost exclusively lime and cement (50:50, but up to 75:25 in certain cases). Earlier work featured lime only (particle size less than 1.5 mm) and also lime/gypsum (75:25) blends, for enhanced long-term stability. The introduction of cement for treating organic soils has been found essential for acceptable strength and stability; cement has been found to contribute relatively rapid strength gain, while the lime promotes the longer term pozzolanic reaction, as described in Chapter 2.

In Finland, slag is cheap (given the iron and steel industrial base). Increasing use is therefore being made of proprietary binders (Figure 197) using slag, gypsum, and other products since lime and cement have to be imported, thus raising their cost. As in Sweden, the use of dry products (especially unslaked lime) is facilitated by the general absence of humidity problems.

Regarding properties of treated soil, Holm (1997) summarizes that such data depend on:

1. The soil, and especially the water and organic contents.
2. The type and amount of binder (below certain minimum contents there is no improvement in shear strength and E-value).
3. Temperature of curing (increased temperature gives more rapid strength).
4. Effective in situ stress
5. Age (Strength gain is most rapid in the first month because of immediate cement reactions with water in the soil. Lime develops more pozzolanic reactions giving a steadier, longer-term increase.)

The Scandinavians use shear strength typically measured by a probe as the most common measure of strength. The maximum design value for treated columns is in the range of 0.10 to 0.13 MPa, but this can vary in the field up to 1 MPa. The pressuremeter is increasingly being used to provide in situ test data, which commonly indicate shear strengths of 0.5 to 0.7 MPa using the typical range of binder factors and compositions. These binder factors range from 80

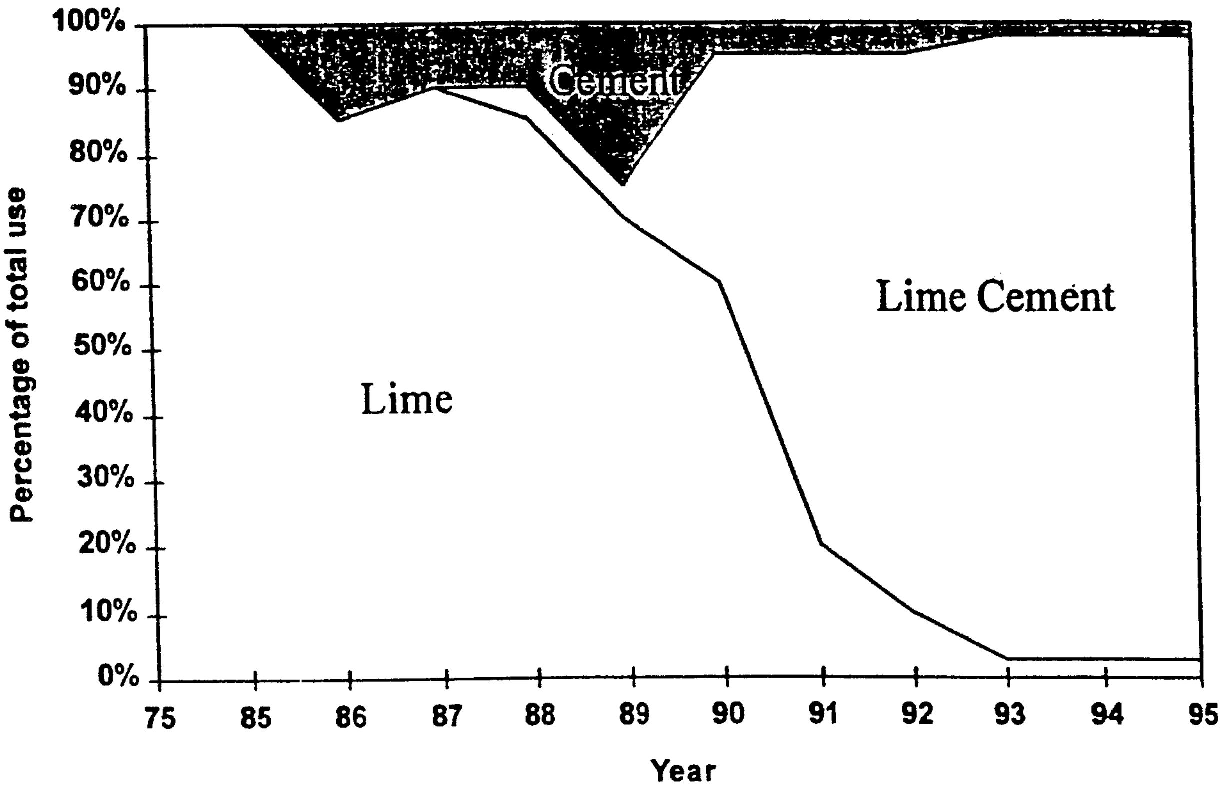


Figure 195. Changes in use of binders in Scandinavia (1975 to 1995)
(Stabilator USA, Inc., 1998).

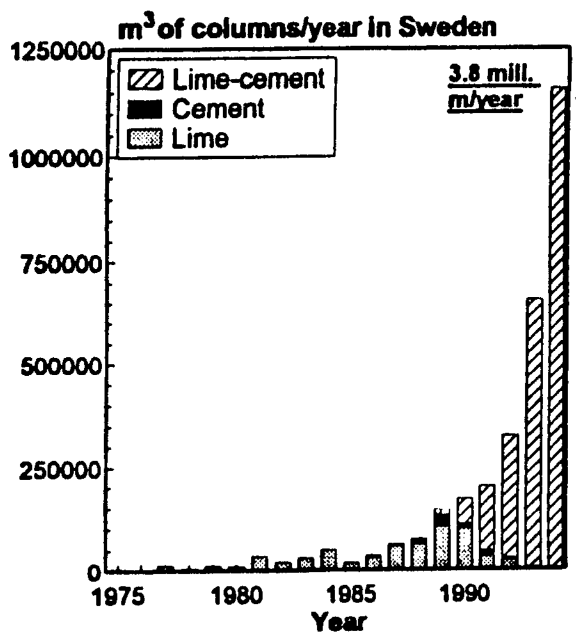


Figure 196. Changes in use of binders with time, based on Swedish output (1975-1994)
(Åhnberg, 1996).

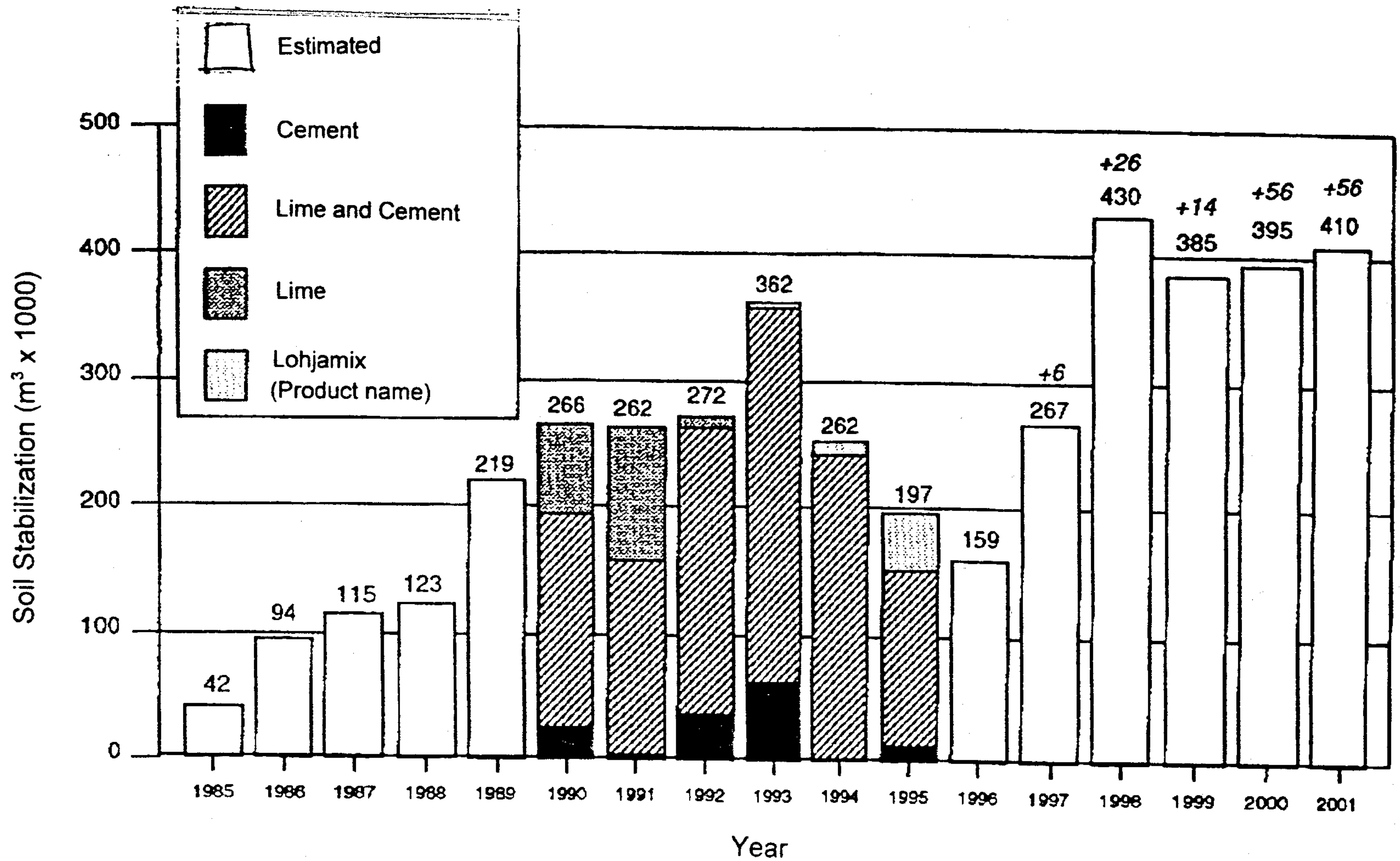


Figure 197. Details of Lime-Cement Column production in Finland
(Finnish Technical Development Center, 1996).

to 150 kg/m^3 and are a principal control over shear strength (assumed to be half U.C.S.) in a variety of soils (Figures 198 through 200).

Strength development with time is illustrated in Figures 201 and 202: for Lime-Cement Columns the 28-day strength is typically found to average about 2.5 times the 7-day strength.

Rajasekeran et al. (1996) investigated the influence of "pollutants" on lime-treated marine clays, since a) many researchers had found that in soils with significant contents of sulphates, the low pH, and the formation of highly swelling compounds such as ettringite and thaumasite resulting from lime-sodium sulphate reactions can cause several adverse effects on the engineering properties of the soil (Mitchell, 1986); and b) columns could be used in marine conditions (sodium ions and chlorides). They concluded that the addition of calcium chloride results in better formation of pozzolanic compounds than lime alone. Sulphates of sodium and potassium were detrimental. The presence of sea water had no retarding effect on the "formation of cementation compounds."

Many current investigations revolve around the treatment of peats and mosses, and as shown in Figures 203 and 204, higher binder factors (up to 400 kg/m^3) are needed to provide significant shear strengths (20 kPa and over). Esrig (1997) reported that the undrained shear strength of treated highly organic soils was typically around 10 times the virgin value, rising to 50 times in silty clays.

Kujala et al. (1996) reported on the role of humus on strength. The resultant humic acid was found to "detract extremely markedly" from the strength of coarse-grained material, but its effect was "considerably less" in fine-grained soil. They found that little improvement can be expected in soils with over 1.5% humus content.

Kivelö and Broms (1999) described low-strength Lime Cement Column research, and noted that a) stiffness and brittleness increase with cement factor, and b) shear strength is strongly influenced by and limited by the strength of the adjacent untreated areas (i.e., by the confining pressure).

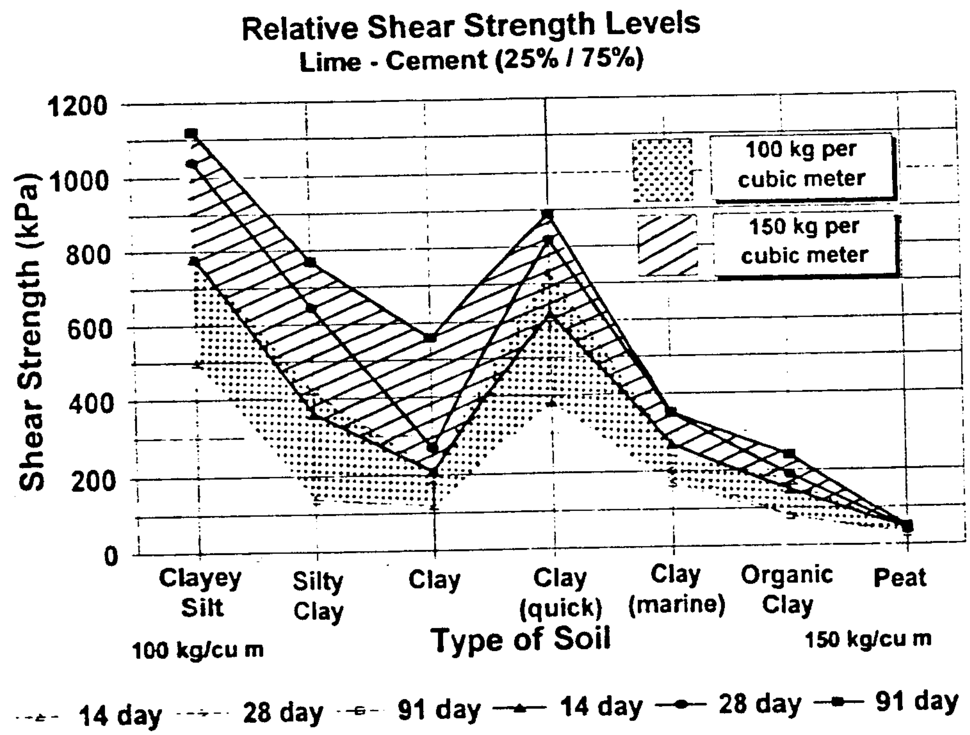
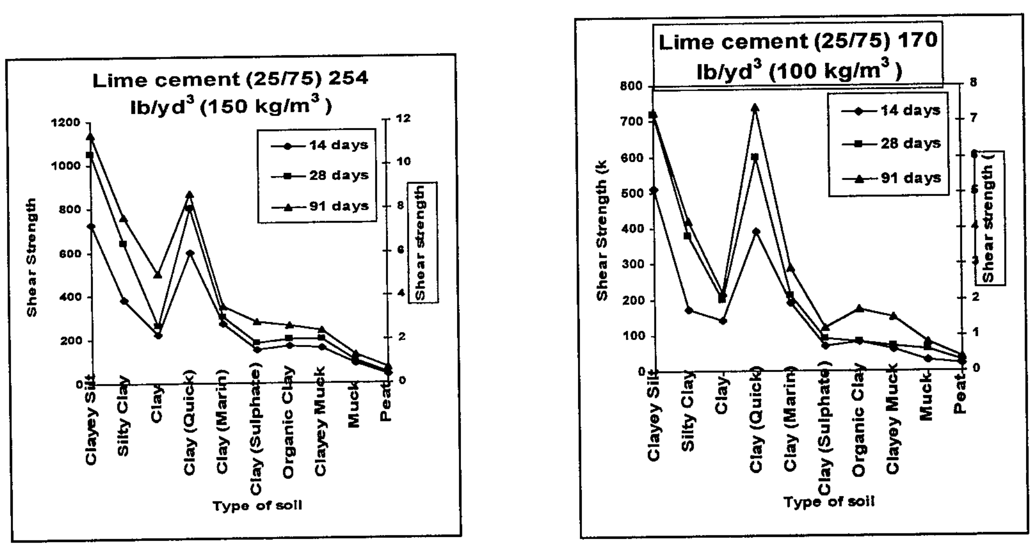


Figure 198. Shear strength of different soils mixed with two quantities of lime and cement at three curing times (after Hartlen and Holm, 1995).



* Shear strength taken as half unconfined compressive strength

Figure 199. Shear strength of different treated soils (Esrig, 1999).

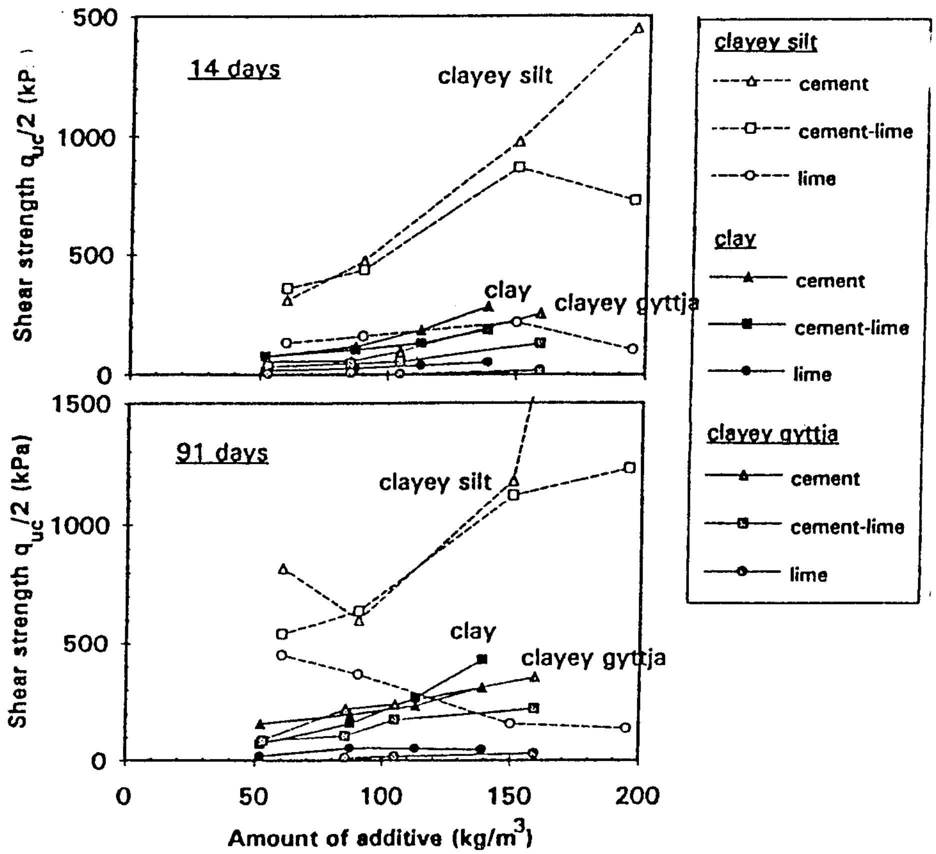


Figure 200. Examples of the effects on shear strength of different amounts of binder for three soils stabilized in the laboratory (Åhnberg et al., 1994).

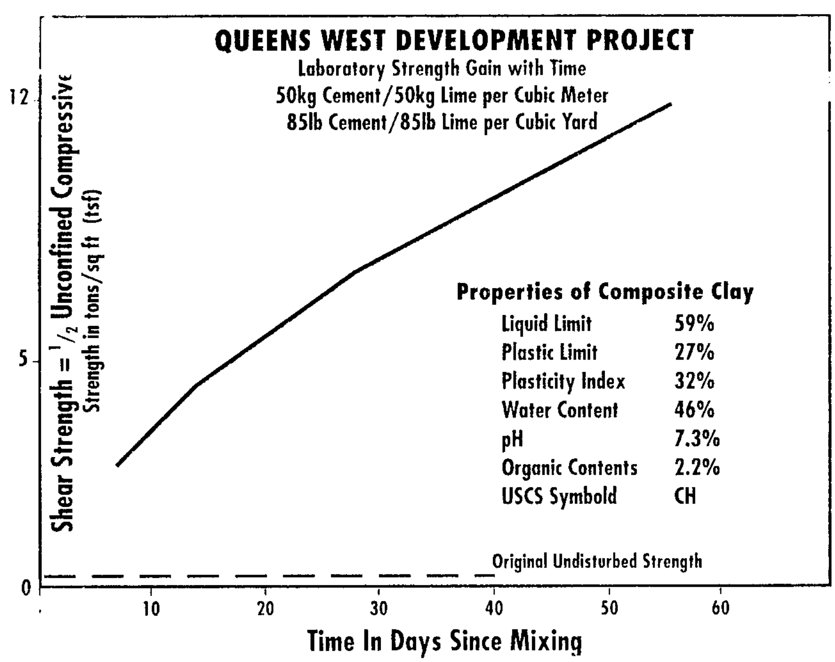


Figure 201. Shear strength versus time (days) since mixing (Stabilator, 1997).

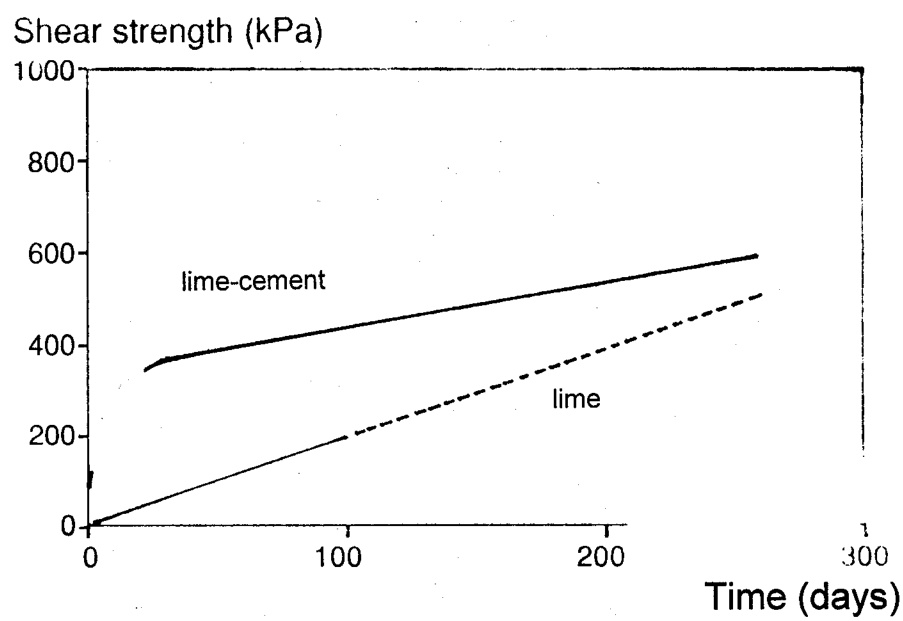


Figure 202. Comparison of strength development with time after mixing, for lime and lime-cement stabilized soil (Åhnberg et al., 1995).

Peat type and degree of decompo-sition	Binder	Binder content	Deforma-tion parameter m	Deforma-tion parameter β	Perme-ability k (m/s)
S H2	P40/28	400 kg/m^3	122,4	0,954	$10^{-5,8}$
C H3	P40/28+ CaO 1:1	"	14,3	0,649	$10^{-9,6}$
CS H3	P40/28	"	150,2	0,639	$10^{-6,1}$
BC H 2-3	P40/28+ CaO 1:1	"	15,7	0,429	$10^{-6,3}$

Figure 203. Deformation parameters and permeabilities of the stabilized peat samples (Huttenen and Kujala, 1996).

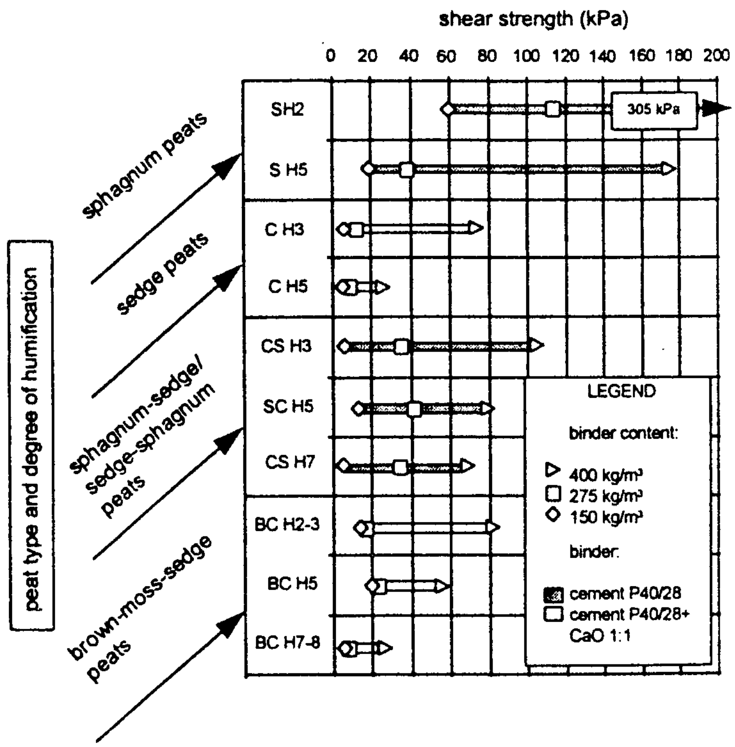


Figure 204. Shear strengths of stabilized peat with different binder contents after 180 days of curing (Huttenen and Kujala, 1996).

Regarding permeability, it is claimed that lime columns are 1000 times more permeable than clay, and that lime cement columns are 400 to 500 times more permeable. Huttunen and Kujala (1996) reported on the testing of some Finnish peats and determined that treated soil values were $10^{-5.8}$ to $10^{-9.6}$ m/s (corresponding to till and clay, respectively).

Broms (1999) provided a comprehensive overview of the properties of Lime Cement Columns that is also relevant to Chapter 6. He first noted that lime (CaO) and cement are both needed to provide significant shear strength in organic soils, while lime alone provides relatively high permeability. The (unslaked) lime contributes "a large part of the increase in shear strength" by reducing the water content of the soil during slaking. The heat generated further reduces the water content. Both the properties and the behavior of treated soil are affected directly when cement is added (Kivelo and Broms, 1999): for example, the shear strength and E-value of columns with cement can be as much as 10 times higher than those of lime columns, whereas the permeability is reduced.

Regarding the details of shear strength:

1. Undrained shear strength: short-term strength of lime-treated soils depends on the (above) slaking reaction and the resultant increase in the plastic limit and the reduction of the plasticity index. Long-term strength increase is mainly due to pozzolanic reactions and the behavior becomes similar to that of an overconsolidated clay. S_u is usually determined by the following tests:
   - Unconfined compression.
   - Undrained triaxial and direct shear tests (UU).
   - (Occasionally) consolidated undrained triaxial tests (CU) are used, where the confining pressure during consolidation corresponds to the estimated lateral pressure in situ.

   Column undrained shear strength (labeled $\tau_{fu.col}$) increases with normal pressure:

   $$\tau_{fu.col} = c_{u.col} + \sigma_f \tan \Phi_{u.col}$$

   Tests indicate $\Phi_{u.col}$ varies 25° to 45° but can be as high as 60°. A value of 30° can be assumed for a normal pressure of 150 kPa (Figure 205). The undrained shear strength is usually assumed to be 50% of the unconfined compressive strength, but Kivelö indicates that this value "could be too high when the normal pressure is less than a critical value $\tau_{fu.crit}$." As shown in Table 56, the $\Phi_{f.crit}$ decreases from 77.8 kPa at $\Phi_{u.col} = 25°$ to 58.4 kPa at $\Phi_{u.col} = 45°$ when $\tau_{fu.col}$ is $< 0.5q_{u.col}$. Undrained shear strength is 34 to 91% higher than the shear strength determined from U.C.S. ($0.5q_{u.col}$) when the normal pressure exceeds 150 kPa.

2. Drained shear strength: τ_{fd} is estimated by

   $$\tau_{fd} = c'_{col} + \sigma_f' \tan \Phi'_{u.col}$$

   where σ_f' is the normal effective pressure. Values up to 25 to 30° are used for $\Phi'_{u.col}$, but must be verified by CU or drained direct shear tests.

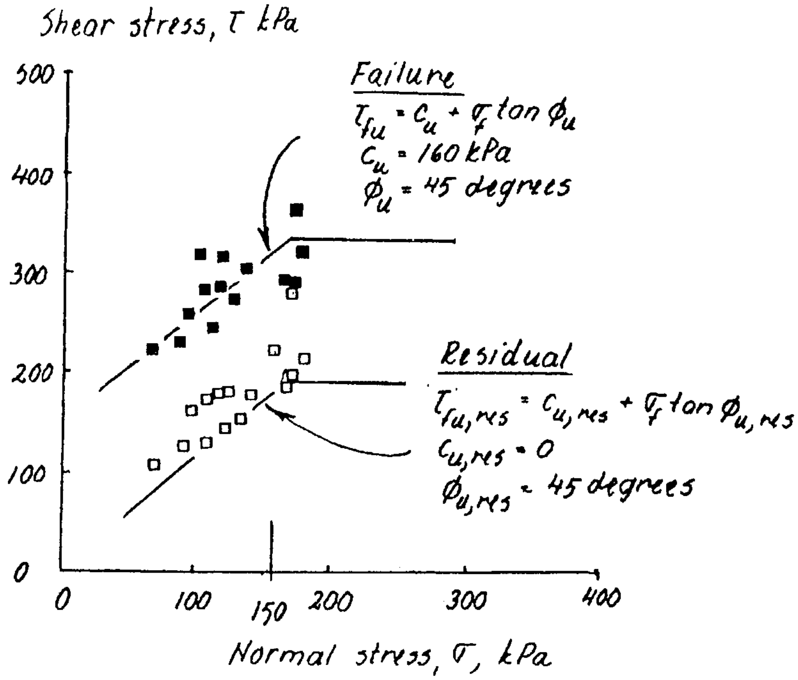


Figure 205. Failure and residual undrained shear strength by direct shear tests (after Kivelö, 1997).

Table 56. Undrained shear strength as determined by unconfined compression tests at $\Phi_{u,\,col} > 0$ (Broms, 1999).

$\phi_{u,col}$	$2\,c_{u,col}/q_{u,col}$	$\sigma_{f,col,crit}/q_{u,col}$	$150 \tan \phi_{u,col}$	$\tau_{fu,col}/0.5\,q_{u,col}$ *
25°	0.637	0.389	69.9	1.34
30°	0.577	0.366	86.6	1.44
35°	0.520	0.342	105.6	1.57
40°	0.466	0.318	125.9	1.73
45°	0.414	0.292	150.0	1.91

* at $\sigma_f > 150$ kPa

3. Residual shear strength: the reduction in strength mainly reflects a reduction in cohesion (Figure 205) as opposed to friction angle. Broms therefore proposes ignoring cohesion when residual strengths are to be used in design.

4. Failure strain and ductility: strain can be up to 5% for U.C.S. < 200 kPa but as low as 0.5 to 2% when U.C.S. > 300 kPa (Figure 206). It seems to be higher in drained than in undrained triaxial or direct shear tests, and to also decrease with time. It is higher, logically, in organic soils, and in wet mix methods for equivalent cement factors, and in soils of higher moisture content – all where shear strength is low.

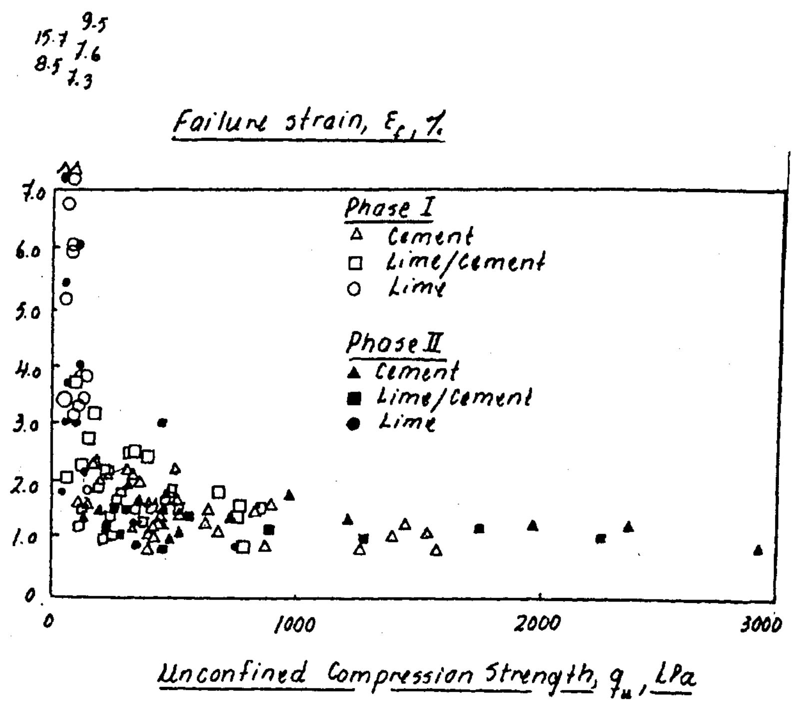


Figure 206. Failure strain (after Åhnberg et al., 1995).

5. Increase in shear strength with time: typically faster in the field than in the laboratory, due to higher ground temperature and the higher confining pressure, which influence the pozzolanic reactions in particular. This increase is initially faster with lime cement than with lime. Many researchers have concluded the rate is proportional to the square root of time.

Broms also reviewed data on Compression Modulus and Modulus of Elasticity:

1. Compression Modulus (M_{col}): tests show that the in situ value may be up to five times higher than that obtained by oedometer testing. The ratio of modulus to undrained shear strength increases with shear strength in the ratio of 50 to 250, being lower for lime columns at equivalent undrained shear strengths. It also increases with time. M_{col} can be estimated from modulus of elasticity, E_{col}, and from Poisson's ratio, υ_{col}:

   $$M_{col} = E_{col} (1- \upsilon_{col})/ (1+ \upsilon_{col})(1- 2 \upsilon_{col})$$

   M_{col} is about 1.35 E_{col} up to about 50% of the ultimate bearing capacity at $\upsilon_{col} = 0.3$. The ratio is 1.60 E_{col} and 2.14 E_{col} at $\upsilon_{col} = 0.35$ and 0.40, respectively.

2. Modulus of Elasticity (E_{col}): as determined by U.C.S. tests on lab samples, E_{col} has been higher than values on cores from field installations (held to be representative). $E_{col}/c_{u\ col}$) is about 200 for Lime Cement Columns and 250 to 300 for cement columns.

Regarding permeability, laboratory tests are judged to be "too low" relative to in situ data. Lime columns function as vertical drains, and the permeability often increases with time (due to shrinkage). For Lime Cement Columns, the drainage role is not so well defined, especially when the confining pressure is high. Permeability usually decreases with time because of shrinkage. Cement columns have low permeability and so do not function as drains.

Larsson (1999), a researcher at the Royal Institute of Technology in Stockholm, Sweden, presented a fundamental appraisal of the "extremely complex" mixing mechanisms as related to the Lime Cement Column Method. The "motion regions" in dense particulate suspension like soft clays display mostly laminar (as opposed to turbulent) flow characteristics. The active mechanisms of laminar mixing are laminar shearing, extensional flow and distributive mixing, molecular diffusion, and shearing (Figure 207).

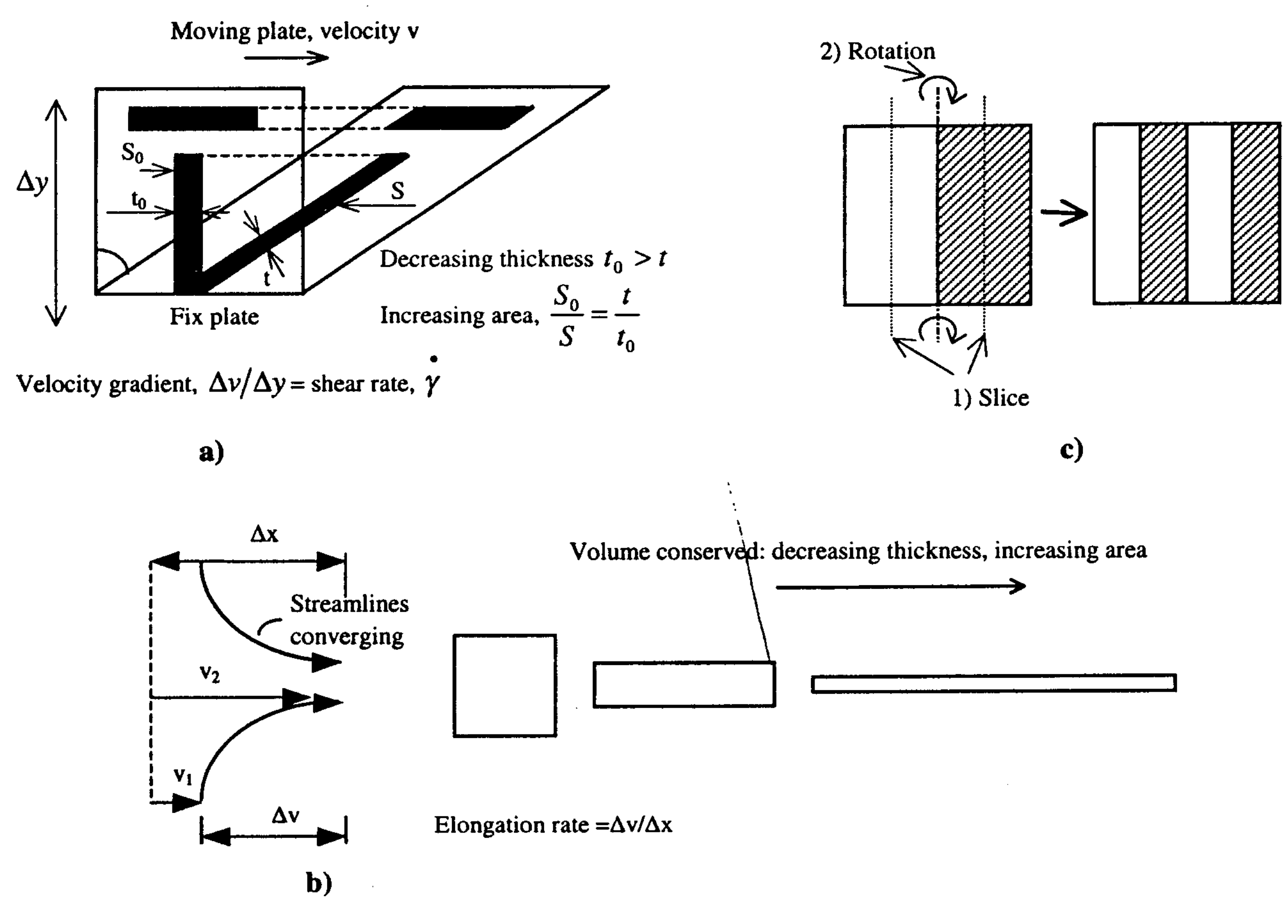


Figure 207. Laminar mixing mechanisms: a) the thinning of elements resulting from laminar shear, b) the thinning of elements because of extensional flow, c) distributive mixing by slicing and rotation (Larsson, 1999).

Laminar shearing decreases the thickness ("striation thickness") and so increases surface area. The efficiency of laminar mixing can be improved by continuously changing the direction of the shearing action, i.e., the blades of the mixer should be oriented at different angles. Extensional flow is a very effective mixing mechanism and can be created by forcing soil through a hole in the mixing device. Teeth in the blade give this effect. Distributive mixing (slicing and rotation) is not suitable for breaking up clumps. In soils, molecular diffusion is slow and its rate depends on the efficiency of other mechanisms. It is only significant if striation thickness (Figure 208) is small.

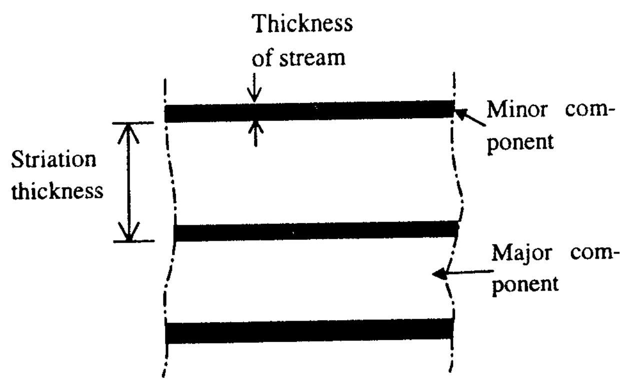


Figure 208. Striation thickness as a measure of mixing quality (Larsson, 1999).

The dry mixing process involves dispersion of the binder into four stages: incorporation (ideally homogeneously), wetting, breakdown of particle clusters, and possible flocculation. These stages overlap in practice. Breakdown, depending on the rheological properties of the mixture and the type of mixing, is governed mainly by a) the effective strain and the number of revolutions of the blade, and b) the shear rate and the intensity of the agitation.

It is very important to reduce the striation thickness so that the molecular diffusion of the binder can become significant. Lime (creating $Ca(OH)_2$) improves the rate of molecular diffusion while cement (creating CSH gel) fills interparticle spaces: molecular diffusion limited to cement therefore requires "considerable" mechanical mixing. The rheology of cohesive soils in particular, which controls ease of mixing, is very complex and depends upon:

a) The chemical and physical properties of the porewater, and its amount.
b) The formation of agglomerates and cementation in the minerals.
c) The clay fabric (which can be modified by clay dispersants).
d) Orientation and density.
e) Mechanical, physical, and chemical interactions between the minerals and between the solution and the air in the pores.

These are further variables to consider when optimizing mixing tool design: the subject is patently very complex. The goal of designing a mixing device that creates sufficient movement in the soil without a great mixing effort or long mixing time is correspondingly difficult to achieve.

The European-Community-sponsored project referred to as Euro Soil Stab was reported upon in 1999 by both Lahtinen et al., and Åhnberg and Holm. More than 30 different soil types from Finland and Sweden were researched to determine the most effective binder compositions (Table 57). Bearing in mind that two seemingly similar soils can be very different in terms of stabilizing features and that binder quantity and curing time have still to be optimized, Åhnberg and Holm described on the results of the project as follows:

"A large number of laboratory tests have been performed on stabilized peat and gyttja, enabling a good overall picture of the effect of different binders. Different types of tests are being performed in the laboratory, and the results from unconfined compression tests presented in this paper have been run in the initial stage of the testing program. These tests show that:

- It is possible to stabilize all the different soils tested. Very good results were obtained with cement-slag as stabilizing agent, both in peat and in gyttja, but high strengths were obtained also with other types of binders. The addition of gypsum to cement-slag proved to be effective in stabilization of the Dölme gyttja but unsuitable in the Holma gyttja.
- It is not merely the ordinary geotechnical parameters such as water content or organic content that influence the effect of different binders. Chemical properties also influence the effect.

Table 57. Properties of soils and binders used in Euro Soil Stab project (Lahtinen et al., 1999).

Components of binders

Code	Definition
F	Finnstabi® (Gypsum)
T	Slaking residue, $Ca(OH)_2$
C	Portland cement
K	Furnace slag (FIN)
M	Merit (furnace slag SWE)
L	Quicklime, CaO
A	Flyash (coal)

Properties of untreated soil

Material	water content %	humus content %	Ø ≤ 2 μm fraction %
Clay /FI	99	0	86
Gyttja /FI	148	8,6	19
Clay /FI	66	4,9	30
Gyttja /FI	111	9,9	<5

Properties of untreated peat

Material	water content %	LOI %	pH
Peat/FI	668	95	4,7
Peat/SWE	869	89	5,8

- The strength of the stabilized soil increases with increasing amount of stabilizing agent. The growth in strength with time continues for an increasing period of time when using larger amounts of stabilizing agent.
- The addition of sand to other stabilizing agents in a quantity of 100 kg/m^3 had no positive effect on the strength of the stabilized peat samples.

- The curing temperature can have a considerable influence on the strength increase in the stabilized soil. According to the laboratory tests, a high temperature should preferably be avoided in stabilization of peat. Cement-lime as stabilizing agent generates relatively high temperatures during slaking and should therefore perhaps be avoided. Apart from generating a larger amount of heat, lime, which as binder normally creates a large number of pozzolanic reactions with the clay's minerals, may possibly in general be less efficient when used in stabilization of peats with high organic contents.
- The stabilized peat samples showed a more extended failure compared to the stabilized gyttja, but decreased with increasing strength in both types of stabilized soil.
- The permeability decreased after stabilization, and loading, of the peat. This decrease in permeability is partly caused by the initial loading and compression of the stabilized peat. The permeability of the Holm gyttja was of the same order of magnitude as that of the unstabilized gyttja.

5.2 Derived and Related Parameters and Properties

Regarding shear strength, DJM Association (1993) provides data that indicate that this is between 33 and 50% U.C.S. (Figure 209). However, in the lower U.C.S. range (less than 1 MPa) the relation is more consistently 50% – in line with the Scandinavian assumption.

DJM Association (1993) has found tensile strength (splitting test) to be 10 to 20% U.C.S., with this percentage decreasing with increasing U.C.S. (Figure 210).

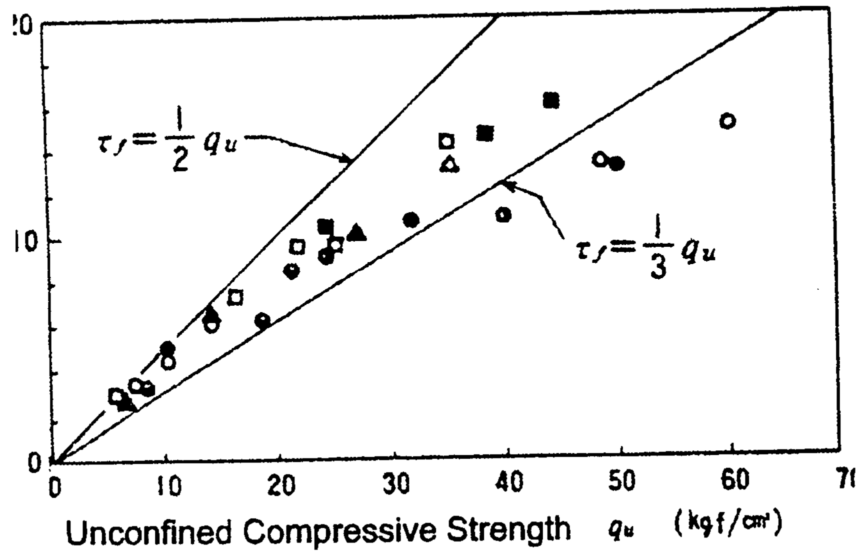


Figure 209. Relationship between shear strength and unconfined compressive strength (after DJM Association, 1993).

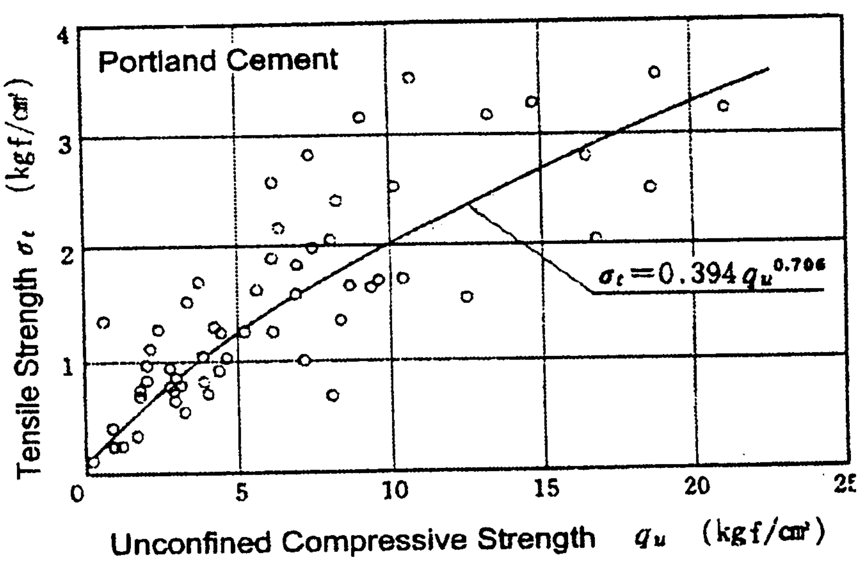


Figure 210. Relationship between tensile strength and unconfined compressive strength (after DJM Association, 1993).

Data on the relationship of E_{50} and U.C.S. are shown in Table 58.

Table 58. Published relationships between E_{50} and U.C.S.

SOURCE	ASSUMPTION
Broms and Boman (1979) (LCC field)	150 x U.C.S. (long term) to 300 x U.C.S. short term
Pavianni and Pagotto (1991) (Trevimix field)	a) 10 to 50 times virgin undrained E-value 30 to 100 times virgin drained E-value b) 1×10^3 to 2.66×10^3 MPa (i.e., about 200 to 500 x U.C.S.)
DJM (1993)	50 to 200 x U.C.S. (Figure 211)
Lin et al. (1997)	100 x U.C.S. (for U.C.S. = 0.6 MPa)
Esrig (1997) (LCC)	50 to 200 x shear strength
Broms (1999)	200 x shear strength for lime and lime cement columns 250 to 300 x shear strength for cement columns (Note: values obtained from U.C.S. laboratory samples are higher than those obtained on cores from field.)

It would seem therefore that the ratio of E_{50}:U.C.S. is somewhat lower, and in a tighter range than for wet methods. There is also a suggestion that the ratio decreases with decreasing strength, being perhaps 25 to 50 for strengths less than 0.3 MPa compared with 50 to 200 for strengths of 0.3 to 2 MPa, and over 500 for strengths of up to 5 MPa.

Overall, some typical properties of lime-cement treated soils, according to Broms (1991) are as follows:

- The shear strength is increased within 1 to 2 hours after completion of mixing.
- The shear strength will normally increase 10 to 50 times if the untreated strength is low. Higher liquid limit soils gain less relative strength.
- The shear strength gradually increases with time over many months.
- Approximately one-third of the ultimate shear strength is usually obtained after 1 month and approximately three-quarters after 3 months for lime columns.
- Quicker strength gain can be expected with Lime Cement Columns.

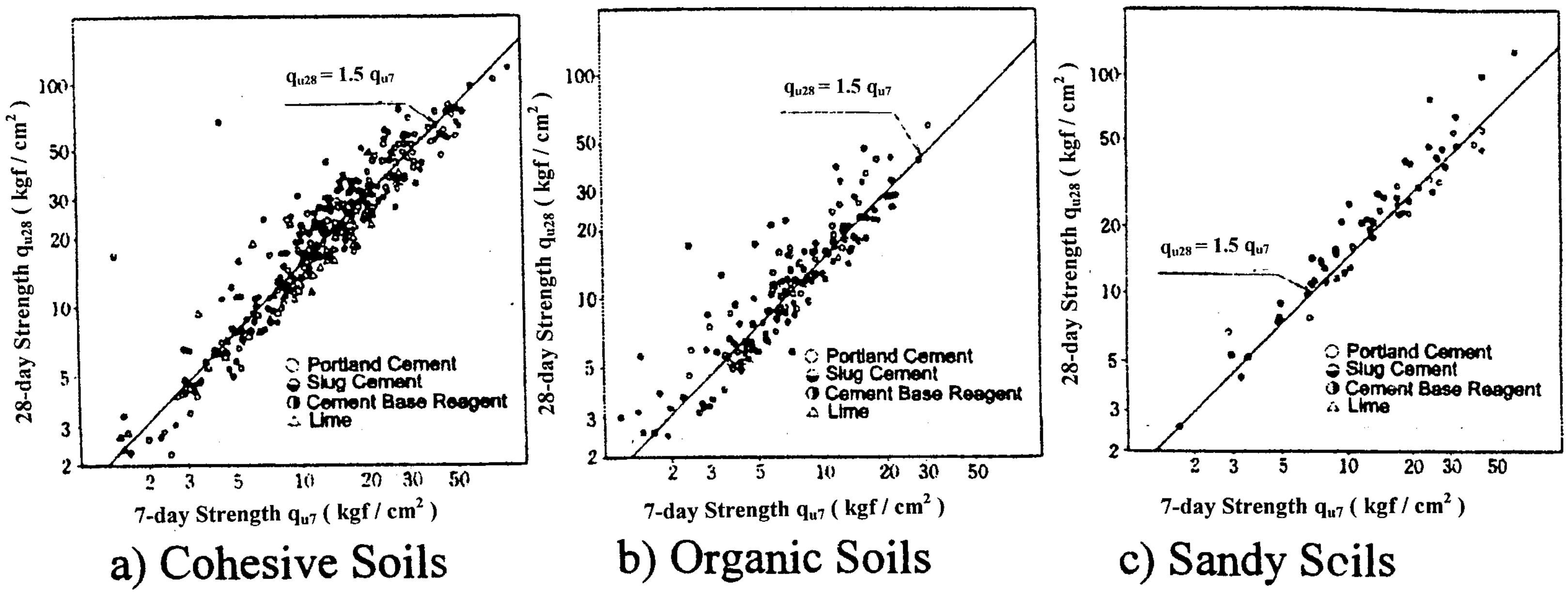


Figure 211. Relationship between E_{50} and unconfined compressive strength (after DJM Association, 1993).

- Usually the undrained shear strength of the stabilized soil in the columns will exceed that determined by unconfined compression or undrained triaxial compression tests on laboratory samples.
- Even with very careful mixing during column construction, the stabilized soil strength is not uniform. The measured strength will vary with the testing method and with the size of the tested samples.
- More binder is required for soils with higher plasticity index.
- The stabilized soil is normally firm to hard and grainy in texture.
- The compressibility is reduced.
- The preconsolidation pressure is increased.
- The sensitivity of the soil is low, typically 1 to 3.
- The plastic limit is increased.
- The plasticity index is reduced.
- The permeability is increased normally 100 to 1000 times for lime columns, which can then function as drains. Lime Cement Columns display less increase in permeability.
- The soil moisture content is reduced.

As a final point, Yang et al. (1998) made the following important statement regarding the uniformity of mixing on treated soil properties:

"Soil type is one of the major factors affecting the strength of soil-cement. It also affects the procedures for the evaluation of the soil-cement produced by deep mixing. Deep mixing is not designed to dissolve the in situ soils before blending it with the cement powder or grout. The cutting heads and mixing paddles are designed to break up the in situ soil and mix it with cement powder or grout. In the case of cohesionless soils with minor fines, sand and gravel particles become dispersed aggregates of the soil-cement mixture. When the fines content and its plasticity increase, the soil lumps increase. In the case of highly cohesive clays, part of the clay cannot be broken down during the soil mixing process and remains as lumps inside the soil-cement mixture. As long as the lumps disperse inside the soil-cement, the performance of the soil-cement will be satisfactory. Therefore, the uniformity of soil-cement should be evaluated from large-scale viewpoint. To focus heavily on the existence of clay lumps might mislead the evaluation on the mass performance of the soil-cement structures."

5.3 Data from Laboratory Tests

Kukko and Ruohomäki (1995) conducted an intensive laboratory research program to analyze the factors affecting the hardening reactions in stabilized clays. New binder alternatives based on industrial by-products were studied for use in deep stabilization and block stabilization. The main emphasis was on pulverized granulated blast-furnace slag activated in different ways, and on binder combinations based on flyash and waste from desulphurization units.

"The results of 1355 laboratory tests with 195 admixtures and 21 soil types indicated that the strength of stabilized clay is strongly dependent on the water/cement ratio, and that the strengthening effect is minimal at water/cement ratios above a certain threshold value. In the case of deep stabilization, the strong dependence on water content, which in clay deposits can vary greatly with depth, makes the system very sensitive as far as strength is concerned. As a consequence of this, it has been found that when the binder dosage is below the threshold value, the clay is not stabilized, thus causing surprising inhomogeneity in the strength of a stabilized column.

The findings of this study are presented in a mathematical model predicting the final compressive strength of a stabilized clay as a function of the water/cement ratio, the humus content and the quantity of fines (< 1 μm). For the types of clays tested and with cement as the main component of the binder, the effect of organic impurity proved to be opposite to that which has traditionally been assumed. Organic matter resulted in an increased compressive strength at a given binder quantity.

In the case of cement as the stabilizing agent, the model is:

$$f_c = 0.347\mu \cdot e^{-0.57w} + 0.372h^2 \cdot e^{-0.27w}$$

where f_c = compressive strength at prismatic sample of size 40 x 40 x 160 mm^3 (MPa)

w = water/cement ratio

μ = amount of clay fraction < 1 μm (%)

h = amount of organic matter (%).

The results of laboratory test series, where the compressive strength of the stabilized material was determined at 20°C and 28-day hardening period for 21 soil types, are compiled in the graphs in Figure 212. Soils with organic contents between 4.3 and 6.1% are clearly grouped separately. The lower and upper limits of the shaded areas represent fines contents between 42.6 and 67.9%."

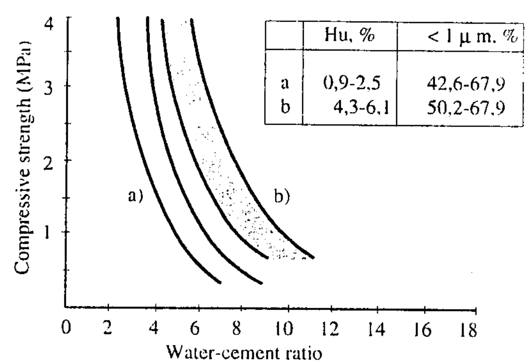


Figure 212. Compressive strength (at 20°C and 28 days) versus water/cement ratio (Kukko and Ruohomäki, 1995).

The results of this research work were used for the design of a field test program in Helsinki. As usual with the available technology, a constant cement factor was used in the soft clay deposit. The strength parameters of the columns were analyzed after a curing period of 90 days both by CPT testing and by unconfined compression tests on samples taken from stabilized columns. The measured values were far beyond the expectations gained from the laboratory tests.

Hampton and Edil (1998) researched the strength gain of organic ground with cement binders, via laboratory tests in Madison, WI, and Delft, Holland. They concluded that "traditional index properties are not suitable to base a reliable stabilization protocol," especially for soils with high organic contents. "Rather, thorough knowledge of the chemical properties of the soil and the hydration properties" of the binder are necessary.

The three Wisconsin peats were of different botanical origins and degrees of decomposition while the Dutch soils were an organic clay and a peat (Table 59). The initial U.C.S. of all soils was less than 200 kPa. Different binders were used, including blast furnace slag (Table 60).

Various experimental parameters were investigated including sample reproducibility, sample size (50-mm specimens unexpectedly gave strengths 50 to 60% of the 66-mm specimens), preparation, and wet vs. dry mixes (Table 61).

Regarding the strength gain, the water content shown in Figures 213 and 214 is only that which is definable as free water. This illustrates the influence of cement factor and binder type. From the data of Figure 214, they concluded that the "characteristics of the solid soil particles dominate the stabilization potential of the organic soil. These could be mechanical or chemical characteristics."

Data support the hypothesis than an important role of the binder is to form strong hydration products, e.g., the small, clay mineral fraction in the peats would not support a pozzolanic reaction with lime (only calcium hydroxide). They concluded that bentonite both bound excess water and served as a filler and source of silica for the C-S-H gel formation.

Regarding the influence of water/cement ratio on strength gain, gypsum was found to reduce U.C.S. (Table 62), and the strength could not be deduced from the contents of the water, cement, soil solids, and organics due to " too much interaction between the properties of the soil" and the fact that the soil solids are potentially reactive. Hampton and Edil found (Table 63) that the blast furnace cement and anhydrite mixture (Binder F) increased strength in organic clay more than lime-cement and that the addition of a small amount of high aluminum cement appeared to give further strength increase. The authors rationalized that the blast furnace slag is an important, reactive form of silica, aiding C-S-H development whereas the organic substances promote (weaker) ettringite.

Table 59. Soil properties (Hampton and Edil, 1998).

	Wisconsin			Netherlands	
	Sphagnum	Reed Sedge A	Reed Sedge B	Reed Sedge D	Organic clay
Origin	Northern WI	Southwest WI	N. Central WI	Abcoude	's-Gravendeel
Classification	Fibrist	Saprist	'Ultra Saprist"	Saprist	OC
water content	1500-2000%	200-300%	300-450%	600-700%	100-200%
pH	2.88	5.61	4.35	5.8	6.7
Loss on ignition	92%	59%	46%	83%	13%
bulk unit weight g/cm^3	1.01	1.15	1.10	1.01	1.31

Table 60. Composition of binding agents (Hampton and Edil, 1998).

	SiO_2 (%)	Al_2O_3 (%)	CaO (%)	MgO (%)	SO_3 (%)	Fe_2O_3 (%)	Blaine cm^2/g
Ordinary Portland Cement (OPC)	21.2	5.8	62.0	6.0	3.0	2.0	3200
High calcium quicklime (CaO)	1.0	0.5	97.0	1.0		0.5	
Blast furnace cement (FSC IIIB)	28.3	9.4	46.4	7.1	4.4	2.4	4375
Blast furnace cement (FSC IIIA)	27	11	49	7	3	2	5500
Binder F	21.8	10.9	47.3	6.1	10.8	1.6	5280

Table 61. Influence of sample preparation on unconfined compressive strength (Hampton and Edil, 1998).

Set	W/C	Specimen Diameter (mm)	Curing conditions	Number of Samples	Unconfined Compressive Strength (kPa)		
					Mean	Std. Dev.	95% Confidence
1	1:1.5	66	sealed	4	1094.77	143.9	162.8
2	1:1	66	sealed	4	842.8	54.6	75.7
3	Dry	66	sealed	4	1745.7	214.1	209.9
4	Dry	50	sealed	4	949.4	41.6	47.0
5	Dry	66	Load + Immersed	4	1118.6	131.3	128.7
All				20	1150.0	372.2	171.9

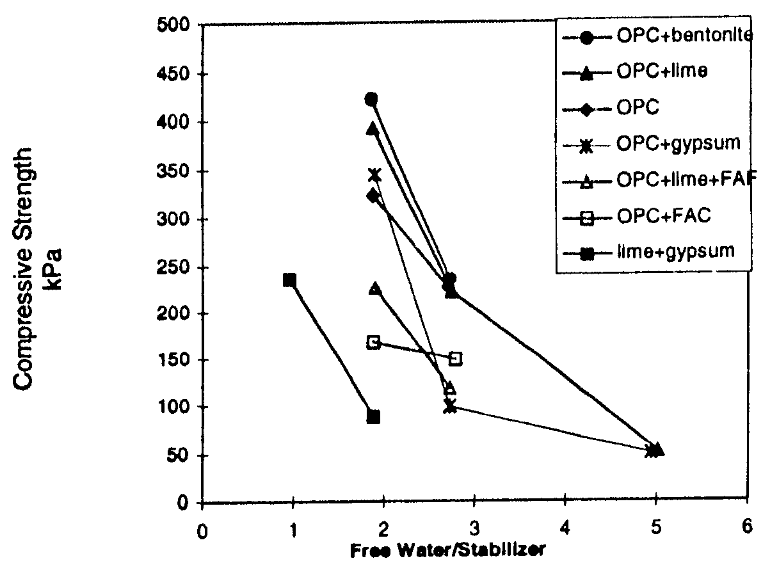


Figure 213. Compressive strength of sphagnum peat stabilized with various building agents (14 days, curing time) (Hampton and Edil, 1998).

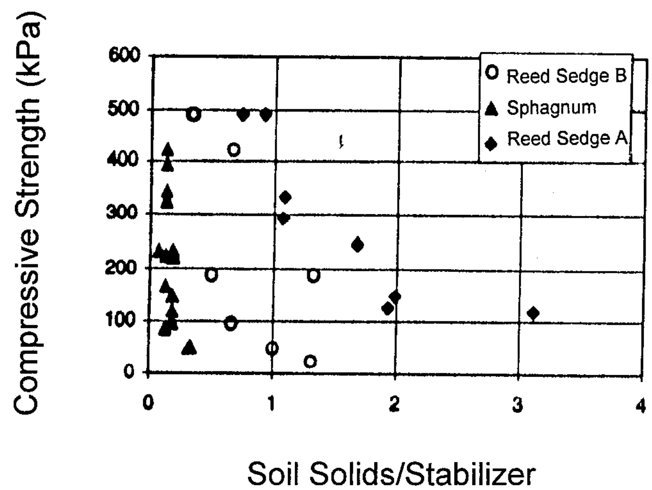


Figure 214. Compressive strength versus soil solids stabilizer – Wisconsin peats (Hampton and Edil, 1998).

Table 62. Unconfined compressive strengths of stabilized Wisconsin peats (kPa) (Hampton and Edil, 1998).

Binder (cure time)	Reed Sedge A		Reed Sedge B		Sphagnum	
W/C/Sθ	1/1/1	1.4/1/1	1/1/1	1.4/1/1	1.4/1/1	2/1/1
OPC (200 days)	3265	1393	238	1398	2011	263
OPC (40 days)	1600	800	158	720	40	75
OPC+$CaCl_2$, 100:1 (200 days)	2577	2498	2159	1293	2083	328
OPC+Gypsum, 2:1 (200 days)	1914	643	368	389	1153	250

Table 63. Unconfined compressive strength of 's-Gravendeel organic clay (Hampton and Edil, 1998).

Binder	28 day compressive strength (kPa)
Binder F, 200 kg/m^3	1460
50% OPC + 50% Lime, 100 kg/m^3	40
80% OPC + 20% Lime, 150 kg/m^3	145
80% FSC A+ 14.5% Anhydrite + 5.5% High Aluminum Cement, 200 kg/m^3	1827

Al-Tabbaa et al. (1999) described laboratory tests to evaluate the efficiency of various designs of augers. Figure 215 shows the following:

- Auger 1: 6-cm-diameter, multiblade.
- Auger 2: 9-cm-diameter, two blades.
- Auger 3: 9 cm-diameter, two closer blades but with mixing teeth.

Tests were conducted in medium-coarse sand of moisture contents of 10% and 18% (saturated). Type III cement and bentonite up to 10% by weight were used.

In wet mixing, the effects were similar in clean sand, but in sandy clay and cohesive fills, Augers 2 and 3 were significantly more effective. In dry mixing, the end mixing principles of Augers 2 and 3 proved to provide better uniformity, as did the wetter sands. The mixing efficiency increased as the binder percentage increased (effective from 6 to 7% onwards) and as penetration rates decreased. Bentonite also improved homogeneity.

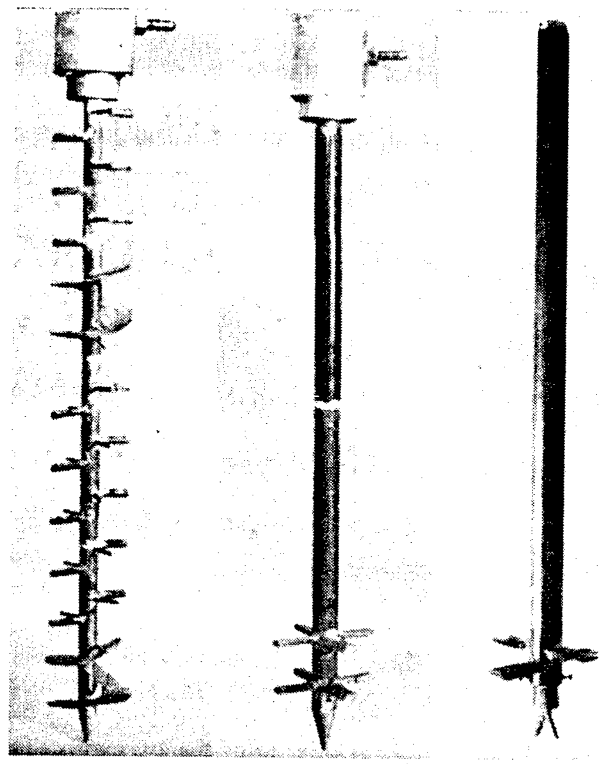


Figure 215. Three laboratory-scale augers (Al-Tabbaa et al., 1999).

5.4 Data from Field Tests

Pavianni and Pagotto (1991) described field tests at Pietrafitta, the first Italian application of a DRE method as a potential deep foundation. The soils treated were as follows:

Soil 1: Dump material consisting of silty clay, with silty sand, levels of lignite and laminations of ash (total 22 to 25 m thick). Behavior similar to a highly plastic clay, of medium consistency and slightly overconsolidated. (γ = 1.75 t/m^3; c_u = 1.7 to 4.6 t/m^3).

Soil 2: Lignitiferous peat soil (3 m thick). Similar to a highly organic clay subjected to moderate precompression. (γ = 1.52 t/m^3; c_u = 10.2 to 17.8 t/m^3).

Soil 3: Silty, peat clay, some sand. ($\gamma = 1.99$ t/m^3; $c_u = 8.5$ to 21 t/m^3).

The owner approved the construction of two test areas (Figures 216 and 217), with 16 CPT penetrometers and 3 core holes giving the following data:

SOIL	**NATURAL SPECIFIC WEIGHT, γ (t/m^3)**	**NATURAL MOISTURE CONTENT,W (%)**	**PLASTICITY INDEX, PI (%)**	**LIQUID LIMIT, W_L (%)**	**ORGANIC CONTENT O_C (%)**	**UNDRAINED SHEAR STRENGTH, C_U (kg/cm^2)**
Soil 1: Soils above the lignite level: consisting of silt with clay, peat in plaçes	1.89 to 2.07	17 to 35	16 to 30			0.30 to 1.00
Soil 2: Lignite level	1.20 to 1.50	60 to 145		99 to 139		
Soil 3: In situ clay	2.00 to 2.05	22 to 26			30 to 60	1.20 to 1.60

The water table was measured at ground level.

The first test (T1) area had 116 columns, each 1 m in diameter, to 16 m depth (two columns to 22 m). The second test (T2) area had 68 columns, each 1 m in diameter to 18.5 m depth in pairs (two columns to 22 m). The cement factor in each case was 220 kg/m^3.

Two to three months after installation, six vertical cores were taken coaxially within six columns, as in the following summary:

TEST AREA	**NATURAL SPECIFIC WEIGHT, γ (t/m^3)**	**NATURAL MOISTURE CONTENT, W (%)**	**U.C.S. (MPa)**	**E_{50} (MPa)**
T1	1.72 to 1.94	18 to 31	2.5 to 6.5	210 to 1090
T2	1.62 to 1.94	19 to 40	1.5 to 4.5	125 to 700

The improvement in strength and in the deformation modulus of the treated soil over the natural soil was noticeable. The increase in strength may be estimated to be 15 to 20 times, while the increase in the modulus is much more widely scattered and may be 10 to 50 times in undrained conditions or 30 to 100 times in drained conditions.

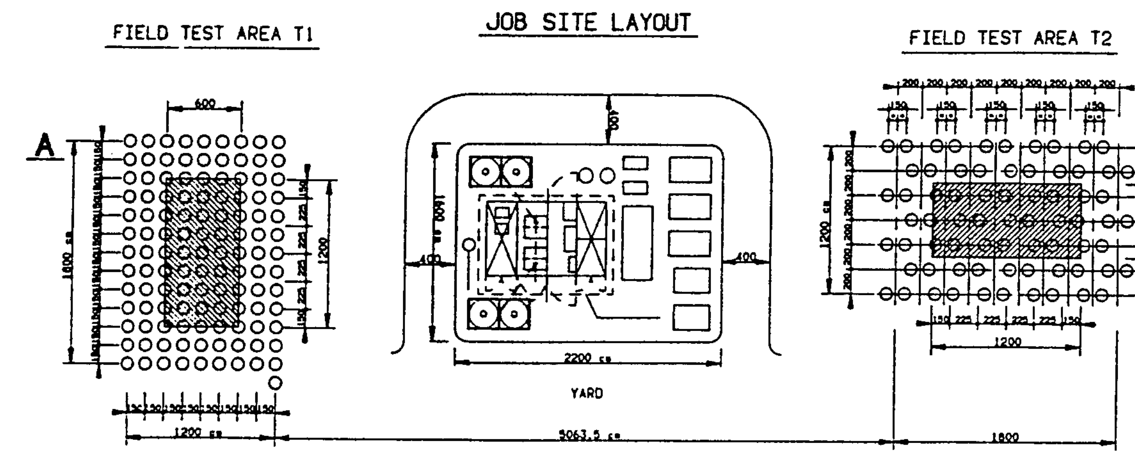


Figure 216. Layout of pile test areas (Pavianni and Pagotto, 1991).

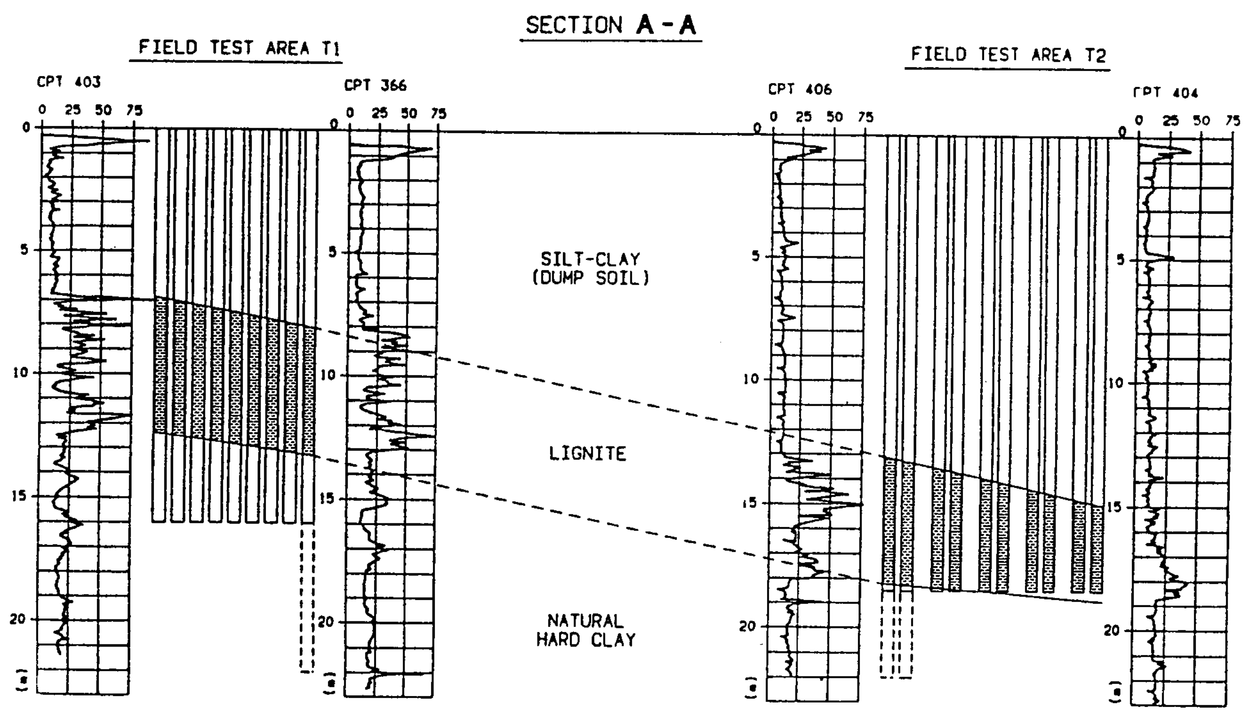


Figure 217. Significant stratigraphic section (Pavianni and Pagotto, 1991).

The tests aimed at defining the mechanical characteristics of treated lignite give extremely scattered values, and further research was needed to obtain a sound trend of behavior. The most significant tests are those aimed at determining the pH value. This value varies from 5 to 6 for natural lignite and from 8 to 11 for treated lignite, proof of the disaggregation and mixing achieved during the treatment.

An inspection shaft was sunk at T1. Cores were taken parallel and perpendicular to the column axes (Figure 216). U.C.S. values (horizontal) ranged from 3.2 to 7.1 MPa (within the range of the vertical cores).

Kujala (1982) investigated the use of gypsum as an additive to quicklime. Although this led to the formation of ettringite crystals, which improved strength significantly, these crystals proved unstable at low pH values and temperatures exceeding 40°C. He wrote that lime-gypsum is usually mixed in soil at 3 to 10% by dry weight and that best results are obtained with semi-hydrated gypsum ($CaSO_4 \cdot \frac{1}{2}H_2O$). He was also aware of temperature effects (Figure 218 and 219), to better explain laboratory and field observations. However, the "still huge gap" in behavior was theorized as being due to air voids in the column (Figure 220).

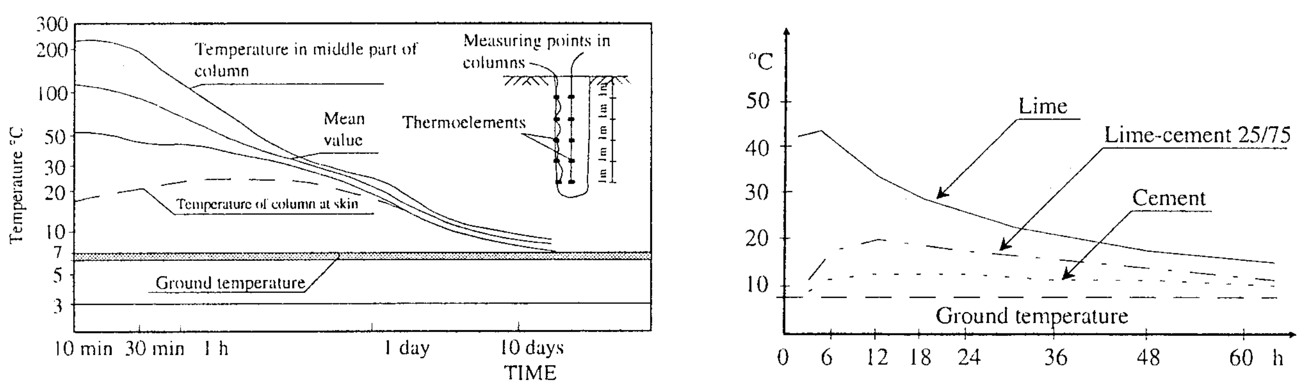


Figure 218. Change of temperature measured for column stabilization: a) stabilization with quicklime, b) the effect of cement on the heat generation in a column (Kujala, 1982).

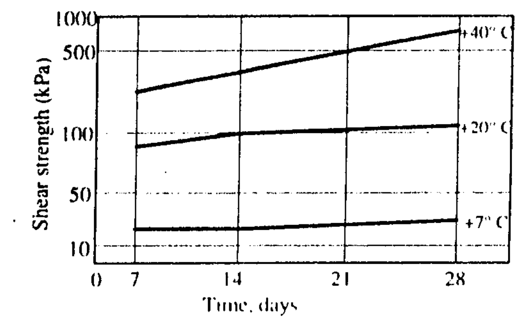


Figure 219. Shear strength gain of stabilized soil as function of temperature environment (Kujala, 1982).

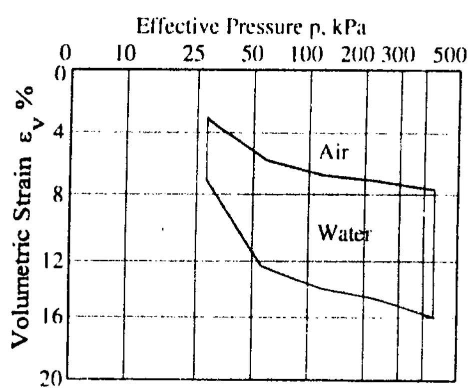


Figure 220. Typical compression curve in an unsaturated stabilized soil (Kujala, 1982).

Holm et al. (1983) continued long-term investigations of lime-gypsum mixes, and recommended mixtures in the range of 3:1 where long-term stability was a concern. Higher lime ratios gave a slower strength increase, but higher final strength (50:50 ratio for temporary stabilization).

Balossi Restelli et al. (1991) describe a DRE field test in 20 m of very soft silty clay prior to embankment construction. The moisture content was 60% (Figure 221). A target U.C.S. of 2.5 MPa was sought in 15,120 lineal meters of columns from 12 to 22 m deep (Figure 222), depending on the height of the subsequent embankments. The cement factor was 290 kg/m^3. Two columns were cored after 2 months and gave strengths (Figure 223) of 1.9 to 4.2 MPa (average 2.5 MPa). A shaft was sunk to check the homogeneity, regularity, and diameter of the columns.

Nishida et al. (1996) reported that quicklime is often used with the DJM system in clays with high water contents. The moisture content of the soil (and the presence of K, Na, Mg salts) controls strength (Figure 224). Microbes may produce sulfides, which inhibit pozzolanic reaction. The presence of dissolvable silica or alumina, i.e., the diatom content of the clays, also influences rate of gain of strength. (Smectites are better than kaolinites.) Also, the more sensitive soils can be mixed more quickly and efficiently, thus providing higher treated strengths. The research focused on the impact of mixing energy: the strength ratio of field to the laboratory samples was usually 33% and was felt to be strongly influenced by this (Figure 225). However – and very significantly – when energy considerations were made (Figure 226) this ratio was much closer to 100%. Figure 227 shows data relating cost, strength, and degree of mixing for lime-stabilized soft clay. They concluded that it was more cost effective to mix longer and to minimize cement factor, to achieve target strengths.

Huttunen et al. (1996) reported on a full-scale 1993 test conducted in highly compressible peats overlying clays in Finland by the Finnish Roads Administration Research and Development unit. The two binders used were "Finnstabi" (a gypsum-based by-product) and portland cement, and a mixture of portland cement and blast furnace slag. The clay was treated with a mixture of Finnstabi and lime, and also a portland cement/lime mixture. Samples were taken as in Table 64.

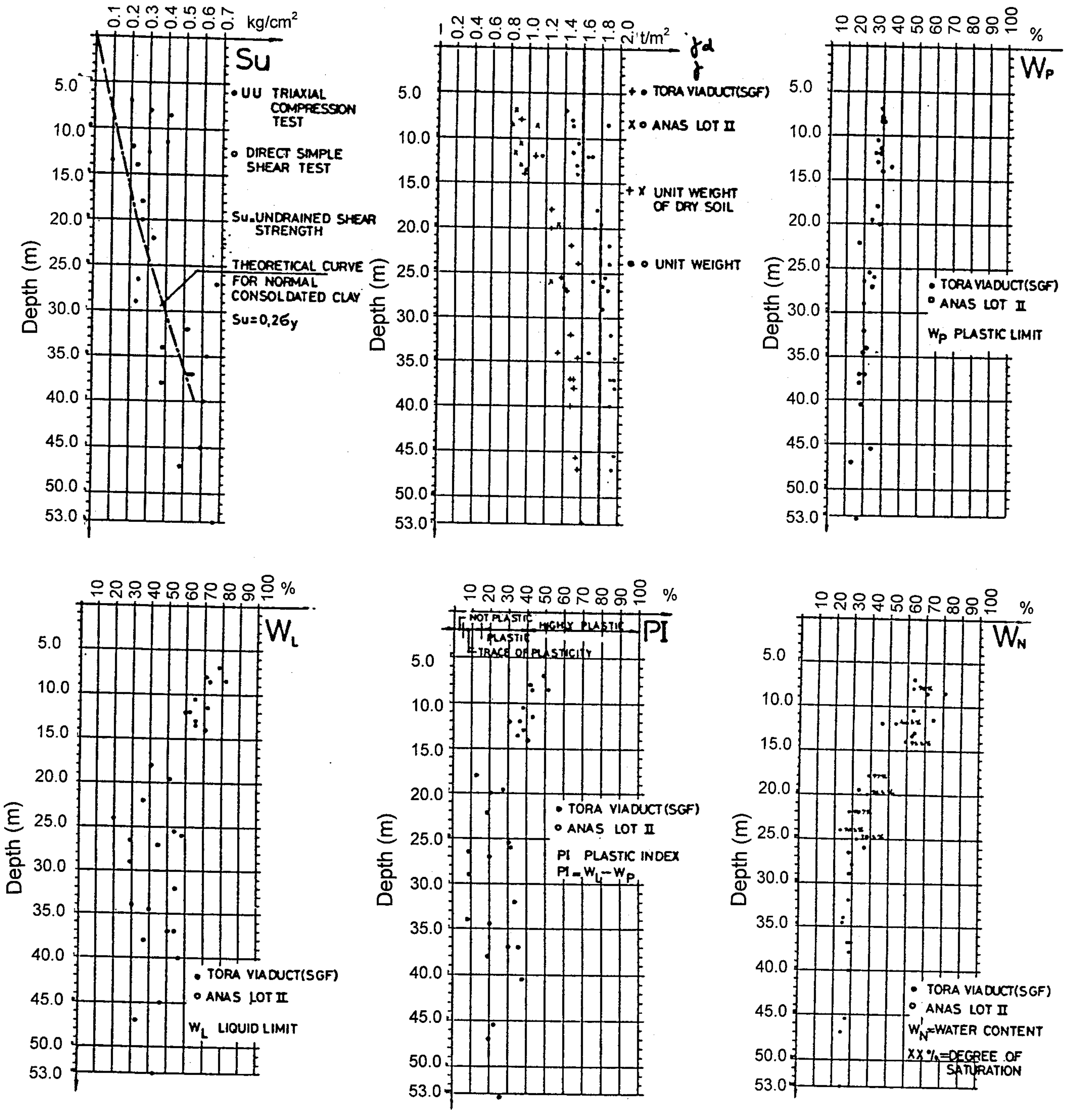


Figure 221. Soil investigation data: unit weight, water content, Atterberg limits, and undrained shear strength (Restelli et al., 1991).

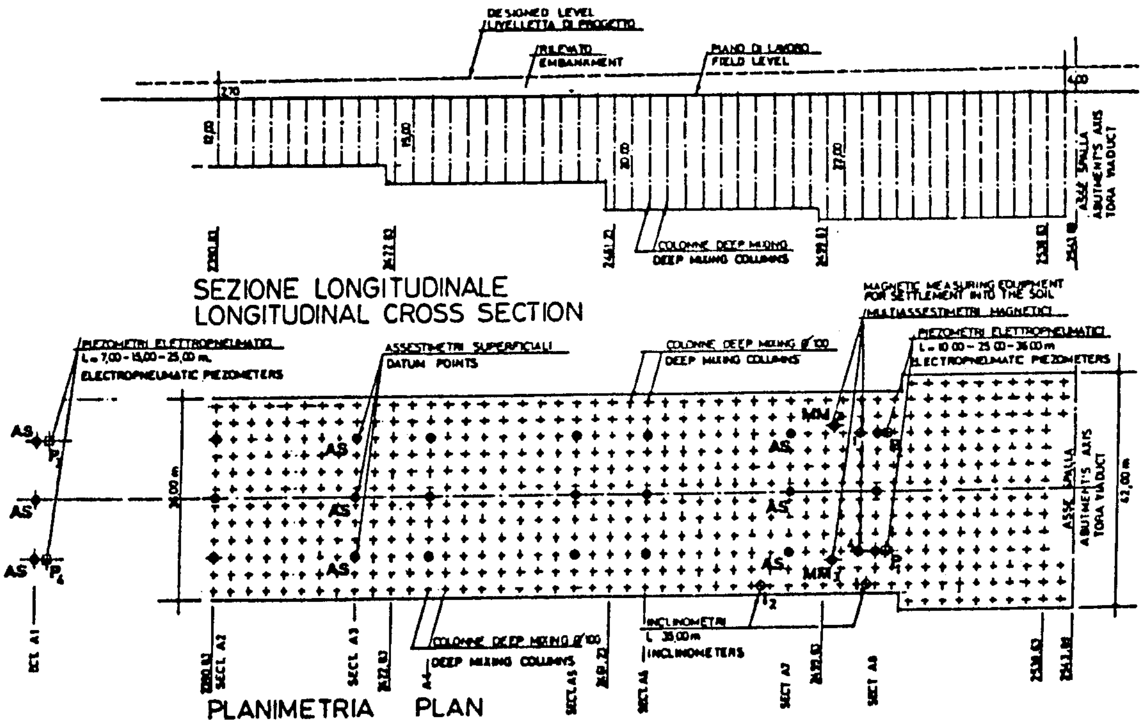


Figure 222. Layout of the deep mixing treatment near Tora Viaduct and of the site control instrumentation (Restelli et al., 1991).

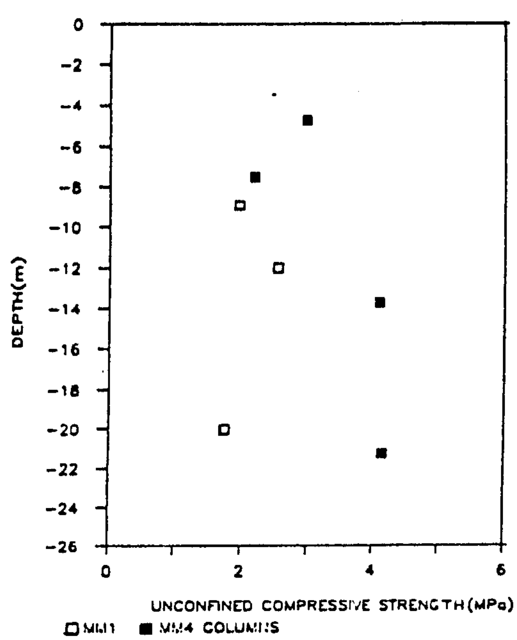


Figure 223. Strength characteristics from laboratory tests on samples recovered in direction parallel to the axis of the columns (Restelli et al., 1991).

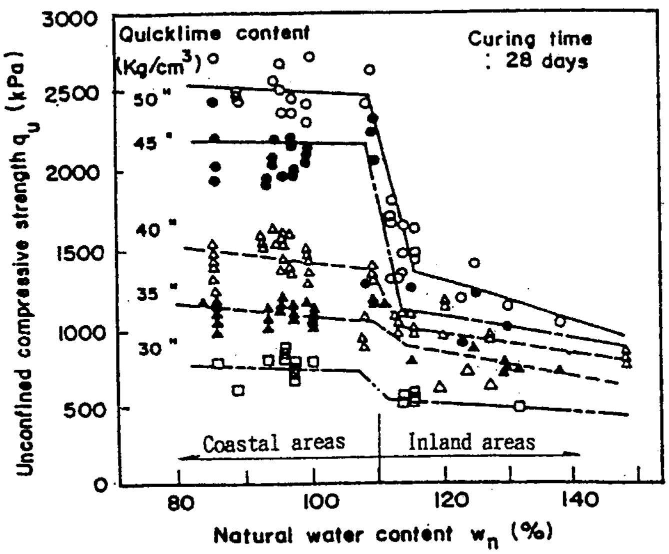


Figure 224. Influence of natural water content and quicklime content on improvement effect (Nishida et al., 1996).

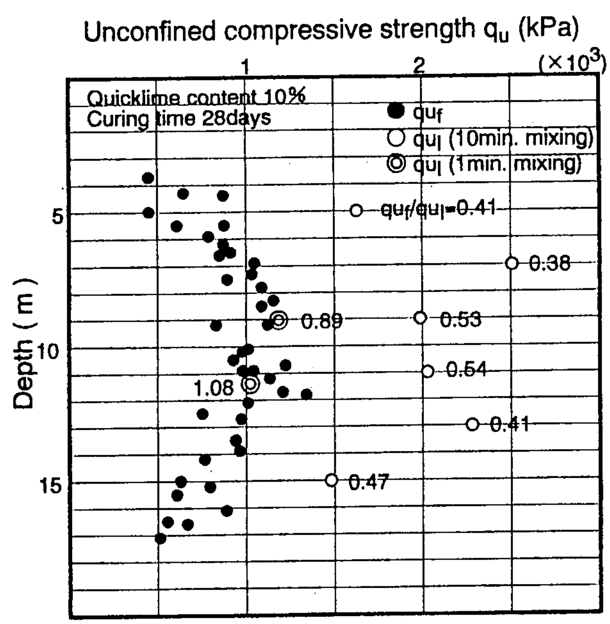


Figure 225. Comparison of q_u in field and laboratory (Nishida et al., 1996).

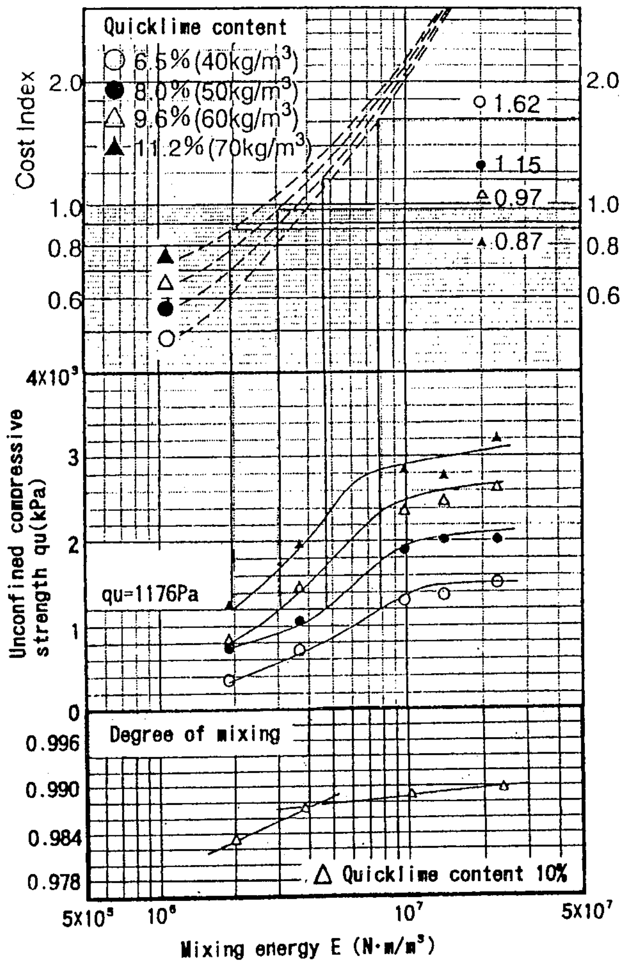


Figure 226. Relationship between q_{uf}/q_{ul} and E_f/E (Nishida et al., 1996).

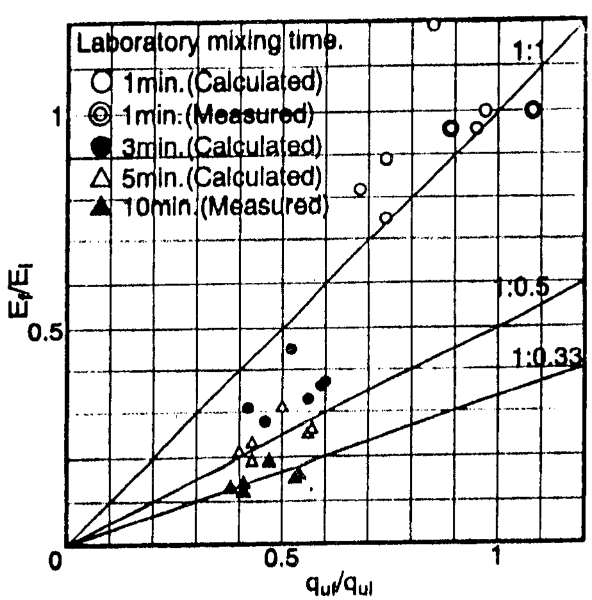


Figure 227. Relationship between improved strength (q_u), mixing energy (E), and construction cost index for DJM construction method (Nishida et al., 1996).

Table 64. Data on the uplifted samples examined (Huttenen et al., 1996).

Sample number	Material	Stabilization method	Binder	Binder content
1	Clay	Deep stabilization	P+CaO 2:1	48 kg/m
2	Clay	Deep stabilization	F+CaO 1:1	48 kg/m
3	Clay	Deep stabilization	F+CaO 1:1	48 kg/m
4	Peat	Mass stabilization	P+F 1:1	250 kg/m^3
5	Peat	Mass stabilization	P+BFS 1:1	250 kg/m^3
6	Peat	Not stabilized	-	-
7	Peat	Deep stabilization	P+F 1:1	95 kg/m
8	Peat	Deep stabilization	P+BFS 1:1	114 kg/m

Legend:
P Portland cement
BFS Blast furnace slag
F Finnstabi (gypsum based by- product)

Data for the treated peat are provided in Figures 228 and 229 and Tables 65 and 66. The columns had mean shear strengths of 30 to 120 kPa, with significant gain in strength between 30 days and 1 year. The effective friction angle was found to be greatly dependent on the binder: ϕ using OPC and slag was 10° greater than using OPC and Finnstabi. Water content was much lower, and pH changed from acid to highly alkaline. Permeability varied from 10^{-6} to $10^{-7.8}$ m/s, and horizontal permeability was significantly greater than vertical.

Data for the treated clay are provided in Figures 230 and 231 and in Table 67. Drilling and vane shear tests gave mean shear strengths at 1 year of 80 to 120 kPa. Uniaxial tests gave shear strengths of 130 to 234 kPa; cohesion was 7 to 60 kPa; and friction angle (ϕ) was 37 to 61°. Permeability was $10^{-7.8}$ to $10^{-8.5}$ m/s. The dynamic cone penetrometer proved to be a "practical and rapid means" of determining variations in the strength of the treatment.

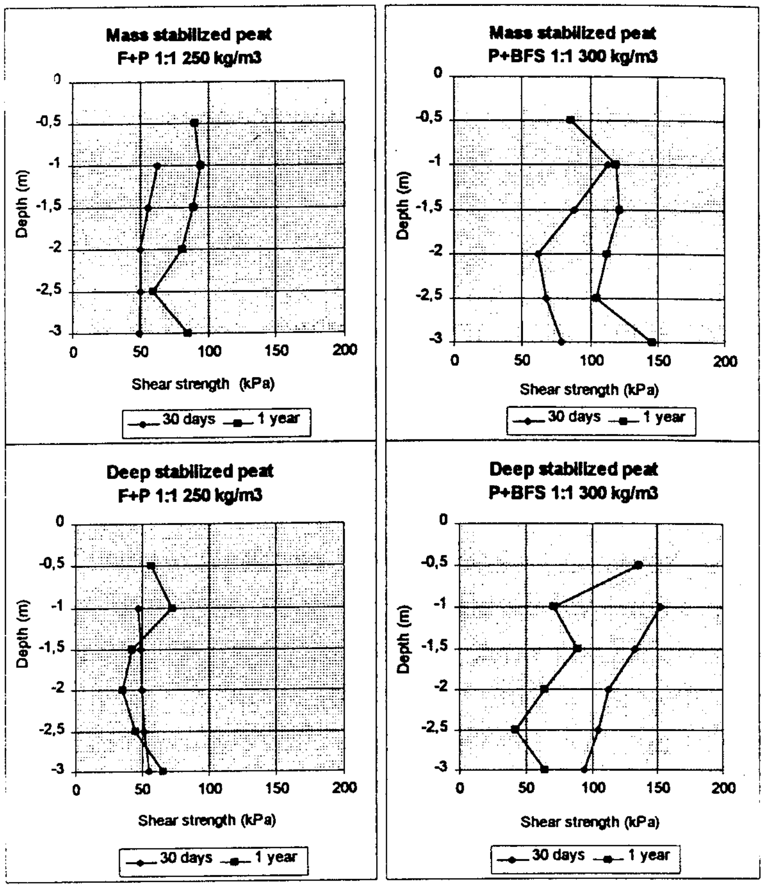


Figure 228. Shear strength of stabilized peat at Veittostensuo as determined by column drilling and vane shear tests after intervals of 30 days and 1 year (Huttenen et al., 1996).

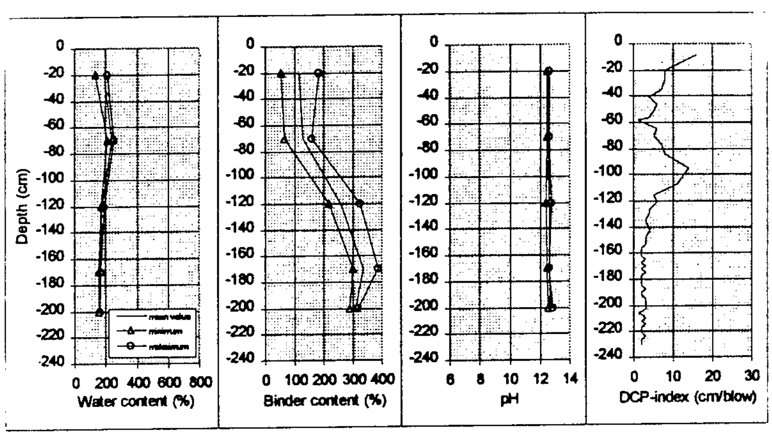


Figure 229. Variations in water content, binder content, and pH on the shear surface of a mass stabilized peat sample and DCP index as a function of depth in the corresponding mass stabilized zone (Huttenen et al., 1996).

Table 65. Parameters of the stabilized peat (Huttenen et al., 1996).

Sample / Binder / binder content	Stabili-zation method	Effective cohesion c' (kPa)	Effective friction angle ϕ' (°)	Shear strength τ (kPa)	Modulus of elasticity E (kPa)
4 / P+F 1:1 / 250 kg/m³	mass stabili-zated	46,2...83,5	28,5...29,5	27,2...113,7 33,2(*)	1746...7906 1686(*)
5 / P+BFS 1:1 / 250 kg/m³	"	37,4...48,5	36,8...41,3	-	-
7 / P+F 1:1 / 95 kg/m	deep stabilized	26,4	21,0	-	-
8 / P+BFS / 114 kg/m	"	62,3	28,7	-	-

*horizontal sample

Table 66. Permeabilities of the stabilized peat in a saturated state (Huttenen et al., 1996).

Sample / binder/ binder content	Stabili-zation method	Water content w (%)	Unit weight γ (kN/m³)	Perme-ability k (m/s)
4/P+F 1:1 / 250 kg/m³	mass stabilized	174...198	11,30...12,31	$10^{-6,3}...10^{-6,7}$
5 /P+BFS 1:1/ 250 kg/m³	"	99,8...185	10,82...12,69	$10^{-7,2}...10^{-6,0}$
7/P+F 1:1/ 95 kg/m	deep stabilized	272...310	10,71...11,31	$10^{-7,8}...10^{-7,2}$ $10^{-6,1}$(*)
8/P+BFS/ 114 kg/m	"	99,5...350		$10^{-6,2}$

*horizontal sample

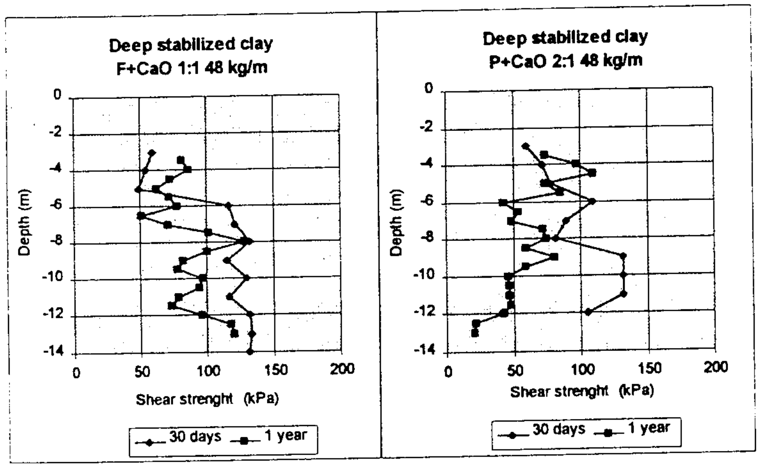


Figure 230. Shear strength of stabilized clay at Veittostensuo as determined by column drilling and vane shear tests after intervals of 30 days and 1 year (Huttenen et al., 1996).

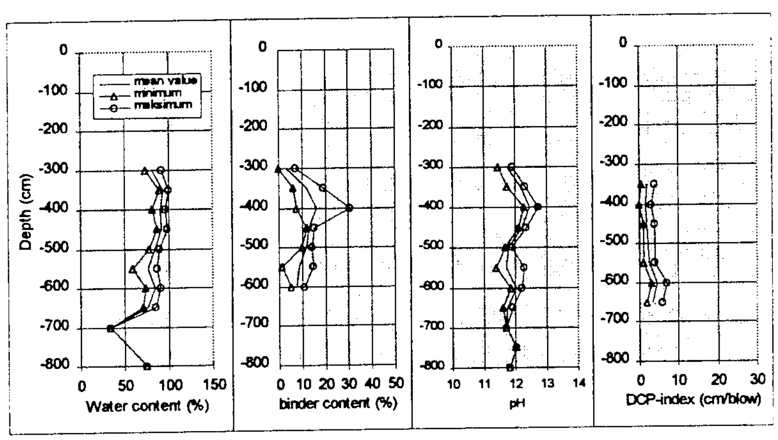


Figure 231. Variations in water content, binder content, pH, and DCP index on the shear surface of a stabilized clay column (Huttenen et al., 1996).

Table 67. Properties of the stabilized clay (Huttenen et al., 1996).

Sample / binder/ binder content	Effective cohesion c' (kPa)	Effective friction angle ϕ' (°)	Shear strenght τ (kPa)	Modulus of elasticity E (MPa)	Perme-ability k (m/s)	Thermal condictivity λ (W/mK)
1 / P+CaO 2:1 / 48kg/m-	7,11... 29.4	48,9... 49,0	130,4... 233,6	13,8... 26,8	$10^{-8,5}$... $10^{-7,8}$	0,55
2 / F+CaO 1:1 / 48 kg/m			150,0	17,2		0,82
3 / F+CaO 1:1/ 48 kg/m	28,8... 60,0	37,1... 60,6			$10^{-8,5}$	0,72

Lin et al. (1999) reported on the use of DMM columns to reduce settlements at bridge abutments built on soft clay near Fuzhou, in China. A field test was conducted to compare the effectiveness of different ground treatment and/or improvement methods (Figure 232). The ground conditions were

0 to 1.5 m	Medium to stiff silty clay
15 m thickness	Very soft marine clay
1 to 2 m thickness	Medium stiff silty clay
0.5 to 2.5 m thickness	Medium dense gravelly sand
4 to 8 m thickness	Dense sand, over weathered granite

The soft clay was found to have a natural water content ranging from 65 to 88%, c_u = 10 to 20 kPa, and k = 4.3 x 10^{-10} m/s. A laboratory test program was conducted using early strength portland cement, and two types of soft clay:

Clay type	Water content (%)	Unit weight (kN/m^3)
Type 1	80 to 83	15.5 to 16.0
Type 2	~ 67.1 to 68.5	15.9 to 16.5

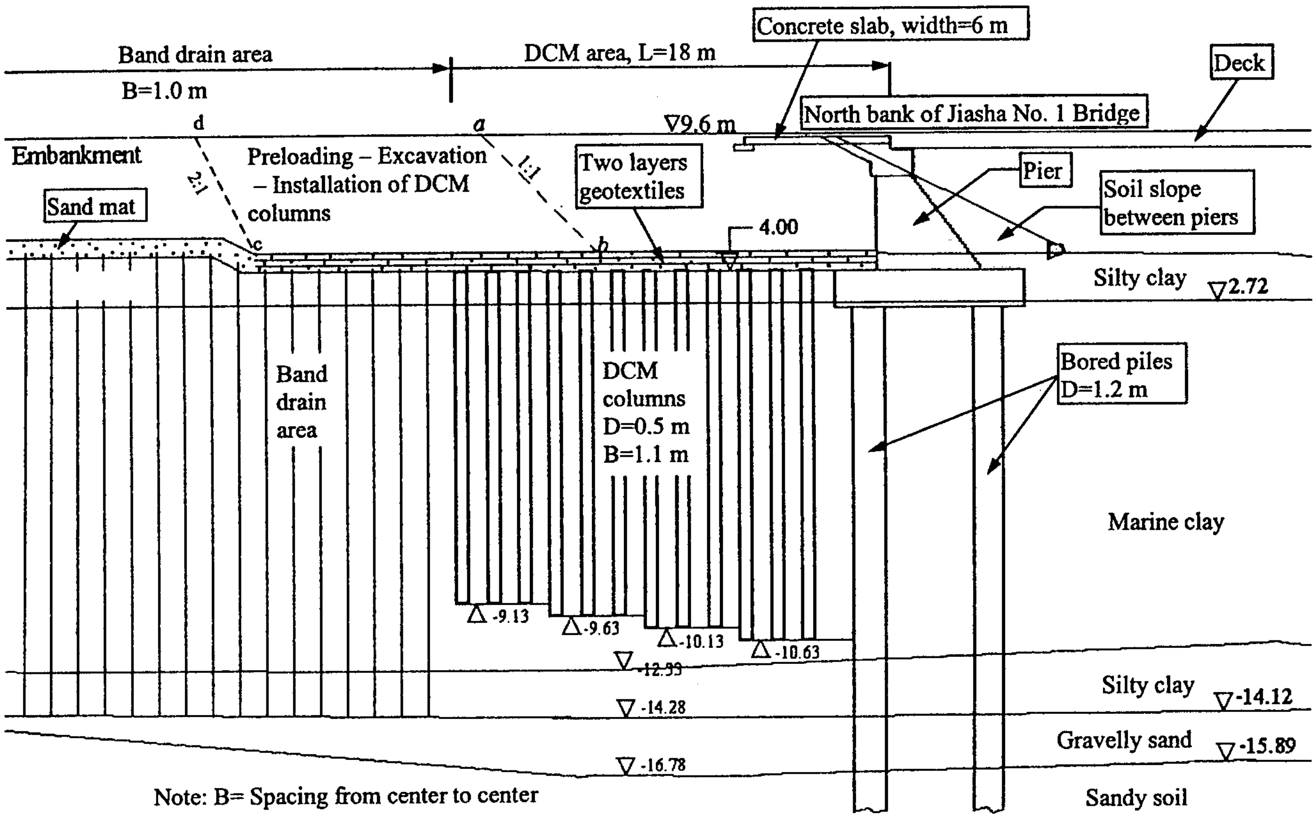


Figure 232. Elevation view of North Approach at Jiasha No. 1 Bridge (Lin et al., 1999).

Tests 1 and 2 featured dry binder in soils 1 and 2, respectively, while Test 3 was on Type 2 clay using slurry of water/cement ratio = 0.4. Data shown in Figure 233 indicate U.C.S. was 150 to 230 times the original clay value; strength is inversely proportional to moisture content (Type 1 had a natural moisture content higher than that of Type 2); and strength is lowest in Test 3.

The field test therefore progressed with dry binder, at a penetration rate of 1.2 m/min and a withdrawal rate of 0.8 m/min. Static cone penetration tests were conducted in one column; samples were taken from a second by excavation, and in situ static loading tests were done in both columns 3 and 4. Tables 68 and 69 summarize the variables.

Figure 234 shows the data for the Static Cone Penetration Test, carried out 24 hours after installation. Until the probe exited the column (at 4 m depth), the mean q_c value of the column was six to seven times that of the clay, but as low as two times.

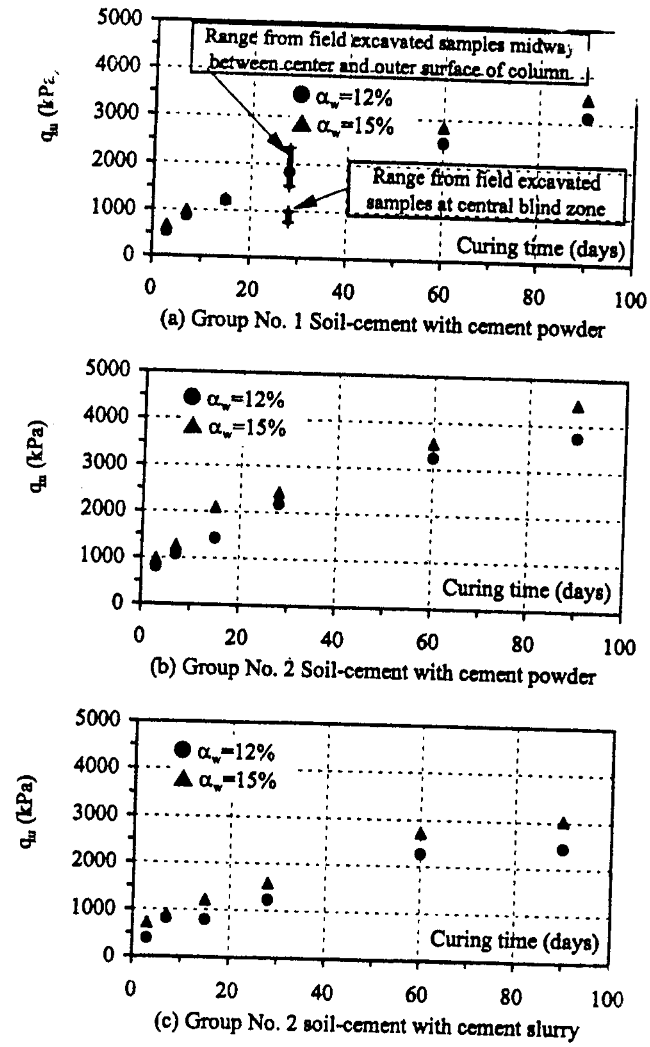


Figure 233. Soil-cement unconfined compression strength vs. curing time (Lin et al., 1999).

Table 68. Basic parameters of four test DCM columns (Lin et al., 1999).

Column (1)	Test item (2)	Average cement content (%) (3)	Column length (m) (4)	Repeated mixing length (m) (5)
1	Static cone penetration test	13.5	9.6	9.6
2	Excavation sampling and unconfined compression test	14.6	9.6	9.6
3	Static loading test	14.3	9.6	9.6
4	Static loading test	9.6	8.6	5.0

Table 69. Cement content versus depth in four test DCM columns (Lin et al., 1999).

Depth (m) (1)	Cement Content Column 1 (%) (2)	Column 2 (%) (3)	Column 3 (%) (4)	Column 4 (%) (5)
0.6–1.6	11.7	15.6	—	8.7
1.6–2.6	15.3	16.4	17.7	12
2.6–3.6	12.8	14.7	18	13.6
3.6–4.6	15	8.5	18.3	10.6
4.6–5.6	16.9	6	13.1	9.8
5.6–6.6	13.9	13.1	7.1	8.5
6.6–7.6	15.6	17.5	11.2	7.1
7.6–8.6	16.6	20.7	15.7	12.0
8.6–9.6	10.0	23.2	15.7	—

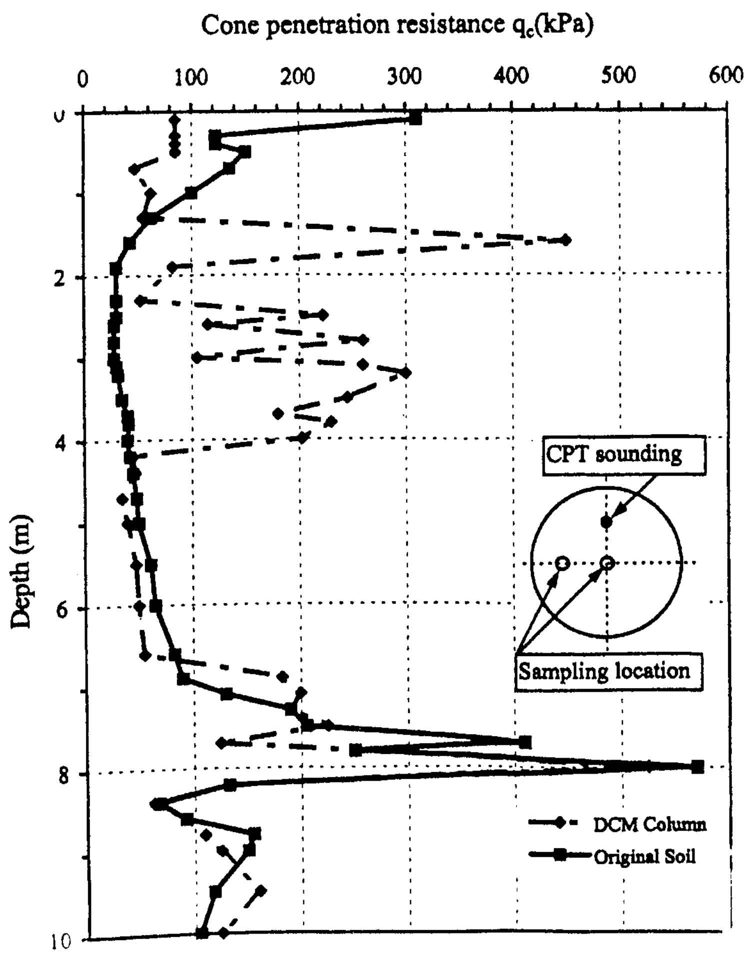


Figure 234. CPT results in original soil and DCM column 1, 24 hours after mixing (Lin et al., 1999).

Excavation, sampling, and U.C.S. were conducted to 3-m depth, and showed Column 2 diameters of 540 to 560 mm, with good continuity. The 28-day core strengths (50 mm) were as in Table 70 and Figure 233. This indicates that for a cement factor of 16.4%, U.C.S. = 1.9 to 2.0 MPa or equivalent to about 60 times the initial U.C.S. of the untreated soft clay. The column had a central "blind zone" left by the withdrawal of the mixing tool, and U.C.S. values there were 0.76 to 1.0 MPa. Below 2-m depth, this weak zone was 40 to 50 mm in diameter and so was only 1% of the total column cross-sectional area.

Table 70. Unconfined compressive strength of soil-cement in field excavated samples (Lin et al., 1999).

Sample (1)	Soil layer (2)	Depth (m) (3)	q_u(28 days) (kPa) (4)
1	Silty clay	0.5–1.0	628
2	Silty clay	0.5–1.0	1,007
3	Silty clay	1.0–1.5	1,169
4	Silty clay	1.0–1.5	1,856
5	Organic silty clay	1.5–2.0	779
6	Organic silty clay	1.5–2.0	974
7	Marine clay	2.0–2.5	2,343
8	Marine clay	2.0–2.5	1,840
9	Marine clay	1.0–2.5	1,883
10	Marine clay	2.0–2.5	996 (center)
11	Marine clay	2.5–3.0	2,241
12	Marine clay	2.5–3.0	1,948
13	Marine clay	2.5–3.0	1,543
14	Marine clay	2.5–3.0	763 (center)

For the subsequent test sections, a cement factor of 15% and a design U.C.S. of 0.6 MPa were chosen. The calculated modulus of the treated soil was 60 to 65 MPa – about 100 times the design U.C.S.

Huiden (1999) reported on laboratory and field tests of both dry and wet mix systems in conjunction with the stabilization of soft, heterogeneous soils under the new Botlek Railway tunnel in the Netherlands. The native soils consisted of very heterogeneous peats, organic clay, and silt overlying dense sand, clearly different from the typical Scandinavian profile. The targets included an E-value of > 20 MPa and a modulus of subgrade reaction > 4 MPa. The goals of the field tests were to provide samples for subsequent laboratory testing; to verify constructability to the target depth (22 m) and into the sands; and to investigate suitable testing methods for the production columns.

The earlier laboratory testing experimented with the fine soils and various dry binders (all with finely ground slag cement and natural anhydrite (Table 71). Shear strengths were high in sands (2 to 3.5 MPa) but low (0.08 to 0.50 MPa) in the peats.

Table 71. Laboratory results of testing on fine soils with various dry binders (Huiden, 1999).

Layer [depth in m below GL]	Amount of binder and receipt*	Shear strength at ... days [MPa]			Youngs Modulus at ... days [MPa]			Strain at failure	Volume wght. kg/m³
		7	14	28	7	14	28	28 days	
1	150A	0.8	1.0	1.4	230	350	580	0.94%	1670
[Clay]	200A	1.3	1.5	1.8	620	700	840	0.74%	1690
[-13.00]	200B	---	---	1.2	---	---	540	0.84%	1700
2	150A	1.6	1.8	2.0	610	870	1160	1.10%	1909
[Sand]	200A	---	---	3.5	---	---	1840	0.90%	1920
[-16.50]	200B	1.6	1.7	2.0	860	940	1060	1.10%	1925
	200 C	---	---	2.6	---	---	1360	1.10%	1920
3	150A	0.6	0.6	0.7	120	150	340	1.60%	1689
[Clay]	200A	1.3	1.5	1.8	370	600	830	1.20%	1693
[-18.50]	200B	---	---	1.1	---	---	530	0.69%	1700
4	150A	0.42	0.8	1.8	148	428	1080	0.58%	1851
[Sand]	200A	---	---	2.5	---	---	1510	0.62%	1852
[-20.00]	200B	1.0	1.3	1.5	417	640	750	0.81%	1884
	200C	---	---	2.0	---	---	1100	0.71%	1862
5	150A	0.03	0.06	0.08	2	5	7	3.20%	1458
[Peat]	200A	0.18	0.31	0.5	30	68	155	0.52%	1480
[-22.00]	200B	---	---	0.26	---	---	45	0.78%	1462

* 200 A means: 200 kg of binder for each m³ of soil; A is the type of binder.
* Binder A: 80% m/m slag cement CEM III A 42.5; 20% m/m natural anhydrite
Binder B: 60% m/m slag cement CEM III A 42.5; 40% m/m natural anhydrite
Binder C: 70% m/m slag cement CEM III A 42.5; 30% m/m natural anhydrite

The subsequent field test involved 23 columns of 600 mm in diameter to 22 m depth (Figure 235) with a cement factor of 200 kg/m^3 (80:20 slag cement:anhydrite). Testing was conducted with CPT tests, BAT probes, and two core holes. Core resistance was improved by 2.5 to 3 times.

The 500-mm-wide BAT probes (assumed equivalent to FOPS, i.e., reverse probes) worked in the clay and peat horizons but were unsuccessful in treated silts. The core samples proved highly variable, but good samples gave undrained shear strengths from U.C.S. of 0.56 to 4.25 MPa. Pocket penetrometer testing indicated minimum strengths of 0.45 MPa except in one – probably peaty layer. Overall, the very few core strengths in peat were only 5% of those obtained in the laboratory, and strengths in clay and sand were 50% of laboratory values. BAT probes showed a 60 to 70% ratio (Table 72).

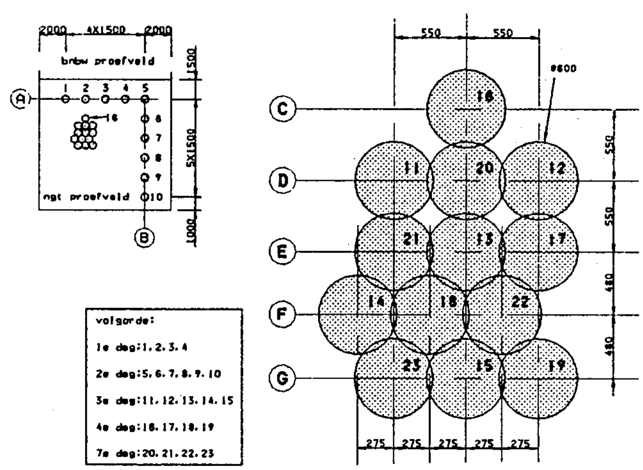


Figure 235. Layout of field test (Huiden, 1999).

Table 72. Results of testing on borehole samples and from BAT probes (Huiden, 1999).

Elevation [m below GL]	Shear strength from borehole samples [MPa]	Shear strength from BAT-probes [MPa]
- 8.00...-10.00	0.06...0.07	
-10.00...-11.00		1.2
-12.00...-14.50	0.60...0.80	1.0...1.7
-18.00...-19.00	2.00...2.80	1.6

Comparing the production and quality control aspects of wet and dry columns, Huiden concluded inter al.:

- Wet mixing gives better homogeneity because of longer mixing and prehydration of the cement.
- Dry mixing has no spoil.
- Coring of such columns is not an adequate quality assurance method.
- Quality control in stratified soil is very difficult in dry methods (CPT and BAT), although in wet methods, wet grab sampling is possible.

Afterwards, the client opted to use the wet method.

Hayashi and Nishikawa (1999) provided a most interesting study on the various factors affecting the mixing efficiency of dry mixing methods as frequently used in peaty soils in Northern Japan. Initially, mixing produced highly variable strengths and some portions of columns did not solidify. "In those days, larger amounts of stabilizer…" were used, but these rendered the process uneconomical. Laboratory and field tests, focusing on mixing efficiency, were therefore run.

In the laboratory tests, local peat (Table 73) was mixed with three (undisclosed) types of binder; and 7 days of wet curing, the samples were subjected to unconfined compression testing. For each binder, strength increased with mixing time, more so for higher cement factors (Figure 236), and the consistency of data improved with time (Figure 237).

Table 73. Engineering characteristics of the peat (Hayashi and Nishikawa, 1999).

Natural water content	(%)	630
Density of soil particles	(g/cm^3)	1.60
Ignition loss	(%)	81.4
pH		6.1

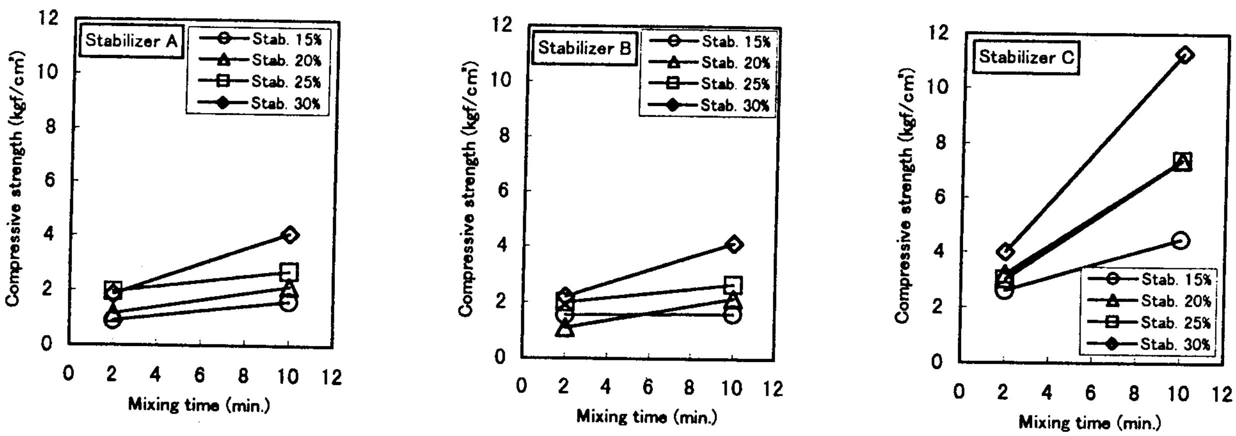


Figure 236. Mixing time and unconfined compressive strength
(Hayashi and Nishikawa, 1999).

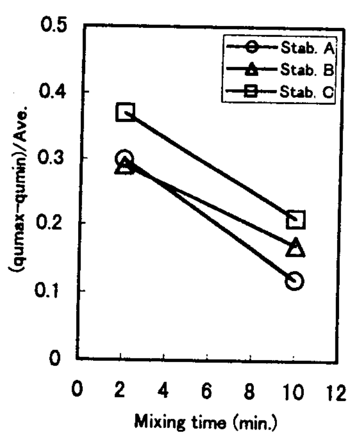


Figure 237. Mixing time and dispersion of unconfined compressive strength
(Hayashi and Nishikawa, 1999).

The subsequent field tests were conducted in the soils of Figure 238, wherein the uniformity of the peat was quite high. A special "high organic soil" binder was used, injected – somewhat atypically for the DJM system – both during penetration and withdrawal. The experimental conditions are shown in Table 74. The "mixing level" is expressed as the mixing number times the weight of binder per meter depth:

$$T = N \times (R_p/S_p \times W_i/W + R_w/S_w)$$

where T = number of mixing times per meter in depth (times/m).

N = total number of mixing blades (=4).

S_p, S_w = penetration and withdrawal rates (m/min).

R_p/R_w = rotation speeds of mixing blades during penetration and withdrawal (rpm).

W_I = stabilizer injection amount during penetration (kg/m^3).

W = total amount of stabilizer (kg/m^3).

A mixing number of 312/m was initially chosen – the same as for clays, but with the knowledge that "peaty soft ground is problematic." Values of 411 to 456/m were achieved by high-speed (48 rpm) rotation – twice the "conventional" rate. To provide 624/m, two penetrations were carried out. Triple tube samples were taken 25 cm from the column center and subjected to unconfined compression testing (Figure 239). For each cement factor, strength increased with mixing number. At 411 to 456/m more compressed air was recovered, contributing to higher in situ strength. Uniformity (Figure 240) was also increased with mixing number. Regarding this uniformity issue, the authors quoted a formula:

$$q_{ue} = \overline{q_u} - 1/2 \times \sigma$$

where q_{ue} = evaluated strength of improved columns (kgf/cm^3).

$\overline{q_u}$ = mean value of unconfined compressive strength (kgf/cm^3).

σ = standard deviation of unconfined compressive strength (kgf/cm^3).

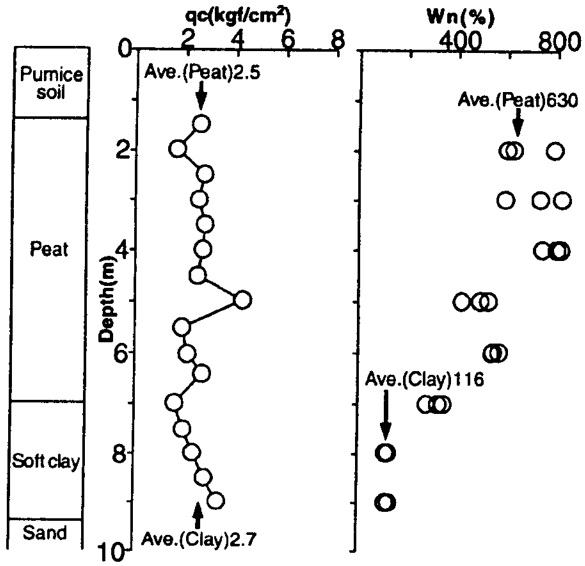


Figure 238. Geological log and engineering characteristics at the site (Hayashi and Nishikawa, 1999).

Table 74. Experimental conditions (Hayashi and Nishikawa, 1999).

Stab. amount (kg/m^3)	Rp (rpm)	Sp (m/min.)	Rw (rpm)	Sw (m/min.)	Rounds of mixing	T (times/m)
200, 250, 300	24	1.0	48	1.0	1	312
	48	0.7	48	0.7	1	411-456
	24	1.0	48	1.0	2	624

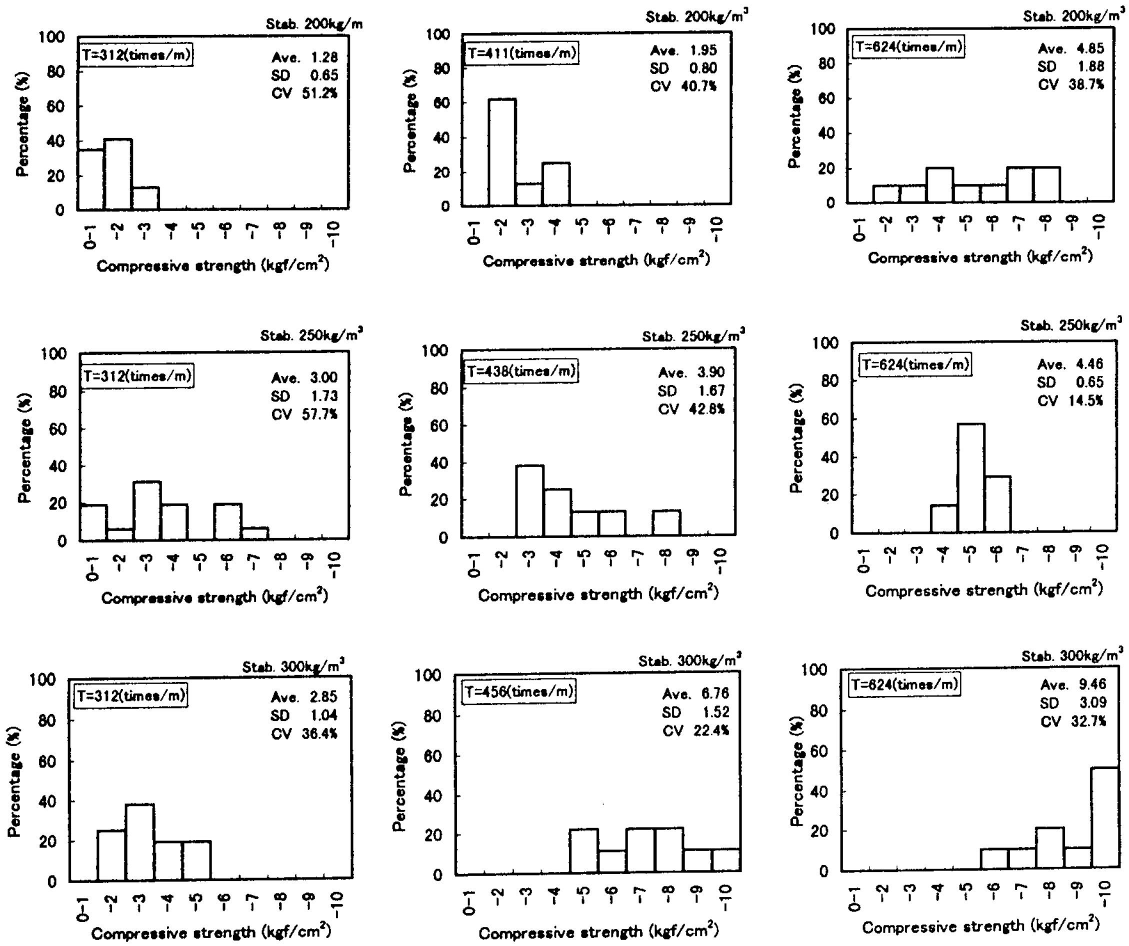


Figure 239. Histogram of column strength on day 28 (Hayashi and Nishikawa, 1999).

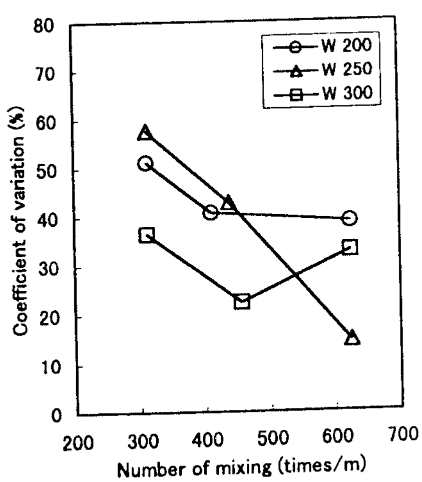


Figure 240. Number of mixing and coefficient of variation in strength (Hayashi and Nishikawa, 1999).

As shown in Figure 241, for the conventional mixing number (312/m), a cement factor of 250 kg/m^3 was needed to provide the target design strength (0.2 MPa), whereas for 438/m, the required cement factor was reduced to 210 kg/m^3.

The authors concluded that mixing efficiency – as opposed to cement factor – was the key to treatment of peaty ground: studies showed that the most cost-effective range was 400 to 500/m.

Regarding the relationship between laboratory and field strength tests, the ratio can usually vary from 2 to 5, and a typically selected value is 3. As shown in Figure 242, this ratio depends on mixing number (i.e., consistency), and therefore varies substantially. The ratio of 3 corresponds to a mixing number of 450/m, whereas at 300/m the ratio is as high as 6 to 8.

This approach has been followed in practice with "no major problems."

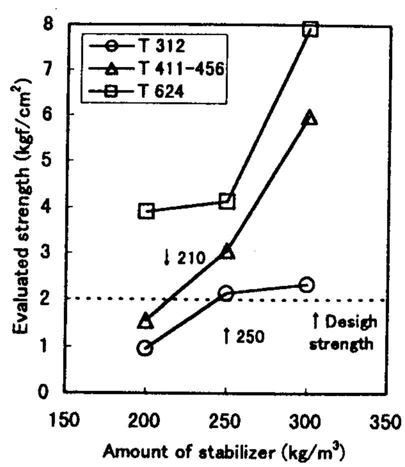


Figure 241. Stabilizer injection amount and evaluated strength of the improved columns (Hayashi and Nishikawa, 1999).

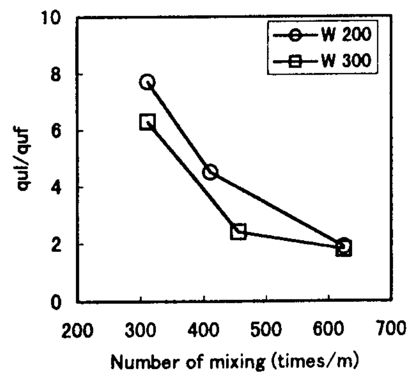


Figure 242. Number of mixing and ratio of column strength between laboratory and field (Hayashi and Nishikawa, 1999).

5.5 General Overview

As for the wet methods, a huge and growing value of excellent work continues to be published, mainly arising from national research projects being conducted in the Nordic countries. Furthermore, a recent conference in Stockholm in 1999 was devoted to the dry mix methods and attracted significant contributions from an international audience.

Much dry mixing is conducted in very soft clays with high moisture and organic contents with a variety of binder types. Experimental data tend to be voluminous, extremely detailed, and challenging to compare and synthesize, reflecting, in fact, the intricacies of clay soil physics, chemistry, and mechanics in general.

These – and other caveats discussed in Section 4.5 notwithstanding – the following broad observations can be drawn:

a) The major controls over treated soil properties appear to be:

- The soil (especially water and organic contents).
- Type and amount of binder.

- Curing temperature.
- Effective in situ stress.
- Age (initial reactions are hydration related, later reactions are pozzolanic).
- Mixing efficiency.

b) Cohesive soils with moisture contents between 60 and 200% are best suited for dry mixing, although successful examples of treatment have been recorded even beyond this range.

c) High soil sulfate contents, and/or high organic contents (and therefore low pH) inhibit strength development, whereas data indicate that the presence of chloride ions (as in salt water) enhances pozzolanic activity and hence increases strength. In addition, little improvement can be expected in soils with over 1.5% humus content, and the effect of humus content on strength is especially marked in coarser grained materials.

d) Dry methods typically use lower binder (usually cement) factors than wet methods, and produce minimal spoil, or binder wastage. Binder factors typically range from 150 to 400 kg/m^3 in Italy and Japan, but are usually 80 to 150 kg/m^3 in Nordic countries where treated soil strength targets are usually much lower.

e) In Japan, ordinary portland cement (OPC), slag cement, and "cement based reagents" comprise about 90% of the materials used, while quicklime is successfully used in marine clays of high moisture contents (over 80%). In Sweden, practice has evolved toward lime-cement blends (usually 50:50), while about 20% of Finnish practice features proprietary binders from the iron and steel industry instead. Tests indicate that blast furnace slag cement and anhydrite cause higher strength in organic clay than lime-cement blends.

f) The Italians and Japanese report U.C.S. values from 0.3 to 7.0 MPa and a higher rate of gain of strength for dry methods than wet methods (28-day strengths = 1.5 times 7-day strengths for all soils). The Nordic countries focus on undrained shear strength (50% U.C.S.) and quote values of 0.10 to 1.0 MPa as being achievable (typically only values at the lower end needed). They find for lime cement that 28-day strengths = 2.4 times 7-day strengths.

g) Regarding permeability, the Japanese observe that the permeability of treated ground is higher than that produced by wet methods (treated soils are "semi-permeable"). The Scandinavians indicate that lime columns are 1000 times more permeable than clay, and that lime-cement columns are 400 times more permeable. Permeability of lime columns may increase with time, but decrease for lime cement columns. Cement columns do not function as drains.

h) With regard to the intensive work on lime cement columns in Scandinavia (and more recently in the United States), the following statements can be made:

- Undrained shear strength of treated highly organic soils is about 10 times the virgin value, rising to 50 times in silty clays.
- Short-term strength is due to the reduction in soil moisture content resulting from the hydration reactions, while longer term strength is due to pozzolanic reactions.
- Φ' varies generally from 25 to 45°, and a value of 30° can be chosen for a normal pressure of 150 kPa.
- Undrained shear strength is 34 to 91% higher than that indicated by U.C.S. (i.e., 50% U.C.S.) when the normal pressure exceeds 150 kPa.

In contrast the Japanese report than shear strength is 33 to 50% U.C.S., the ratio being inversely related to U.C.S. (i.e., at U.C.S. less than 1.5 MPa, the ratio is 50%). The Chinese reported that U.C.S. can be 60 to 230 times virgin values (in the range of 0.5 to 4.5 MPa).

i) Tensile strength (splitting) is 10 to 20% U.C.S. in the range of 0.3 to 2.0 MPa.

j) The relationship between E-value and shear strength generally indicates ratios from 100 to 600, being higher in the short term than in the long term, and lower for lime-cement mixes than for cement alone. The ratio may increase with strength.

k) As in all DMM techniques, dry methods must strive to produce a homogeneous product in order to provide acceptable, homogeneous, and predictable treatments and closer correlation between laboratory and field data. In this regard, the basic investigations being conducted by the Japanese in particular on mixing efficiency are extremely valuable, especially with respect to the treatment of peaty soils where otherwise the use of exceptionally high binder factors would render DMM uneconomic.

6. STRESS-STRAIN BEHAVIOR OF SOILS STABILIZED BY DEEP MIXING

6.1 One-Dimensional Compression

Stabilizing soil by deep mixing with cement has been demonstrated to substantially reduce soil-cement compressibility in one-dimensional compression tests (oedometer). Figure 243 illustrates the stiffening effect that 25% apparent cement factor has on soft Bangkok clay. For comparison, oedometer test data for untreated Bangkok clay are also shown. The apparent pre-consolidation pressure of the stabilized clay is more than an order of magnitude greater than that of natural clay. At stress levels in the "recompression" range, very little strain occurs with changes in vertical pressure. This stiff response to applied load results from the cementation that binds soil and cement particles together.

Bergado et al. (1996) report that the compression index (C_c) of soil-cement decreases with increasing cement factor up to about a_w = 15%, after which further increases in soil-cement stiffness are minor. Furthermore, stiffness increases with curing time and C_c decreases. This reflects continuing strengthening of the cement bonds and pozzolanic activity in the soil-cement mass. However, when stress level in the oedometer increases to the point where the cementation bonds begin to yield and break, then the soil-cement begins to lose its stiffness, and vertical strains become larger.

The onset of cementation breakdown is termed the "yield point" or "yield stress," which has been reported by the CDM Association (1994) to be about 27% higher than the soil-cement's unconfined compressive strength (Figure 244). When the soil-cement is consolidated to a pressure higher than the yield stress, P_y, then large strains will commence. On unloading and reloading, the soil-cement will "remember" the higher loading and experience small deformations as would soils that are preloaded and then reloaded. Thus, the yield stress on reloading can be greater than would be expected based only on the relation with unconfined compressive strength. However, it is unusual for soil-cement to consolidate at pressures greater than P_y, because the P_y will be of considerable magnitude in all cases except where very low cement factors are used, or possibly where large fill loads are placed.

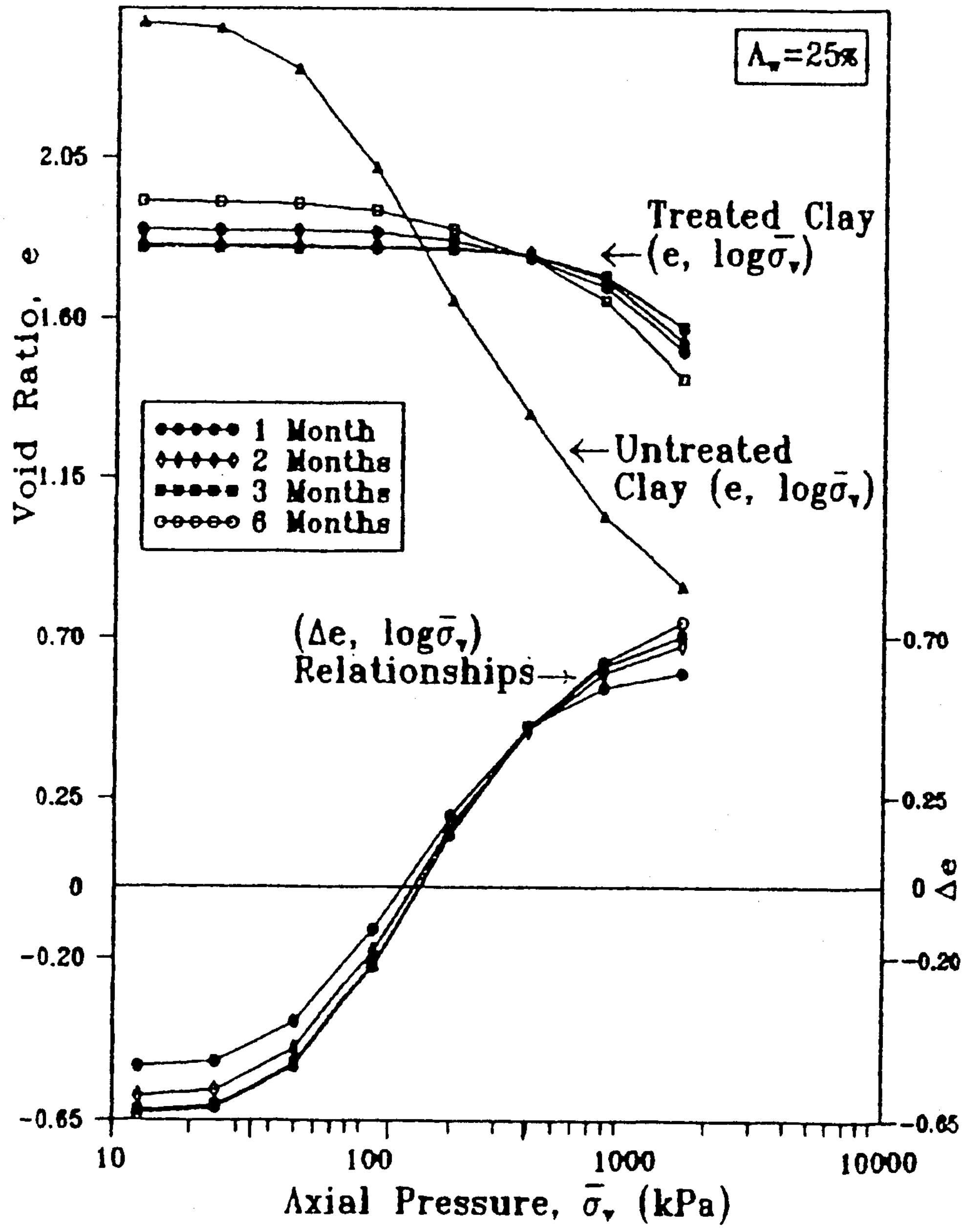


Figure 243. Void ratio vs. axial pressure for soil treated with 25% cement factor (Bergado et al., 1996).

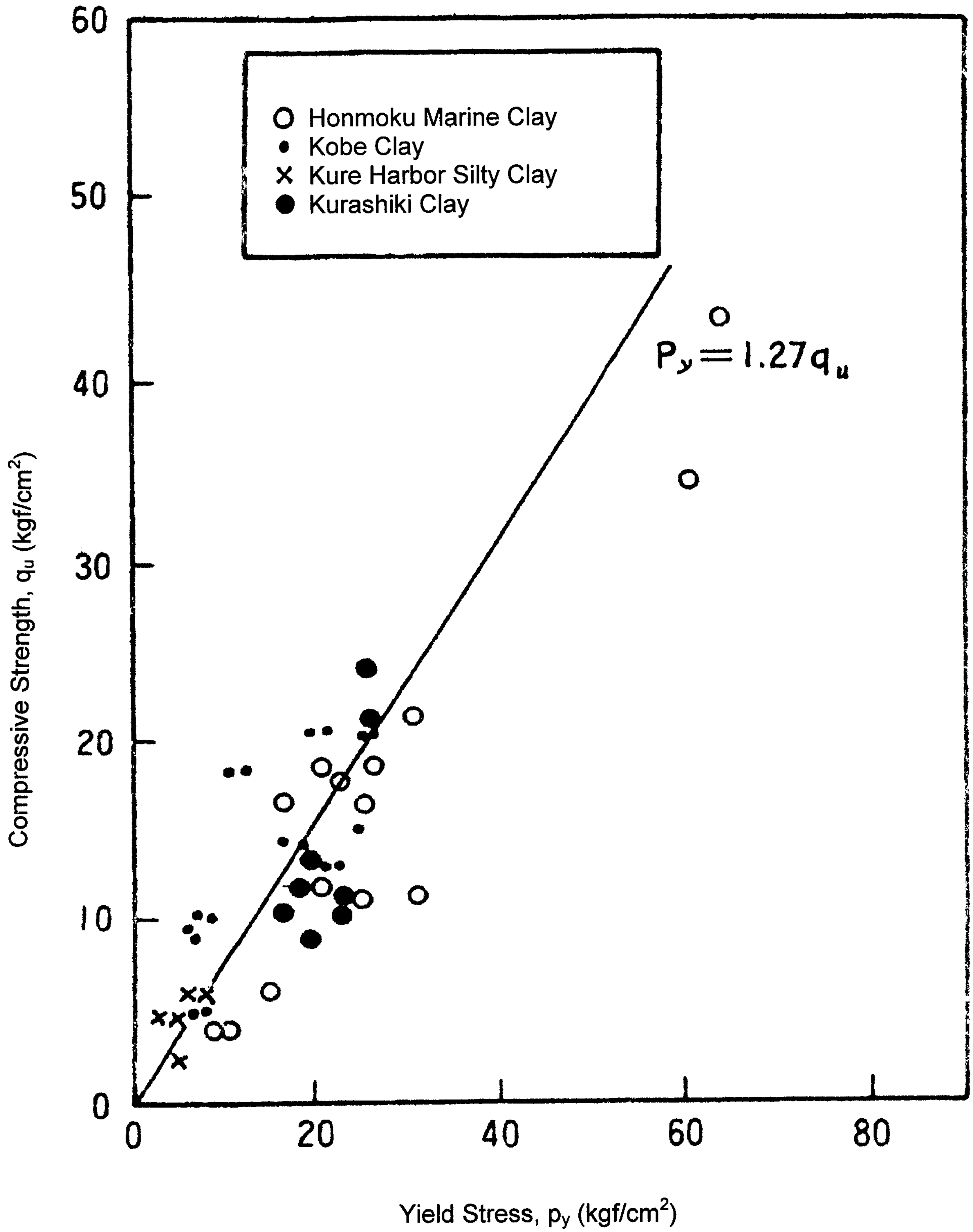


Figure 244. Compression stress vs. yield stress (CDM, 1994).

The influence of cement content on the yield stress is the same as found for unconfined compressive strength. As shown in Figure 245, the yield stress of treated Shinagawa clay increases with apparent cement factor in the same manner as would the unconfined compressive strength. The example in Figure 245 again illustrates that the yield stress will increase greatly, and the difference is more than an order of magnitude when the cement factor is increased from 5% to 15%. It is also noted in Figure 245 that the new post yield behavior (or new "virgin compression" response) becomes more compressible as P_y increases. The ramification is that once the cementation bonds are rendered ineffective by high applied stresses, then very large vertical settlements will occur.

The CDM Design Manual (1994) also provides an example of the effect of cement stabilization on one-dimensional compression behavior, as shown in Figure 246 for stabilized clay beneath Tokyo Harbor. Two cement factors were used, as indicated. Data presented are for both laboratory and field samples. Test results from both environments demonstrate similar behavior, with the higher cement contents producing larger yield stresses.

The treatment with cement or lime and cement will, however, have a residual effect on the soil when the yield stress is exceeded. As the soil-cement reverts to a frictional material, vertical deformations will manifest themselves rapidly. In clay soils, the coefficient of consolidation, c_v, usually decreases as stresses reach and exceed the yield stress. Bergado et al. (1996) report that laboratory tests showed the c_v value increases as apparent cement factor increases to about 15% and c_v remains fairly constant thereafter. They also observed that c_v increased with longer curing times, for up to 2 months, after which little further increase was observed.

Åhnberg (1996) conducted oedometer tests on clayey silt, clay, and clay gyttja soils treated with different binders: lime, lime-cement, and cement. The results (Figure 247) showed that after passing a yield stress, the compression modulus (M) reaches a minimum value, and thereafter increases as stress increases to a value governed by the unstabilized soil. Åhnberg concluded that the behavior of the treated materials did not differ in general during loading for the

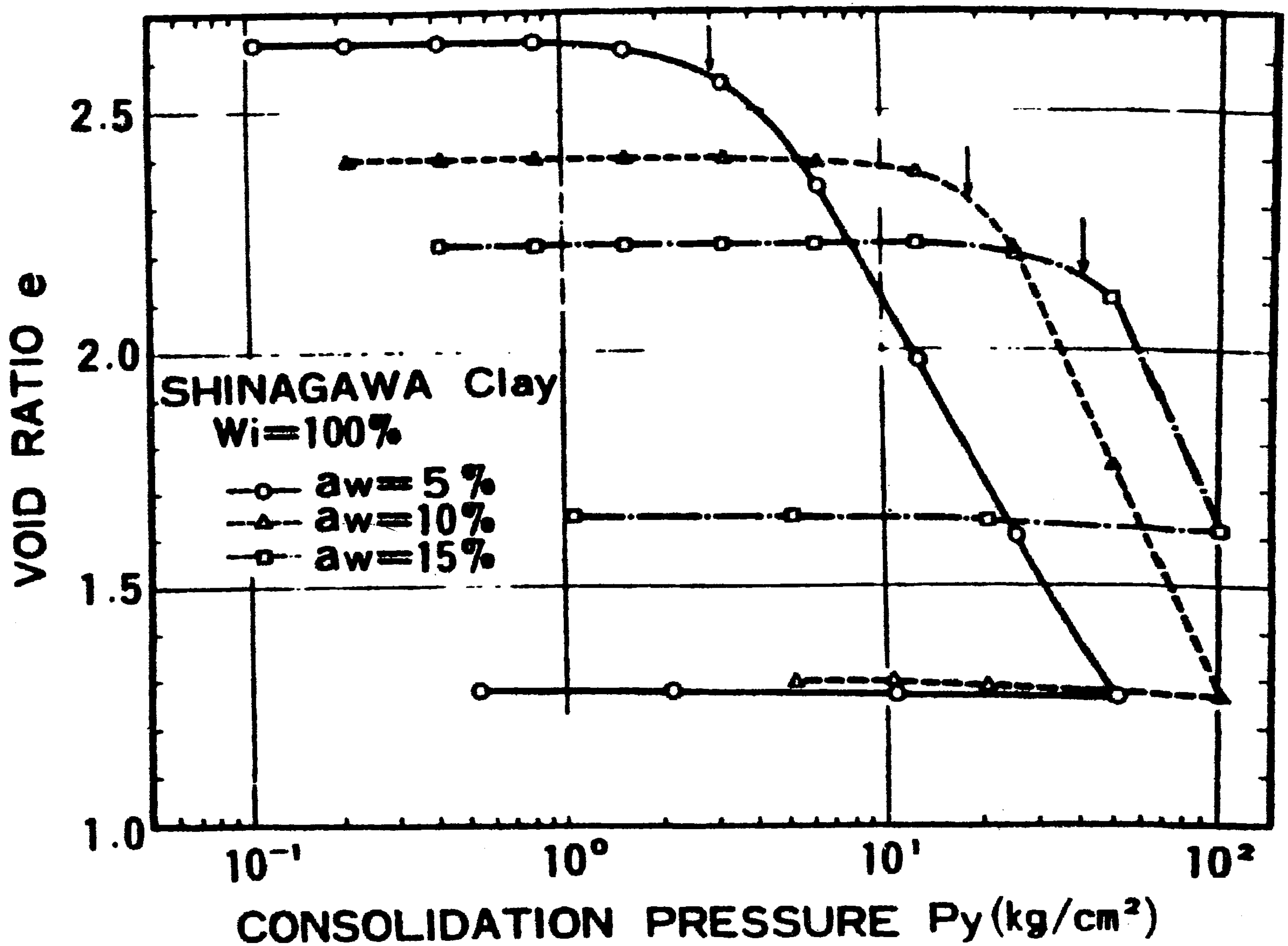
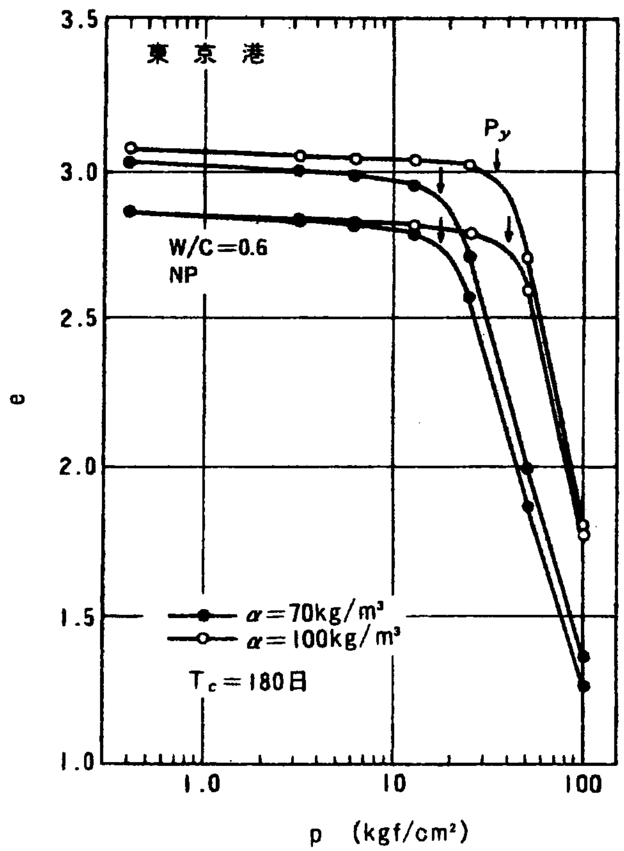
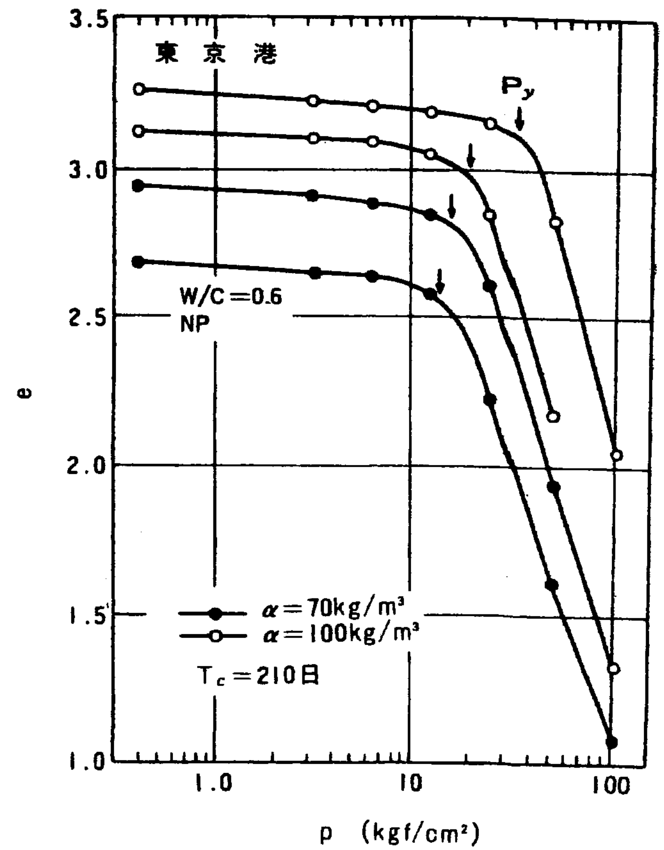


Figure 245. Relationship between void ratio and log P of improved soil (Bergado et al., 1996).



a) laboratory



b) field

Figure 246. One-dimensional compression test results (CDM, 1994).

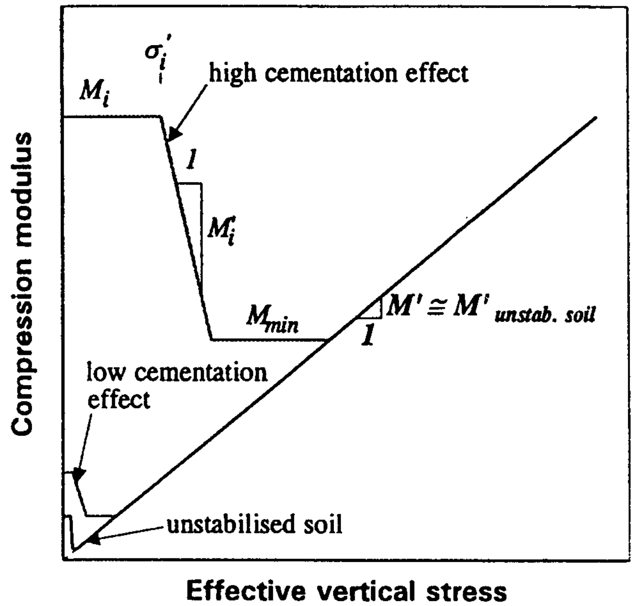


Figure 247. Schematic variation in compression modulus with increasing stresses (Åhnberg, 1996).

various soils and stabilizers, and it "resembles more or less that of a cemented and overconsolidated soil."

6.2 Stress-Strain Behavior in Unconfined Compression

Soil-cement is considered by many engineers to be a brittle material. The stress-strain behavior of soil-cement in unconfined compression is compared schematically with untreated clay in Figure 248. While soil-cement is much stiffer than soft or medium clay, it is not a classically brittle material. Only in unconfined compression does soil-cement lose nearly all of its strength after peak strength is reached. Therefore, the notion that soil-cement is "brittle" is more a manifestation of the common use of unconfined compression to test its strength than it is an actual stress-strain characteristic.

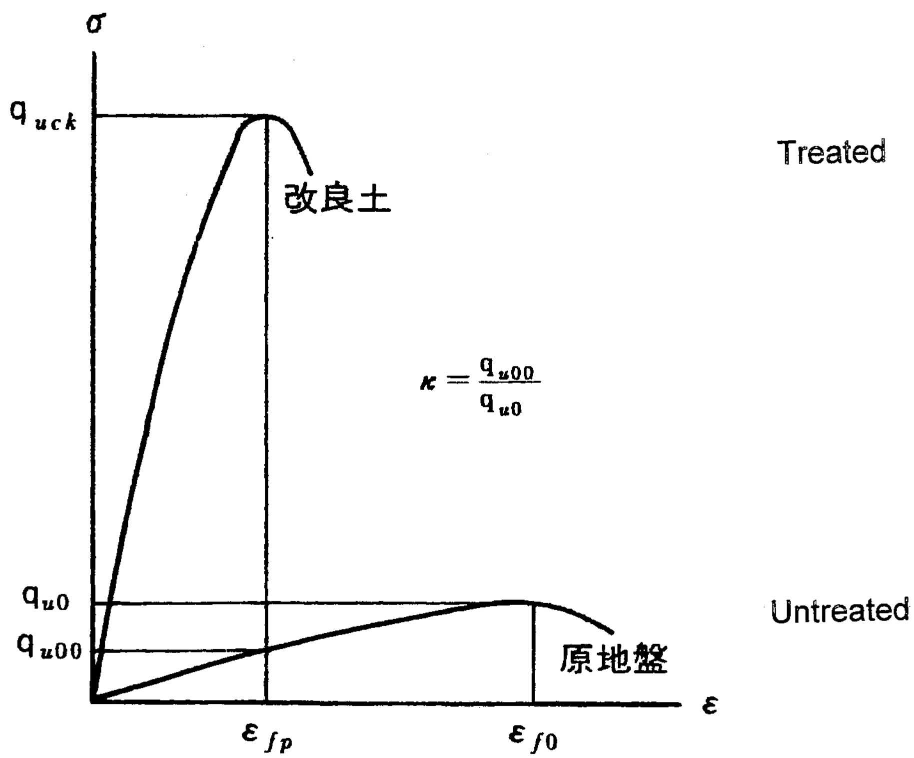
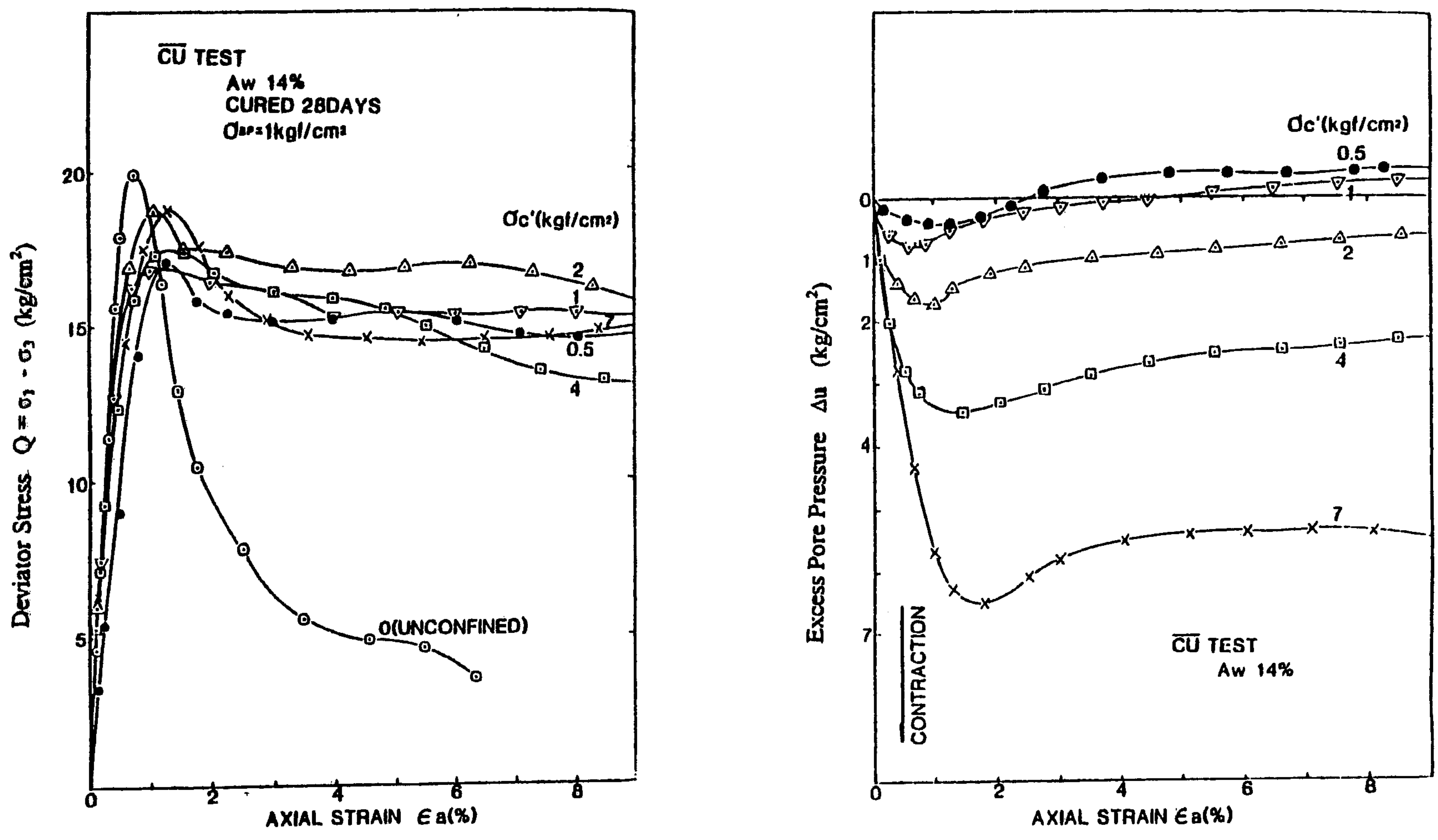


Figure 248. Schematic of stress-strain behavior for treated and untreated soil (CDM, 1994).

A typical stress-strain curve for an unconfined compression test and several for confined-undrained triaxial tests on cement stabilized Tokyo Bay clay (a_w = 14%) are shown in Figure 249. As clearly indicated, once the peak strength is exceeded, a marked and rapid decrease in strength of the unconfined sample occurs to a residual strength of less than ¼ of the peak. However, the data of Figure 249 show that for the CIU triaxial tests, such dramatic loss in strength at strains past the peak strength does not occur, as discussed further in Section 6.4 below.

The initial stress-strain behavior of soil-cement in unconfined compression, as well as in other forms of shear strength tests, is very much stiffer than that of unimproved soil. The data in Figure 249 indicate that the unconfined and CIU triaxial tests have similar initially stiff behavior. From such data, elastic modulus values may be calculated. It is widespread practice to report the elastic modulus of soil-cement as E_{50}, which is the secant Young's modulus at half the sample



(a) Stress Strain Response

(b) Pore Pressure Response

Figure 249. Consolidated undrained triaxial test data (Kobayashi and Tatsuoka, 1982).

peak strength (as referred to in Chapters 4 and 5). However, magnitude of strain and methods of measuring soil-cement deformation under loading have a significant/major influence on modulus values.

The information included in Chapters 4 and 5 shows a wide scatter of data and a wide range in boundary line determination. For example, two sets of data showing E_{50} versus unconfined compressive strength were discussed in Chapter 4: Takenaka (1995) found E_{50} = 350 to 1000 x U.C.S., while O'Rourke et al. (1997) recorded E_{50} = 50 to 150 x U.C.S. (Figure 176). It appears that the difference may simply be explained by the methods used to determine E_{50} modulus. The Takenaka data appear to have been developed using direct "on the specimen" measurements to determine vertical strain, whereas the modulus values of O'Rourke et al. (Figure 176) used dial gauges to measure vertical deformation externally (measuring the top cap deflection). There can be considerable difference in strain measured by the two methods, with the external measurements including "extra" deformations from system compliance, in addition to sample strain. Therefore, the deformation modulus values determined by external measurements can be several times smaller than when strain is measured directly on the soil-cement sample. The influence of strain level on deformation modulus of soil-cement is discussed further in Section 6.4 below.

6.3 Deformation Behavior in Confined Triaxial Shear

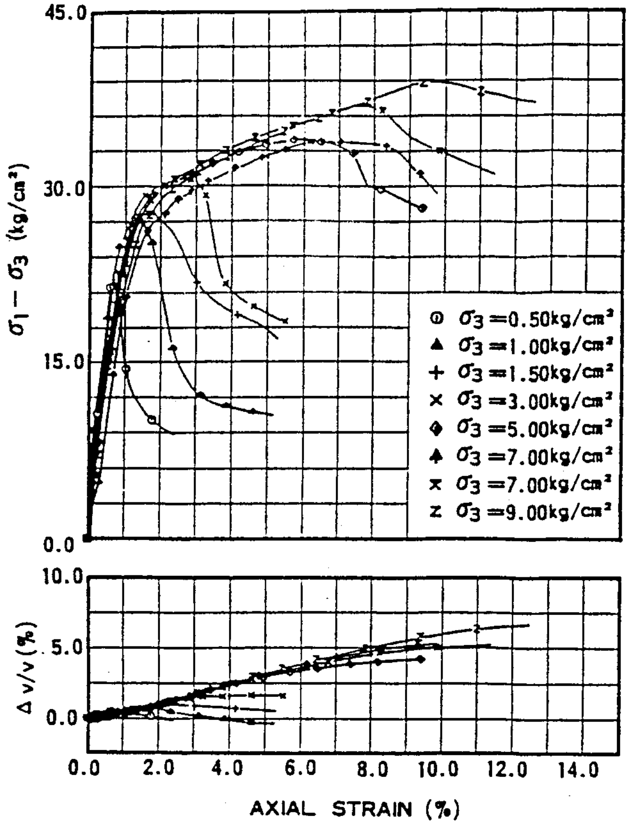
It is appropriate to consider the stress-strain behavior of treated soil under confined triaxial conditions because these are the stress conditions that will exist in nearly every actual installation, whether surrounded by soil or within a large mass of soil-cement. Researchers have found considerable differences in the deformation behavior of soil-cement, depending on whether the tested specimens are drained or undrained. Therefore, the following sections discuss the behaviors of soil-cement for each drainage condition separately, while the final section examines the strain dependency of soil-cement deformation.

6.3.1 Undrained Triaxial Compression

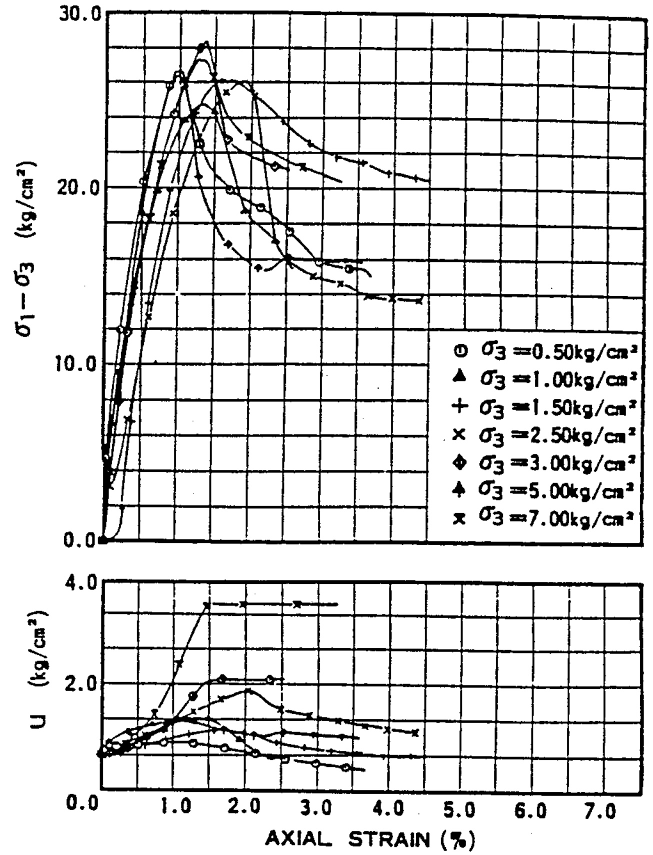
Kobayashi and Tatsuoka (1982) presented examples of stress-strain behavior of cement treated soft clay in undrained triaxial compression (CIU Test) (Figure 249). The specimens were initially consolidated to confining pressures of 0.5 to 7 kg/cm^2, and then loaded axially to failure without permitting drainage. The shearing caused excess positive pore pressures to develop, which for these tests approach the initial effective confining pressures (which were also the initial consolidation pressures) of the sample. Thus, the samples eventually reach conditions of zero effective confinement at or shortly following development of peak strength. Except for the confinement provided by the triaxial cell pressure, the stress-strain behavior of samples loaded in undrained loading is quite nearly the same as unconfined specimens, with one significant exception.

For undrained triaxial compression, there is only minor reduction in deviator stress after the peak strength is reached. For the test data shown in Figure 249a, there is only about a 15% to 20% decrease in vertical deviator stress at strain levels substantially beyond peak. This is in sharp contrast to the nearly total loss of strength for unconfined specimens at strains beyond peak strength. Results of several researchers have similarly demonstrated that the effect of confinement on soil-cement when loaded under CIU triaxial conditions is to cause ductile behavior. Most of the soil-cement strength is actually retained at large strains during undrained shear. This is significant in the modeling of stress-strain behavior of soil-cement. Also important to note from Figure 249b is the moderate decrease in the excess pore pressure at larger, post-peak strength strains, which is related to soil-cement dilatancy as shear strains become large.

Data from undrained triaxial tests on soil-cement specimens with a_w = 15%, as reported by Takenaka (1995), are shown in Figure 250b. These indicate quite similar stress-strain behavior, with peak strengths being of similar magnitudes irrespective of confining pressure. The reduction in deviator stress after peak strength is somewhat greater (up to 35 to 40%) for the lower confining pressures, although the soil-cement still retains its ductility, and does not suffer complete failure as a brittle, uncontained body. It is noteworthy that actual initial in situ



a) Drained Tests



b) Undrained Tests

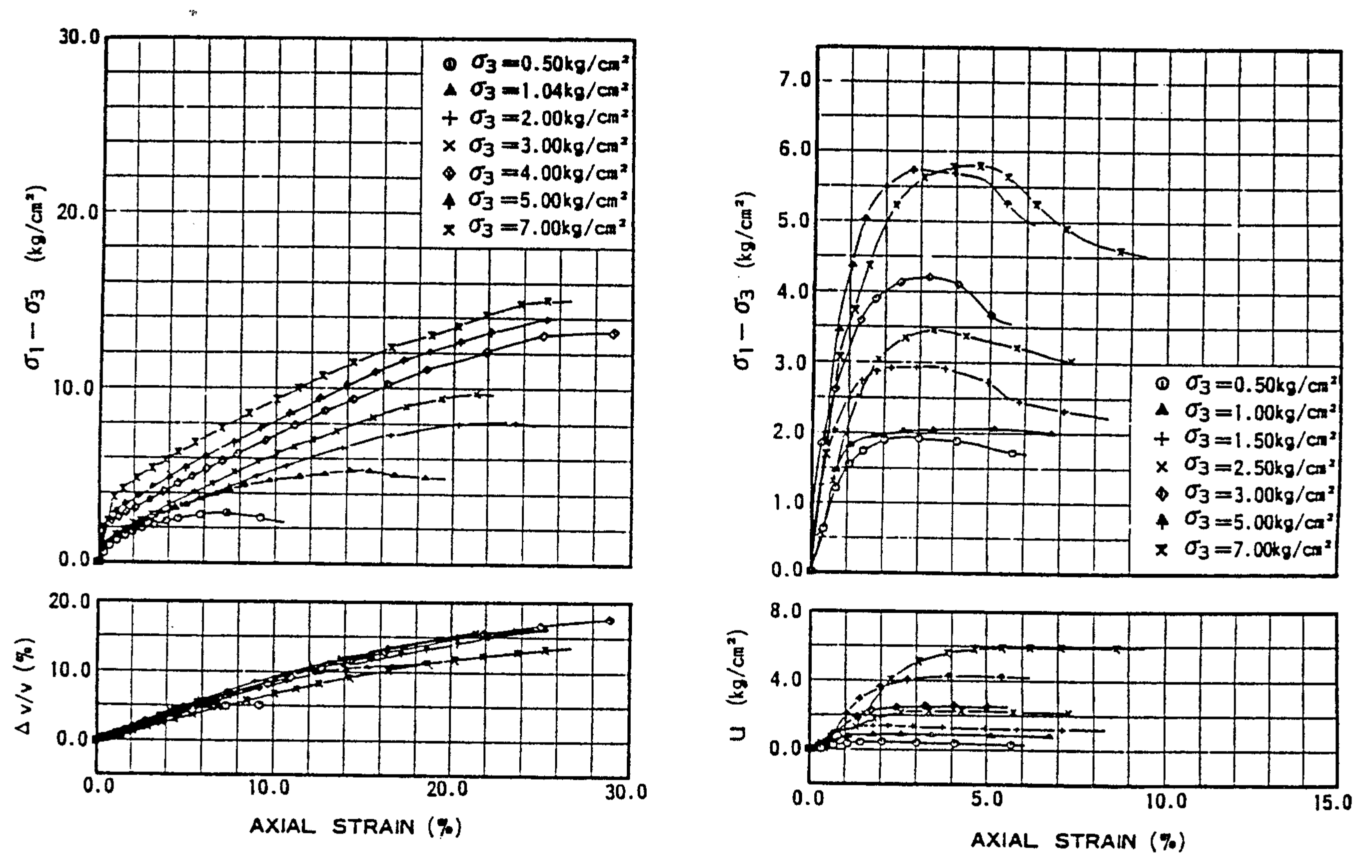
Figure 250. Drained and undrained triaxial test data (a_w = 15%) (Takenaka, 1995).

confining stresses within soil-cement structures would rarely exceed 2.5 kg/cm^2. Also, the excess pore pressures reported in the Takenaka data grow only to about half of the confining pressures, rather than becoming equal as indicated in Figure 249b.

At low cement contents, the effect of the confining pressure on undrained stress-strain behavior is pronounced, as evidenced in Figure 251b. With cement content of 5%, the strength of the soil-cement increases with higher confining stresses, and reaches peak strength at substantially greater strains than the samples tested at 15% cement content. The explanation for this difference in behavior is straightforward. At $a_w = 5\%$, the cementation has not been so completely developed as at $a_w = 15\%$. Peak strength at $a_w = 5\%$ is only 10 to 20% of that at $a_w = 15\%$. But again, even at these lower strengths, the behavior of the soil-cement at strains beyond peak strength is still ductile, and perhaps more ductile than at the higher cement factor. Excess pore pressures reported for the lower cement content approached the confining pressure and remained high at strains beyond the peak strength, rather than decreasing as had occurred in the higher cement content specimens.

Undrained triaxial tests reported by Bergado et al. (1996) for Bangkok clay using $a_w = 5\%$ showed similar effects of increasing strength with increasing confining pressure (Figure 252). Peak soil-cement strength is reached at 2 to 5% strain, with the larger strains corresponding to the higher confining pressures. The higher confining pressures act to decrease the stiffness of low a_w soil-cement. It is interesting to compare undrained triaxial compression behavior of soil-cement to that of untreated clay. The relatively light treatment with 5% cement to stabilize the clay causes improvement in strength of from about 40% at higher confining pressures up to more than 150% for the lowest confining pressures.

In comparing cement stabilization effect on different soils, it is seen that at the higher cement factors, which are more likely to be applied in creating soil-cement structures, the two different clays stabilized with cement exhibit quite similar trends and magnitudes. For example, compare Bergado's tests using soft Bangkok clay at $a_w = 15\%$ in Figures 253 and 254 with the data reported by Takenaka in Figure 250b. In both, peak strength is reached at similar strains, and excess pore pressures are generated that approach the confining pressure. At strains beyond the



(a) Drained test

(b) Undrained test

Figure 251. Drained and undrained triaxial test data ($a_w = 5\%$) (Takenaka, 1995).

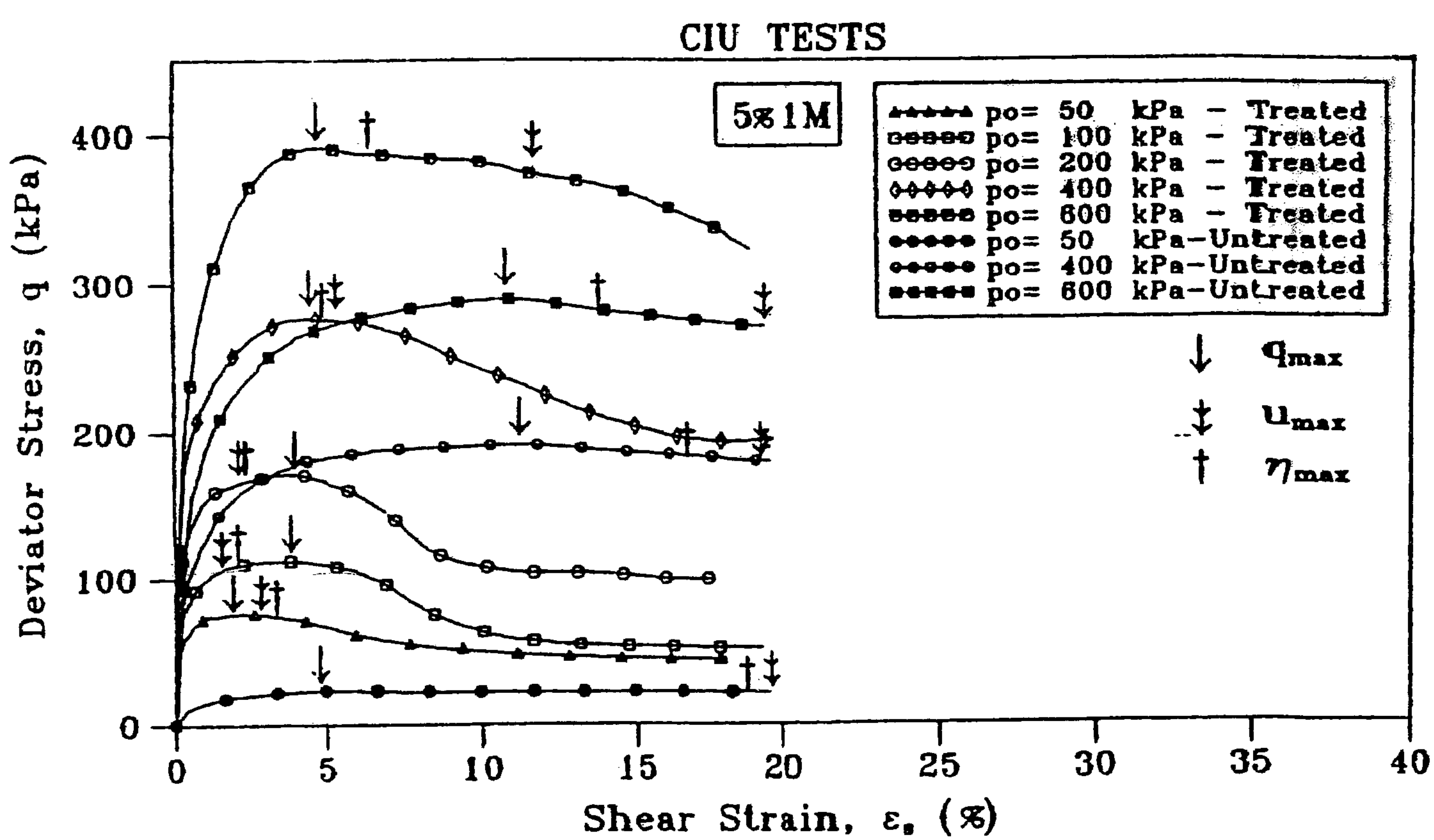


Figure 252. Consolidated undrained stress-strain behavior for 5% cement content (Bergado et al., 1996).

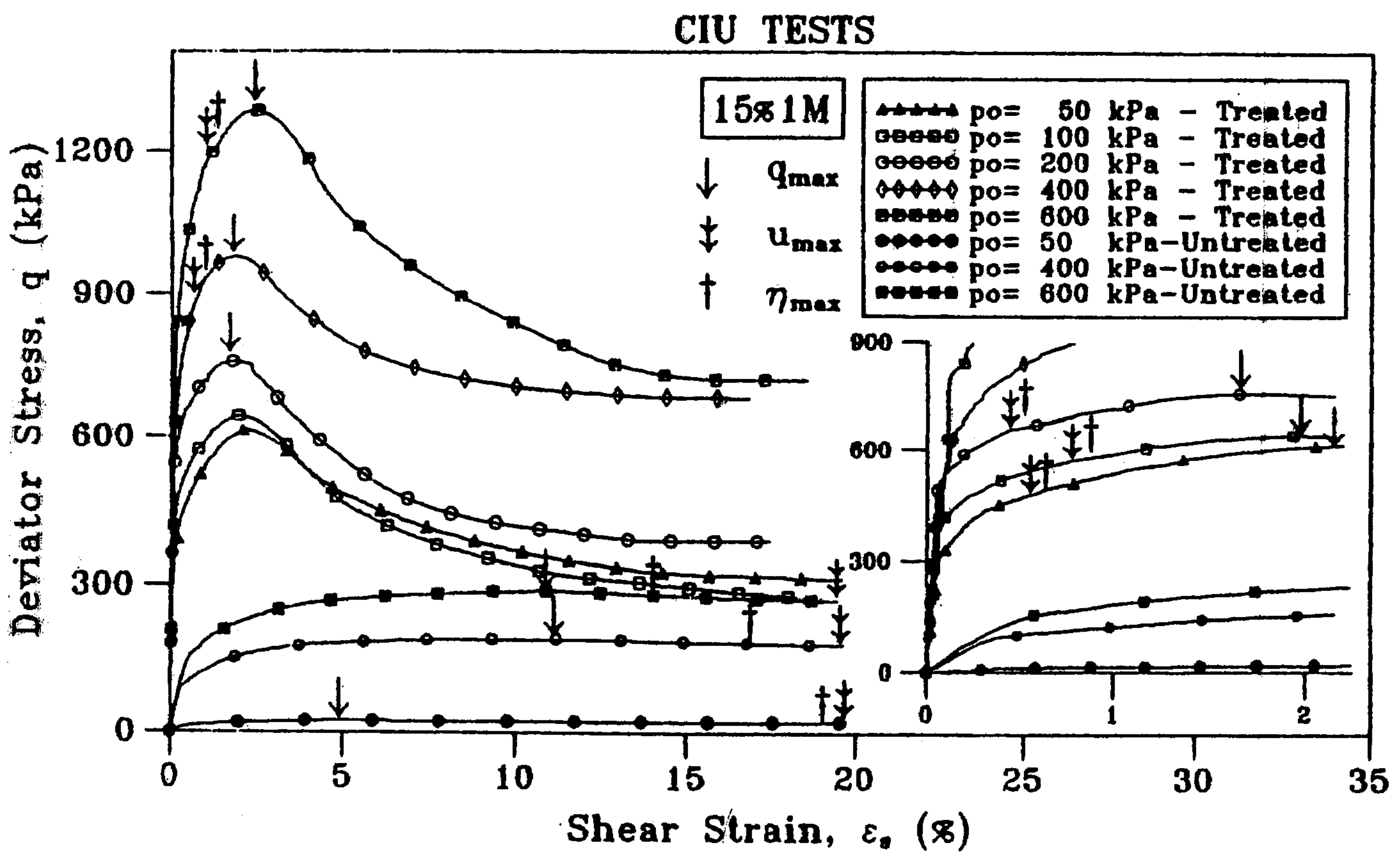


Figure 253. Consolidated undrained stress-strain behavior for 15% cement content (Bergado et al., 1996).

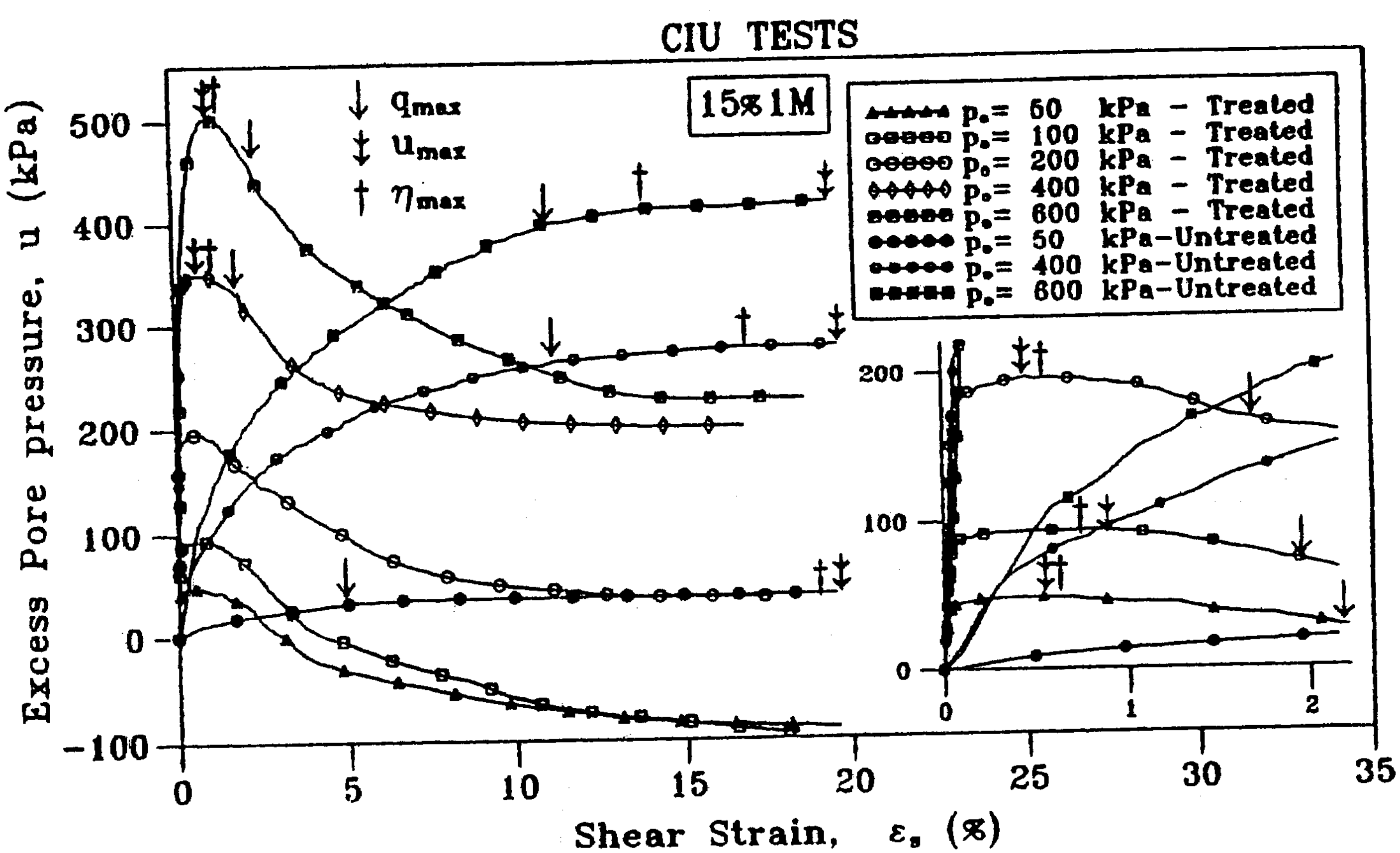


Figure 254. Excess pore water pressure generation during consolidated undrained tests for 15% cement content (Bergado et al., 1996).

peak strength, excess pore pressure decreases as does deviator stress as the cementation bonds are broken and the soil-cement reverts to a frictional behavior.

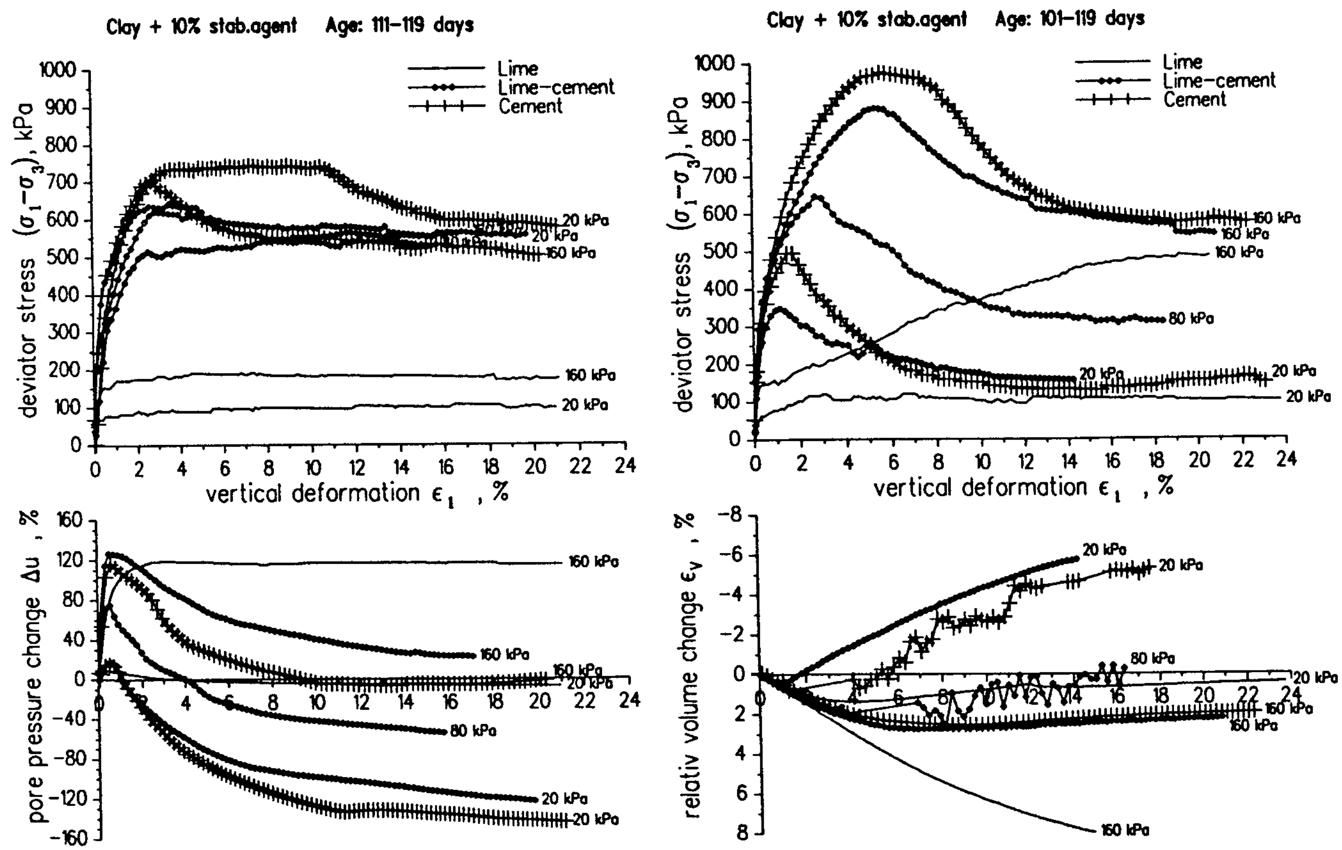
Åhnberg (1996) presented results from undrained (Figure 255a) and drained triaxial tests (Figure 255b) on clayey silt, clay, and clay gyttja soils treated with different binders: lime, lime-cement, and cement. Samples were tested at confining stresses of 20, 80, and 160 kPa. These undrained results also show that stress-strain behavior is dependent upon strength and confining pressure, with most often an initial increase in pore pressure followed by a decrease prior to failure. This behavior indicates a tendency for dilation at failure, that which is more pronounced at low confining stresses.

Undrained failures occurred at relatively small deformations of 1 to 2% for the stabilized silty clay and at various strains between 1 and 17% for the clay and clayey gyttja.

6.3.2 Drained Triaxial Compression

Soil-cement behavior during drained triaxial compression tests is considerably different than in undrained tests. Data from Takenaka for $a_w = 5\%$ and 15% are shown in Figures 251a and 250a, respectively. The difference in behavior is largely attributed to the ability of the drained tests to maintain constant effective confining stress. Therefore, the soil-cement continually experiences the consolidation confining pressures throughout the application of shear stress up to peak strength, rather than reverting to an unconfined condition as in the CIU test.

At the lower cement content illustrated in Figure 251a, the peak strength of soil-cement increases with greater effective confining pressures. These soil-cement specimens behave very much like samples of cohesionless soil. For $a_w = 5\%$, there is not a large cementation effect. Therefore, the application of the initial triaxial consolidation stresses cause breakdown of cementation bonds that formed in the secondary cementation and pozzolanic strengthening phase. The behavior of the lightly cemented clay illustrated in Figure 251a is similar to that of very loose sand or soft clay, with strain to peak strength exceeding 20% at the higher confining pressures. Treated sand at $a_w =$ 5.5% shows similar behavior, as shown in Figure 256.



a) undrained triaxial tests

b) drained triaxial tests

Figure 255. Examples of results from triaxial tests on soils stabilized with different binders (Åhnberg, 1996).

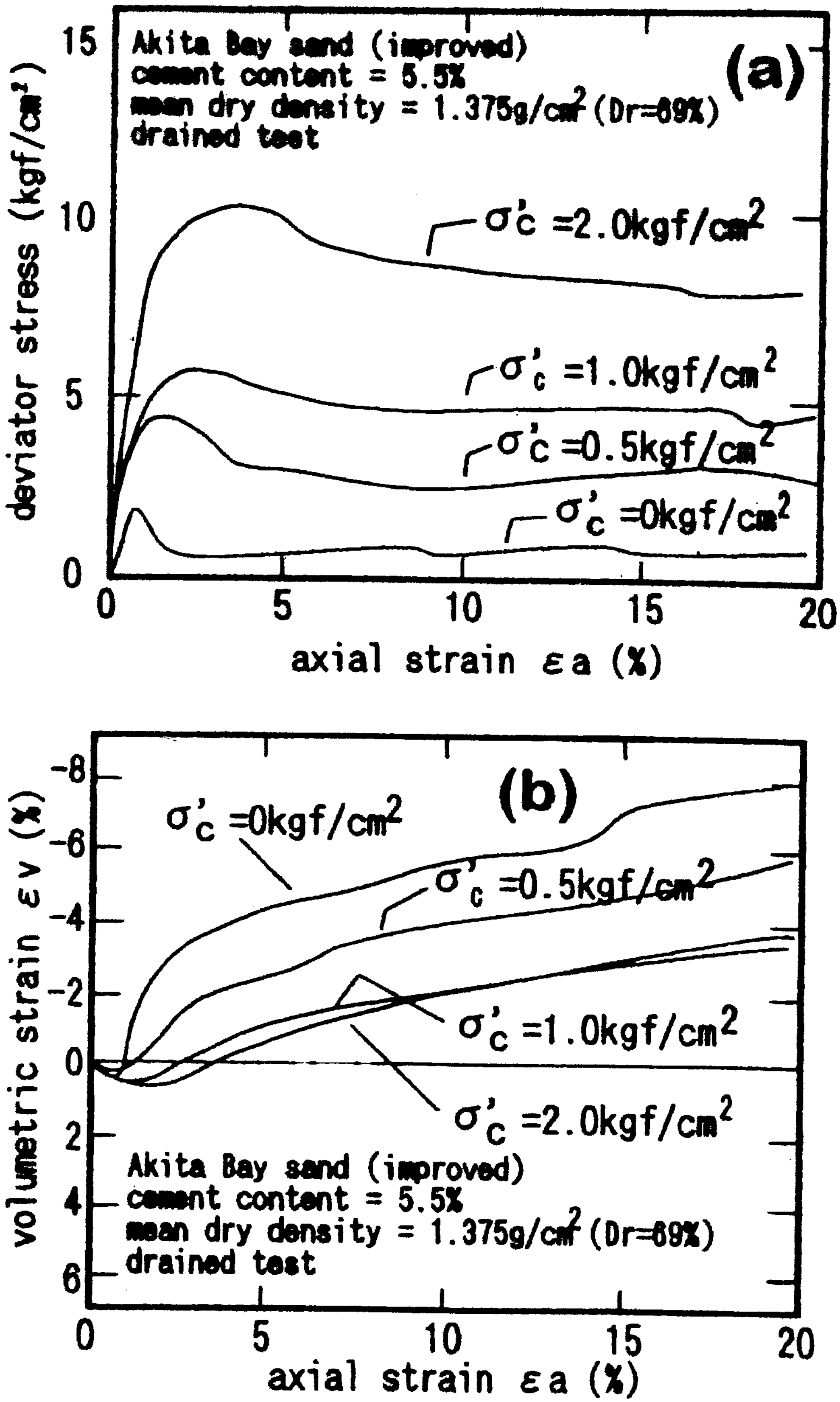


Figure 256. Consolidated drained triaxial test results (Zen et al., 1990).

At the 15% cement content, two different stress-strain behavior patterns are observed: one for lower confining stresses and the other at the high confining stresses. The data shown in Figure 250a show substantial peak strengths at small axial strains, less than about 2%, and then decrease in deviator stress for the four tests at lower confining stress. But at the higher confining pressures, strength continues to slowly increase from about 2% until 7 to 10% axial strain. As with the specimens loaded at low confining stress, the stress-strain behavior at higher confining pressures is initially very stiff when strain is less than 1 to 2%. But after the initial stiff response, which brings the strengths up to about 75% of the peak strength, considerably more straining occurs as the specimens with high confining pressures gradually experience yield through an additional 5 to 7% strain before reaching peak strength. For the samples under high confinement in drained tests at $a_w = 15\%$, the decrease in deviator stress during the residual strength-large strain phase is similar to that of the undrained samples, with residual strength being about 10 to 15% below peak strength. With the lower confining pressure (0.5 to 3.0 kg/cm^2), a 30 to 50% strength decrease was observed in the reported data.

Volumetric strains throughout the strain development for drained triaxial shear are either contractive or dilative, depending on the degree of cementation and the confining pressure. The samples showing brittle behavior, i.e., those having higher cement content, when sheared at the lower confining pressures are dilative, which is evidence of effective cementation. On the other hand, most of the tests showed contractive behavior. The contractive results are compatible with the development of positive excess pore pressures for undrained tests on soil-cement.

The effect of differing cement content on the drained stress-strain behavior of cement treated Bangkok clay under small confining pressure is shown in Figure 257. These data dramatically illustrate the effect of cementation with the soft clay soil sample treated with a cement content of 15% having five times the peak strength of a sample treated with only 5% cement content. However, as the plotted data indicate, all of the residual strengths approach a very well defined, small range.

For cement treated sand, similar behavior is observed in stress-strain, but the volumetric strain response is quite different. Drained triaxial test data shown in Figure 256 (Zen et al., 1990), in

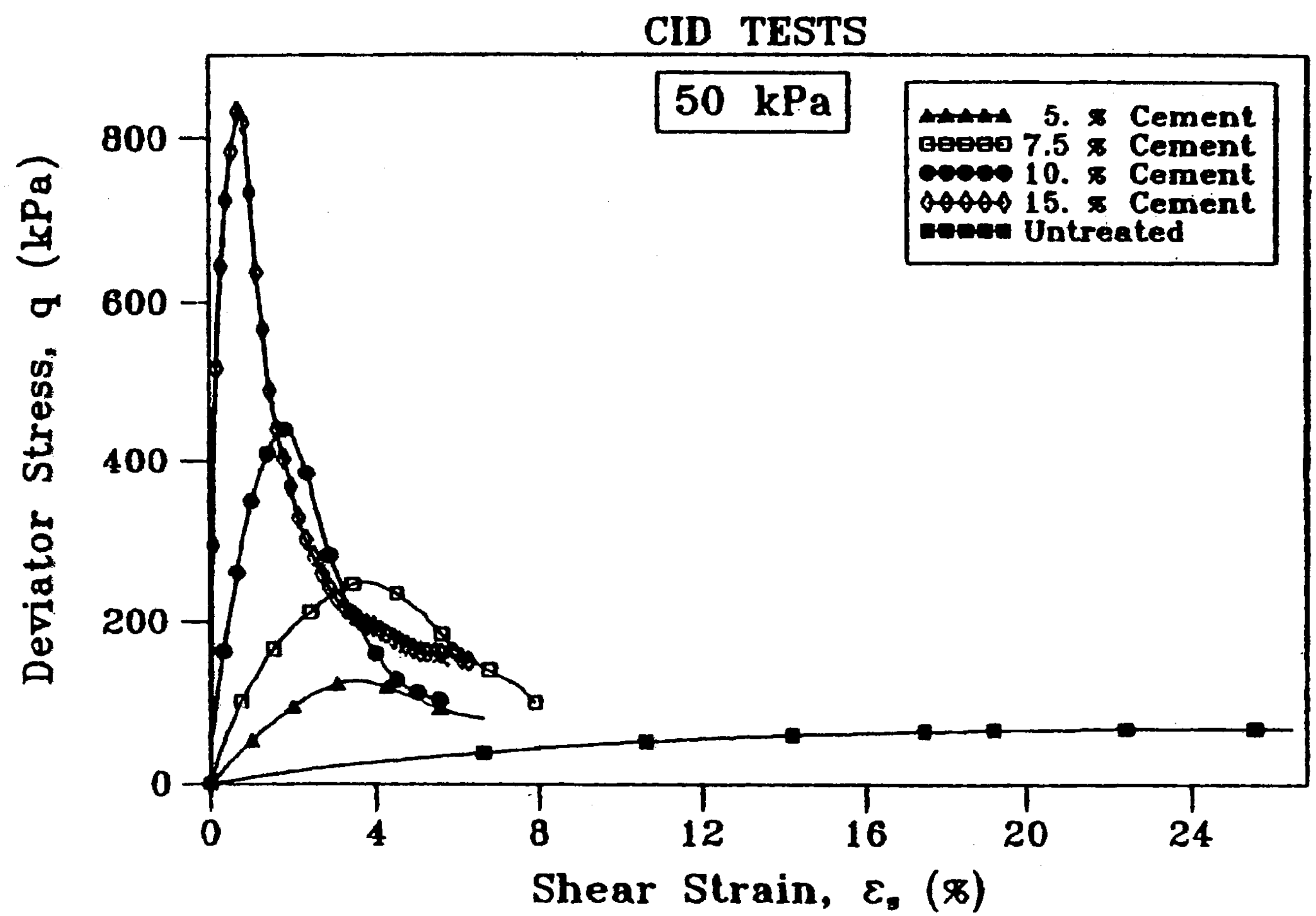


Figure 257. Consolidated drained triaxial test results for varying cement contents (Bergado et al, 1996).

which a sand was treated with 5.5% cement, show stiff response to loading with peak strength achieved at 1 to 5% strain, the larger strains being at the higher effective confining pressures. Residual strengths decline by 15 to 50% from peak. This behavior is quite similar to that for clay soils given low cement content treatment, (e.g., Figure 251a). However, the volumetric strain for the treated sand shows strong dilative behavior as strains move beyond peak strength. This difference in volumetric strain behavior can be quite significant to strength behavior if the sand-cement undergoing shear loading cannot imbibe water quickly enough to permit dilation, In such condition, the sand-cement would behave as though undrained, with negative pore pressure momentarily arising, giving greater strength for a very short amount of time. This can be significant for transient, seismic loading conditions.

There is definite trend in the drained stress-deformation behavior in that it is controlled by the level of initial confining pressure in relation to the isotropic yield stress. When the confining pressure is less than the yield stress, the effect of cementation bonds is retained, and the very stiff behavior with only 1 to 2% strain to peak strength will materialize. Also, the drop-off in deviator stress will be pronounced as the cementation bonds break down and the soil-cement reverts to frictional behavior.

When the confining pressure exceeds the isotropic yield stress, the cementation bonds are considered to have been largely disrupted by the confining pressure, and subsequent stress-strain behavior will be dominated by the frictional behavior of the soil-cement. Some residual benefit of the cementation will remain when higher cement contents are used. Jameson (1996) reports that the isotropic yield stress is 0.8 to 0.9 times the unconfined compressive strength for clay mixed with 8% cement. However, Kohata et al. (1996) report the ratio to be more in the range of 1.2 to 1.7, while also reporting that there is an effect of confining pressure during initial soil-cement curing impacting on the yield strength. As would be expected, the specimens tested by Kobayashi and Tatsuoka (1982) proved to have increased strength when first subjected to confining pressure and drainage during curing.

Results from drained triaxial tests reported by Åhnberg (1996) are shown in Figure 255b. As the pore pressure generation implied during undrained testing (Figure 255a), the results from drained tests showed that the samples behaved in a dilative manner in almost every test, with more dilatant behavior occurring at the lower confining stresses. Failure occurred at very small deformations (0.5 to 1.5%) for the treated clay silt, while the strain at failure for the other materials varied from 1 to 19%. Deformation at failure for the latter materials increased with increased confining stress.

Effective stress friction angles and cohesion intercepts were evaluated from stress path data collected during drained testing. Values of friction angles at failure (peak strength), Φ_f, and at large strain (residual strength), Φ_{ls}, were of the order of 34 to 44° (Figure 258). Åhnberg states that Φ_f and Φ_{ls} values "did not differ in any significant way." The cohesion intercepts were found to increase with increasing material strength, and corresponded to 25 to 48% of the U.C.S. (Figure 259).

6.3.3 Summary of Stress-Strain Behavior Under Confined Compression

Three factors have significant influence on the stress-strain behavior of treated soil subjected to confined compression loading: the soil type, cement content, and the confining pressure. Equally important however is the drainage condition under which the soil-cement is sheared. At low cement contents, initial stiffness offered by the cementation can be overpowered by the confining pressure, which may be large enough to break down cementation bonds, thus rendering an otherwise stiff, cemented clay soil material to behave more like a loose sand. Sand soils and clay soils behave differently. Cement treated sand will retain a tendency to dilate at strains past peak strength, likely due to higher initial density, unlike cement treated clay.

Drainage characteristics during shear strongly impact soil-cement stress-strain behavior. In undrained conditions, pore pressures build up to equal the confining pressure such that by the point of peak strength, the effective confining pressure is nearly zero. At substantial cement contents, the peak strengths measured in undrained tests are nearly the same magnitude, regardless of initial confining pressure. However, when the cement content is low, it appears that the initial application of confining pressure breaks down cementation, and the soil-cement then reverts to friction behavior wherein the strength is dependent on the level of confining pressure.

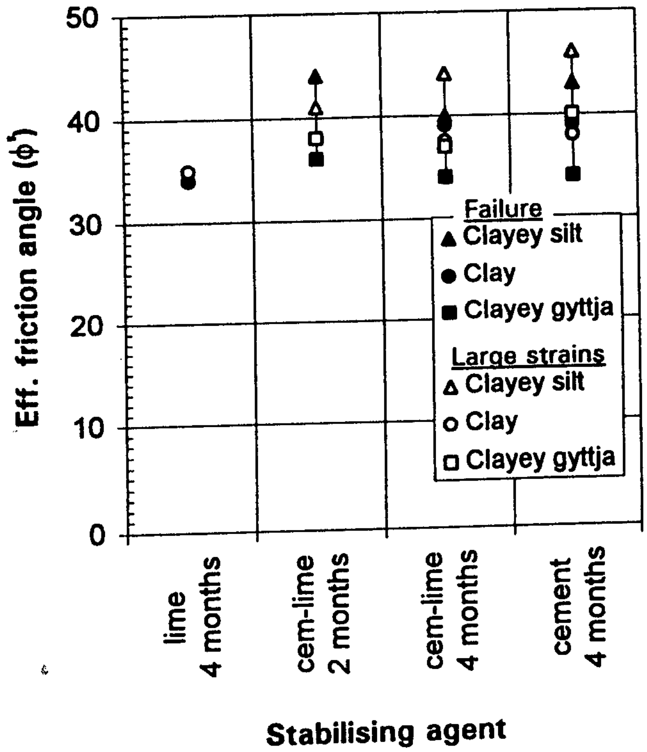


Figure 258. Friction angles evaluated from drained triaxial tests on lime, cement-lime, and cement stabilized soils (Åhnberg, 1996).

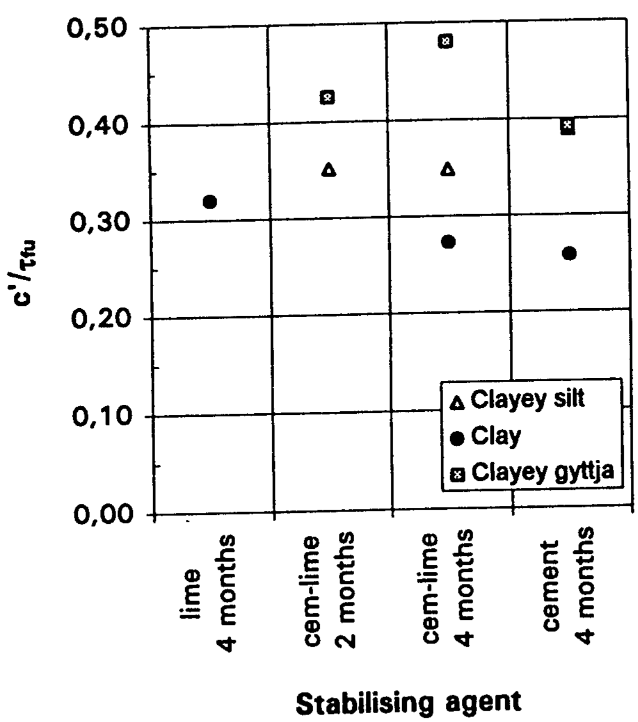


Figure 259. Ratio between cohesion and undrained shear strength evaluated from drained triaxial tests on lime, cement-lime, and cement stabilized soils (Åhnberg, 1996).

The question remains as to which drainage condition will occur in the actual constructed facility. Slow, steadily increasing loads can likely be expected to permit at least partial drainage. Where loads are sustained, drainage can be expected to develop with time, and with such drainage, some reduction in strength of the soil-cement could well occur based on the above discussed trends in laboratory test results.

6.4 Deformation Modulus of Treated Soil

The stress-strain behavior of treated soil depends on several factors, including the type of soil, the amount of cement added, the degree of mixing, drainage conditions, and the stress level to which the soil-cement is loaded. The first three factors also determine the strength of the treated soil. But several other auxiliary "factors" are also involved in determining the "modulus." For instance, the modulus is stress-level dependent so it varies with the degree to which it is loaded. Also because soil-cement is many times stiffer than soils commonly tested in the geotechnical laboratory, the accurate measurement of strain is vital to determining the "real" modulus, one that will truly reflect the constructed conditions. This section reviews first the different regions of strain through which the soil-cement deforms in reaching yield, peak, and residual strengths, and then presents the findings of several researchers on the modulus at different magnitudes of strain.

6.4.1 <u>Regions of Stress-Strain Behavior</u>

As indicated in Section 6.3, the stress-strain behavior of soil-cement is highly dependent on the level of initial confining pressure. For soil-cement to be most effective and derive substantial benefit of the cementation effects, the confining pressure should be well below the isotropic yield stress. Then, under application of shear stresses the soil-cement will respond, as one would expect, as though it were a stiff cemented soil or weak rock. Under such stress levels, the behavior of the soil-cement is dependent on the cementation, and is little affected by confining pressures. If, however, the confining pressures exceed the isotropic yield stress, then the cementation bonds will be disrupted either by the isotropic stress or early during the build up of shear stresses. In this case, the soil-cement behavior will be that of a largely frictional soil, which would be highly dependent on confining stresses.

For the first situation, wherein the initial confining stresses are well below what Jameson (1996) refers to in his thesis as the "cementation yield envelope" of confining pressure, three regions of stress-strain behavior can be conceptualized as shown in Figure 260. Jameson's work included an extensive examination of triaxial test data reported by Tatsuoka and Kobayashi (1983) on soil-cement specimens made of Tokyo Bay clays for research performed at the University of Tokyo. The regions of stress-strain behavior defined as functions of unconfined compressive strength (U.C.S.) in Figure 260 are from the earlier testing. The space shown in Figure 260 defines bounds within which would lie the stress-strain curves for different specimens.

Essentially, elastic straining occurs up to the yield stress. At stresses below the yield level, the cementation of the soil-cement is largely responsible for stiffness, and bonds are strong enough to maintain elastic behavior, even under several cycles of load and unload. The data analyzed by Jameson indicated that yield, or the start of cementation bonding breakdown, would begin when applied axial deviator stress reaches 0.6 to 0.9 of the U.C.S. While reaching the yield stress, the soil-cement samples of Tokyo Bay clay were found to deform axially with an average modulus of 160 times U.C.S. (the 160 is a mean and is within ±45).

Beyond yield, as the cementation bonds begin to break down, the soil-cement loses stiffness. The idealized stress-strain behavior and the stress-strain curves discussed in Section 6.3 clearly illustrate that strain would increase at a quicker rate. In reaching peak strength, the behavior shown in Figure 260 indicates that an additional 30 to 45% vertical load is required. But in proceeding from yield to peak, the modulus becomes greatly reduced, being less than half of the initial elastic modulus, as can be visualized from the representation in Figure 260.

Finally, as straining continues beyond peak strength, the soil-cement loses strength down to some residual condition. Jameson found that the residual strength would be in about the same range as the yield stress level.

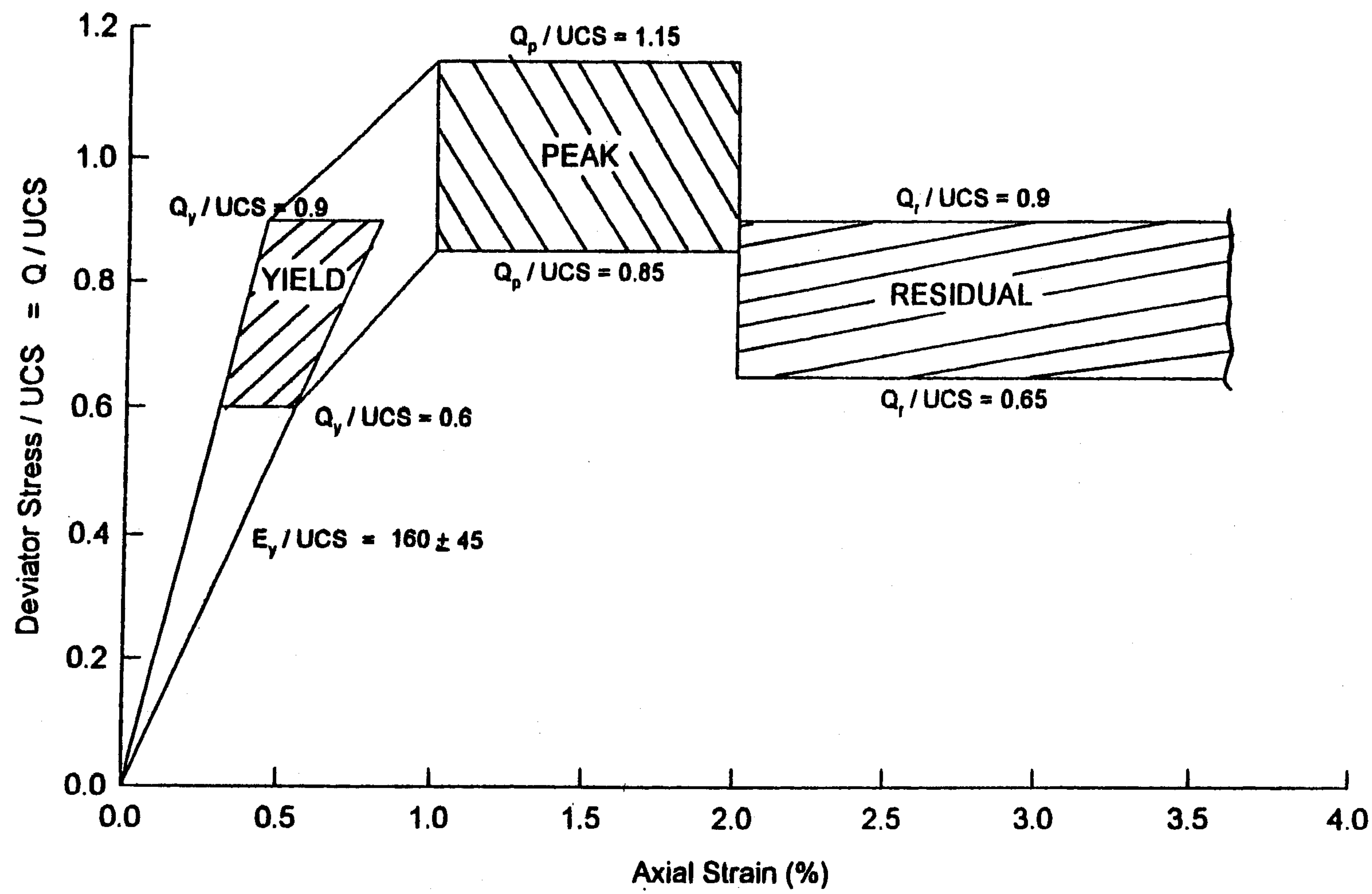


Figure 260. Summary plot of normalized undrained soil-cement stress-strain response for shear from isotropic stress within cementation yield envelope (Jameson, 1996).

Strain levels for each of the three regions are as shown in Figure 260, with yield stress occurring at up to about 0.5% axial strain. Peak strength occurs at strains of 1 to 2%, and residual strength occurs beyond 2% axial strain. Clearly, the modulus and strain level for different loading conditions need to be established in recognition of the strain levels that can be acceptably tolerated in the actual soil-cement structure under field conditions. In certain situations, large strains may be tolerable, particularly when soil-cement is expected to work integrally with the existing ground. In such situations, design would need to use residual strength as the limiting condition, while recognizing that the soil-cement must first exceed peak strength, and a drop-off in strength would occur in going to the residual condition.

In other situations, perhaps where the soil-cement is installed as a separate structure to restrain soil lateral loads, it may be necessary to minimize lateral movement around an open excavation. In such a situation, strain levels may not be allowed to even approach yield conditions, and completely different stress-strain properties would be used in design.

However, in all design and analysis studies into treated soil, it is important to note that other researchers and engineers have reported somewhat different ratios of stress level to U.C.S. for defining boundaries of yield, peak, and residual stress-strain behavior. Thus, for major projects where deformation of the soil-cement body is an important design consideration, it is imperative to undertake a separate, well-planned, laboratory study of the stress-strain behavior of the particular soil as stabilized with cement.

6.4.2 Modulus of Deformation as a Function of Strain Level

There are many ways to calculate the modulus of soils, as illustrated in Figure 261. The most common measure of soil-cement modulus is the secant modulus to 50% of the peak strength, E_{50}, as shown in Figure 261b. This secant modulus averages soil-cement behavior over the full strain range for loads up to half of the peak strength.

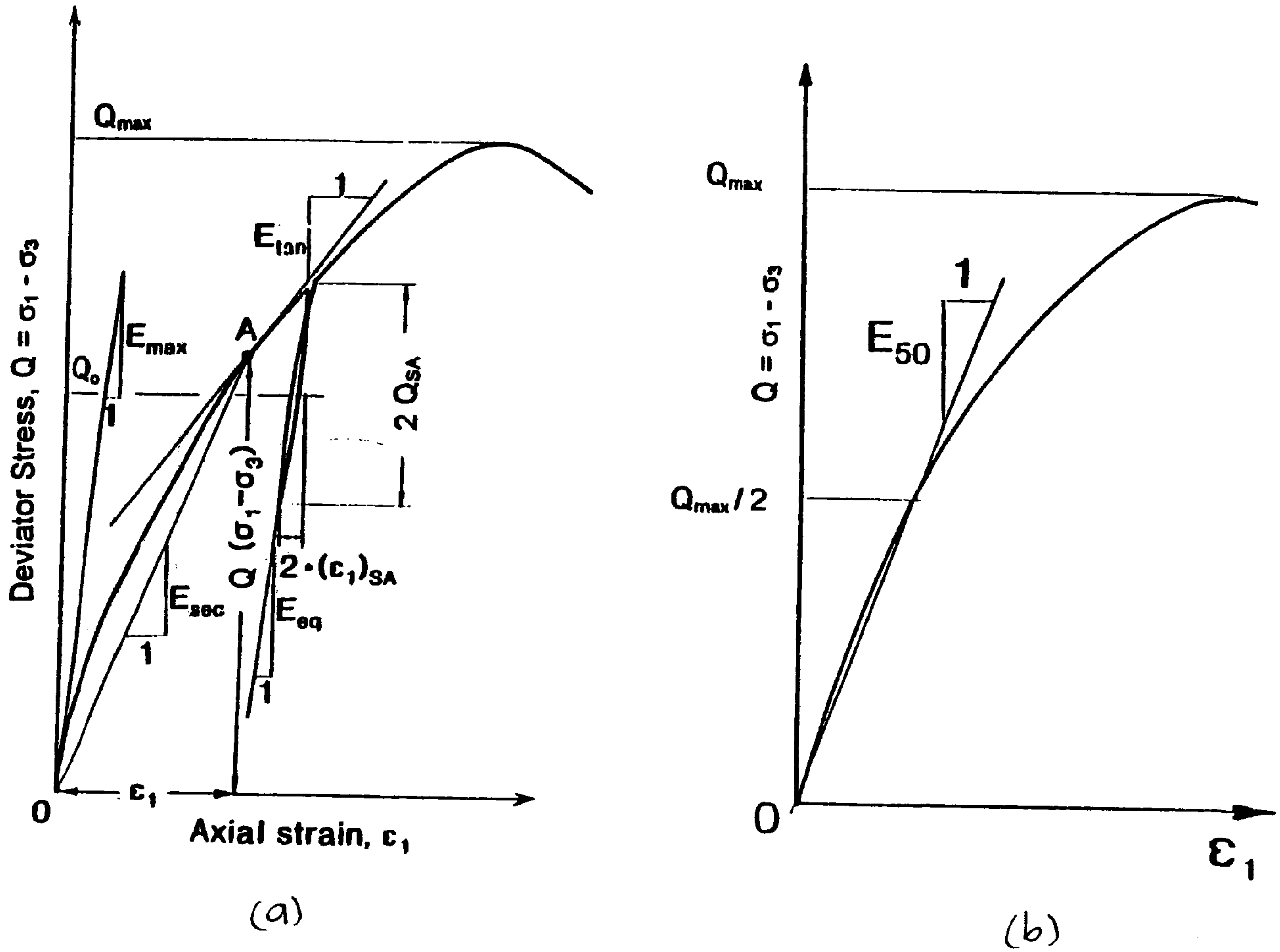


Figure 261. Definitions of Young's moduli (Tatsuoka and Shibuya, 1992).

However, the in situ soil-cement may be stressed to only 20 or 35% of the peak. Therefore, the secant modulus E_{50} may be unrealistically small, and its use would overpredict deformations. As indicated in Figure 261a, the modulus appears very high at low stresses (or small strains), and thereafter, the modulus decreases as strain increases. For some situations, it may be more appropriate to utilize the initial modulus. This would represent the stiffest behavior of the soil-cement since it is the tangent modulus to the initial portion of the stress-strain curve.

As the stress increases, the stress-strain curve becomes less steep, so the incremental modulus for any portion of the stress-strain curve reduces as stresses approach the peak value. The tangent modulus represents the stress-strain relation at a particular point, and will continually decrease to zero as stress levels go to peak strength. Also of importance in certain situations is the reload modulus, which is used when stress has been relieved, and then is reapplied such as in earthquake loading or other transient load such as tidal activity.

The determination of modulus depends on an accurate measurement of stress and strain. For soil-cement testing, the initial strains are very small. Shibuya et al. (1989) have shown that the usual method of measuring vertical deformation in triaxial tests overestimates the strain, and therefore will underpredict the modulus. The bedding of the end platens on the stiff soil-cement and its local crushing as the platens set in firm contact are considered the major source of "excess" vertical deformations. To avoid these factors, LDT instruments have been attached directly to the sides of triaxial test specimens to make "local" measurements of specimen deformations. Figures 262 and 263 illustrate the difference in stress-strain curves determined in CU triaxial tests on soil-cement made from both the sand (i.e., the slurry fill) for the Kawasaki man-made island and the natural clay.

A comparison of the magnitudes of modulus from both externally and locally measured strain by Shibuya is shown in Figure 264 for both undrained and drained triaxial tests. There appears to be an influence of confining pressure, wherein the externally measured modulus increases with confining pressure, but the modulus determined by locally measured strains remains fairly

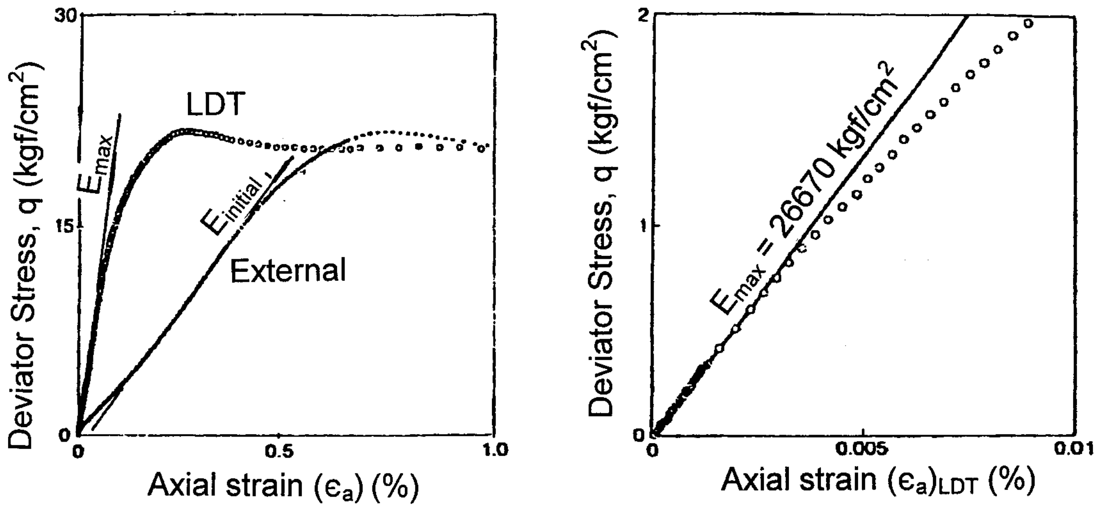


Figure 262. Consolidated undrained triaxial test result from slurry fill at Kawasaki man-made island (Shibuya et al., 1989).

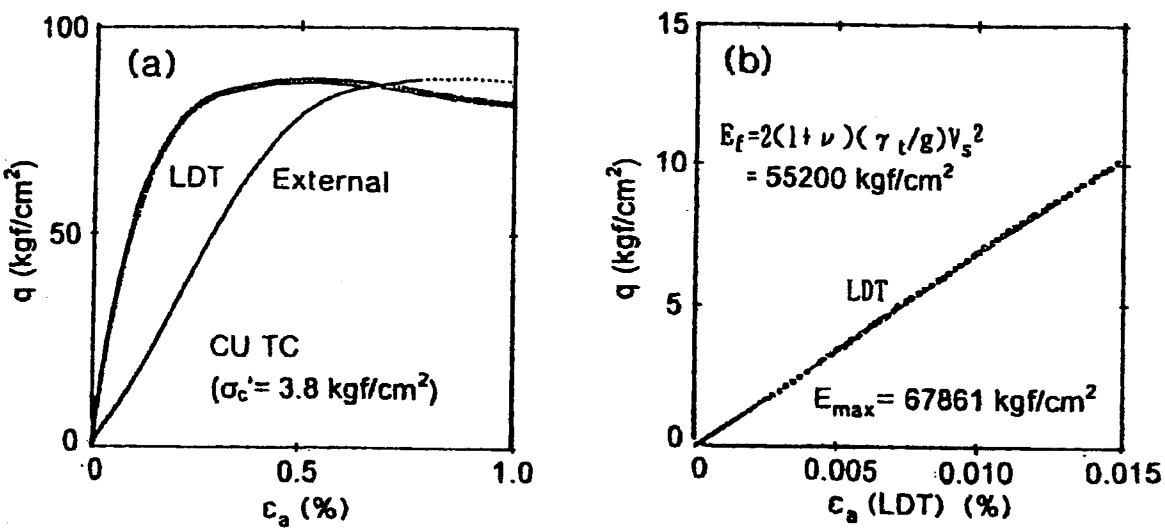


Figure 263. Consolidated undrained triaxial test results of treated clay at the Kawasaki man-made island (Shibuya et al., 1989).

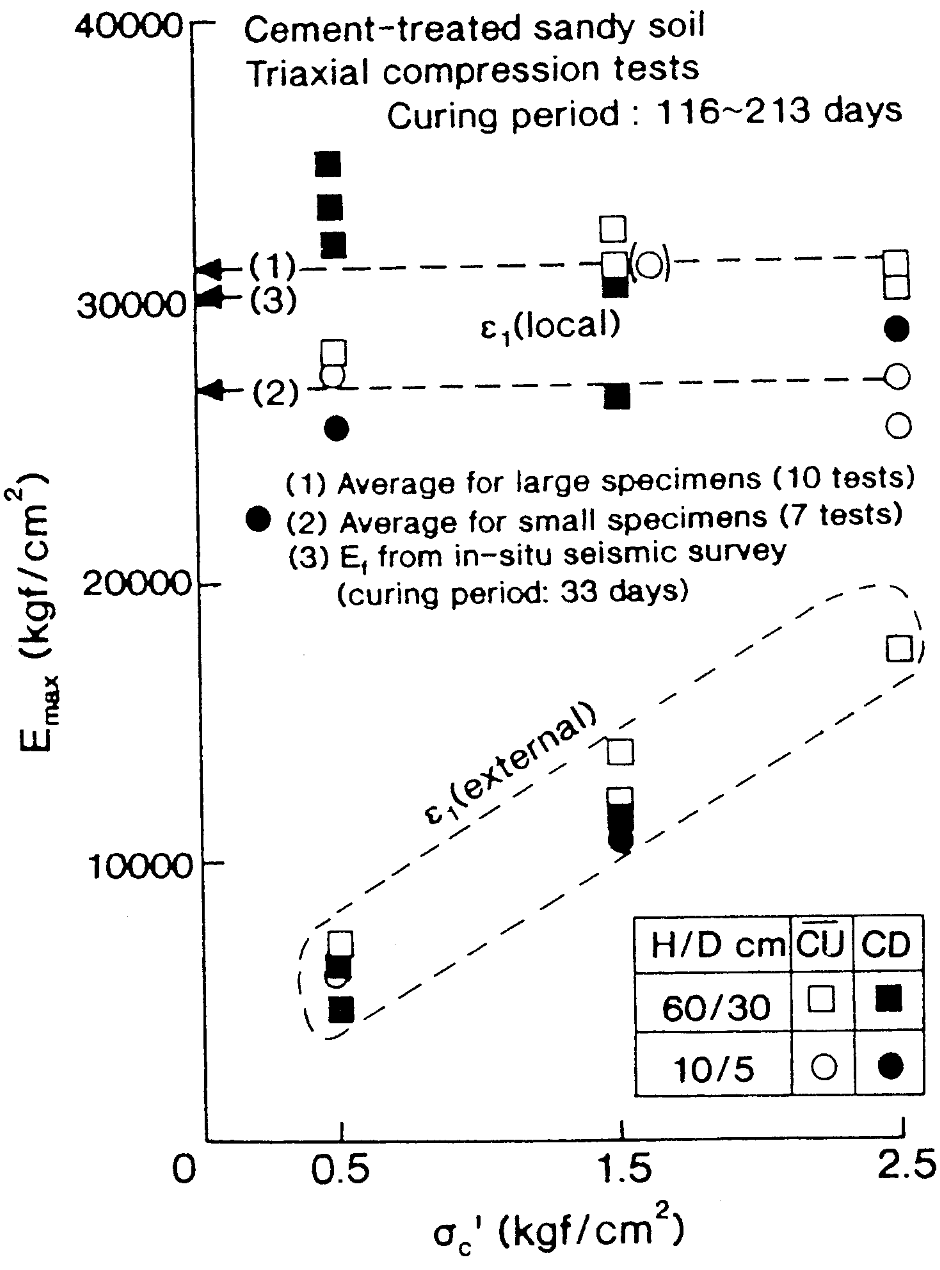


Figure 264. Comparison of E_{max} and $E_{initial}$ of cement-treated sandy soil (Shibuya et al., 1989).

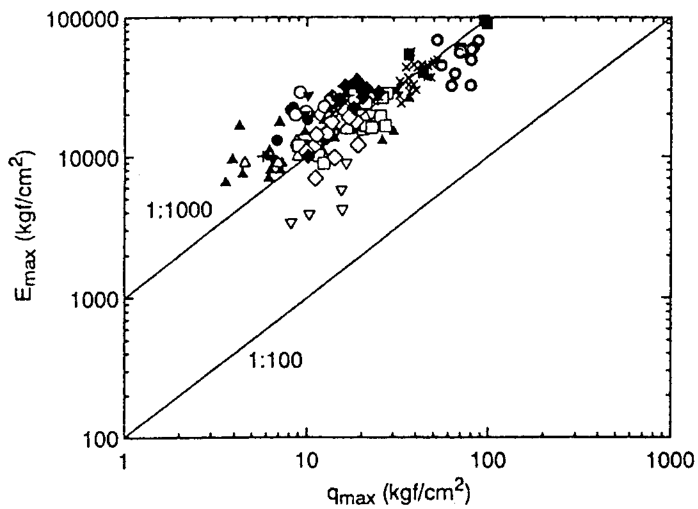
constant. There appears to be a 4 to 10 times stiffer modulus from locally measured strains versus the traditional external measurement.

The modulus of soil-cement has been found by many to be directly related to the unconfined compressive strength. An example is in data compiled by Tatsuoka et al. (1996) that are shown in Figure 265. Most of the data points are for cement treated sand and cinders, but the two groups of points indicated as DMM are for mixed clays. These data show the relationship of Q_{max} to E_{max} (determined with local strain measuring devices) to average 1:1200 for the sand and cinders, but to be in the range from 1:1200 to 1:500 for the clay soils. Data from Takenaka (1995) in Figure 108 for a variety of clay and silt soils show slightly lower ratios of 1:1000 to 1:350 for E_{max} to Q_{max}.

Measuring the modulus of soil-cement depends on the expected strain levels. The method of measuring strain should subject the specimen being tested to the same strain levels as expected in situ. If the soil-cement is primarily intended as seismic mitigation, then strains may be quite small, and field seismic wave tests may be appropriate. But if large strains are anticipated, such as in the case of load sharing between soil-cement and existing soil, then deformations may be quite large, and triaxial tests may be most appropriate.

Figure 266 summarizes data on shear modulus versus shear strain level for tests made on treated sandy soil. Both monotonic and cyclic loading triaxial test data are shown, with strain measured by local measurement of strain and by external measurement. At small strains of less than 0.001 (or 0.1%), the shear modulus values determined by locally measured strains are nearly twice those determined by external measurement. Interestingly, the shear moduli determined in triaxial tests with locally measured strain are about the same as those determined by seismic wave velocity testing, at yet smaller strain levels.

Of significance in Figure 266 is the degradation of modulus beyond strains of 0.0001 (or 0.01%). The reduction of modulus is expected given the shape of stress-strain curves (Figure 261, for example). At increasing strains, and up to about the yield strength of the soil-cement (judged to be 0.002 or 0.2%) the strains determined by local measurement still provide



	Site and Material	Testing method
■	Hawaii, Mauna Kea, Cement-mixed cinder soil	CD TC tests
▲	Prepared in the Lab., Cement-mixed sandy soil	CD TC tests
△	Prepared in the Lab., Cement-mixed sandy soil	CU TC tests
▽	Ukishima Access, Low strength-type DMM, tc=91 days	CU TC tests
◎	Kawasaki Island, DMM, tc=2.5 years	CU TC tests
□	Kawasaki Island, Slurry type fill, tc=28-91 days	CU TC tests
◇	Prepared in a mold, Slurry-type, tc=14-91 days	CU TC tests
◆	Full-scale experiment, Slurry-type fill, tc=116-190 days	CD TC tests
+	Full-scale experiment, Slurry-type fill, tc=100-188 days	CU TC tests
×	Ukishima Access, Slurry-type fill, tc=300 days	CU TC tests
●	Full-scale experiment, Cement-mixed sandy soil (dry type)	CD TC tests
○	Full-scale experiment, Cement-mixed sandy soil (dry type)	CU TC tests
▼	Kisarazu Island, Dry-type fill, tc=120-290 days	CD TC tests

Figure 265. Summary of E_{max}/Q_{max} relationships for cement treated soils (Tatsuoka et al., 1996).

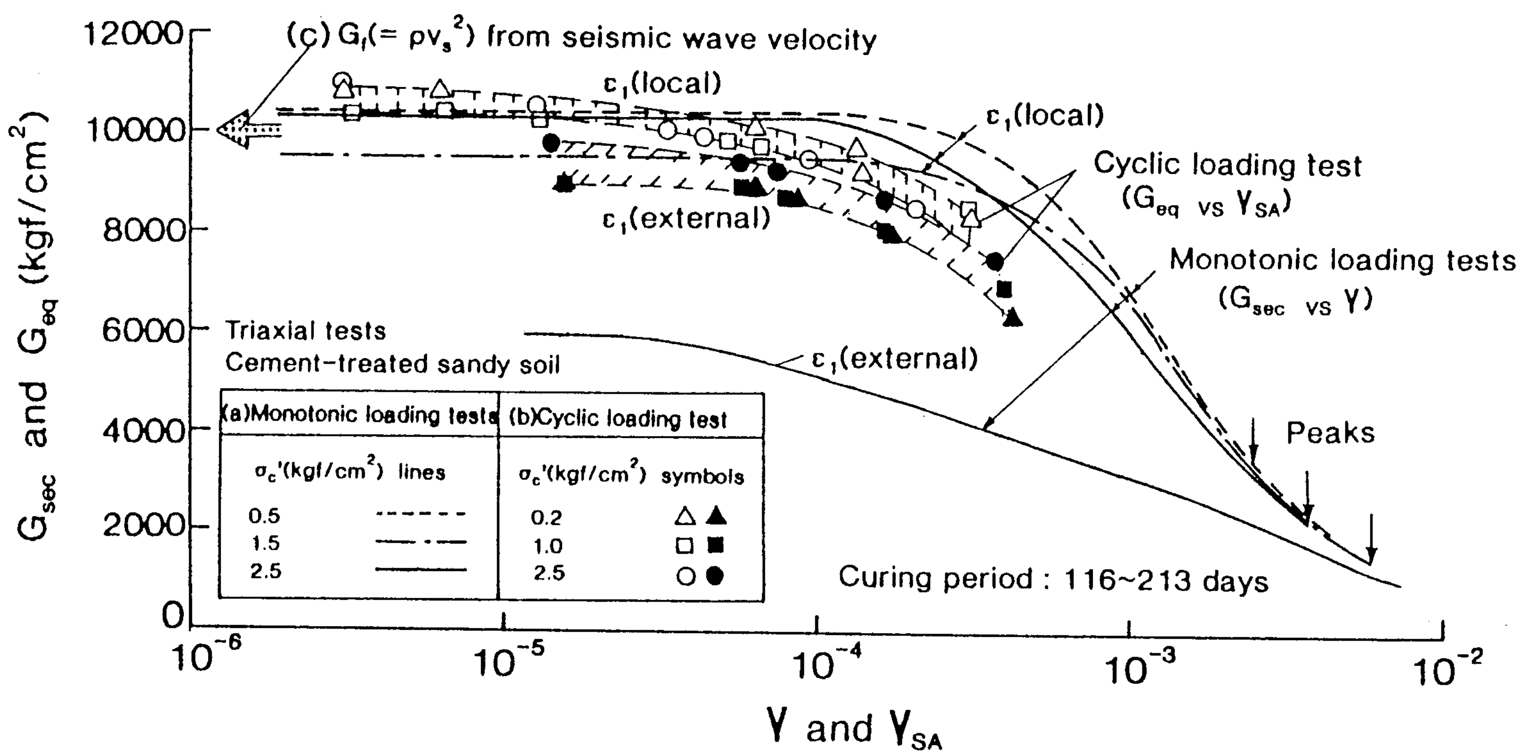


Figure 266. Comparison of strain-level dependency of secant shear moduli (Shibuya et al., 1992).

modulus values of about twice those from tests made with external dial gauge readings. Finally, as peak strengths are reached at strains of 0.003 to 0.008 (or 0.3% to 0.8%), the modulus values from the two measurement methods become closer and nearly converge.

Similar trend of strain level dependency is shown in Figure 267 wherein the E_{max} modulus values have been normalized with Q_{max}. (Note that there is a difference in strain designation of 100 times between Figures 266 and 267, i.e., direct strain versus percent strain.) The data in Figure 267 also include results of in situ tests (PLT - plate load tests, and BHLT - borehole loading tests). In their respective strain ranges, the BHLT determined modulus values that are slightly greater than laboratory tests, but the PLT substantially underestimates the modulus. Both tests induce large strains, greater than 0.1%, which would negate their applicability for use in small strain modeling situations. However, for large strain situations, the borehole loading test (or pressuremeter) appears to have potential applicability. But it is of utmost importance to be sure that tests are made on representative specimens of soil-cement, whether in situ or on core samples. The degree of soil-cement variability (or conversely uniformity) of the mixing, key aspects in any modeling applications, can far outweigh the importance of the method used to determine modulus.

Appropriate ranges in the ratio of E_{max}/Q_{max} that are derived from Figure 267 appear to suggest ratios of 1000 to 5000 for small strains (less than 0.01%), and lesser values of 500 to 100 for large strains (greater than 0.1%). It must be emphasized that these data are for cement treated sands. Although for clay soils the modulus/strength ratio appears somewhat smaller (see Figure 265), it is reasonable to expect this would only cause a downward shift in the trend envelope of decreasing modulus with increased strain.

Soil-cement behavior at strains less than yield (i.e., smaller than 0.1 to 0.2% strain) is due principally to cementation effects. At larger strains, as cementation bonds are broken down, the behavior shifts to more frictional, which for clay soils is a derivative of the cement treatment. In either case the fundamental strength and stress-strain behavior of soil-cement results from the cementation effects, and therefore normalized relationships such as illustrated in Figure 267 should find wide applicability in the modeling of soil-cement structure behavior. The data in

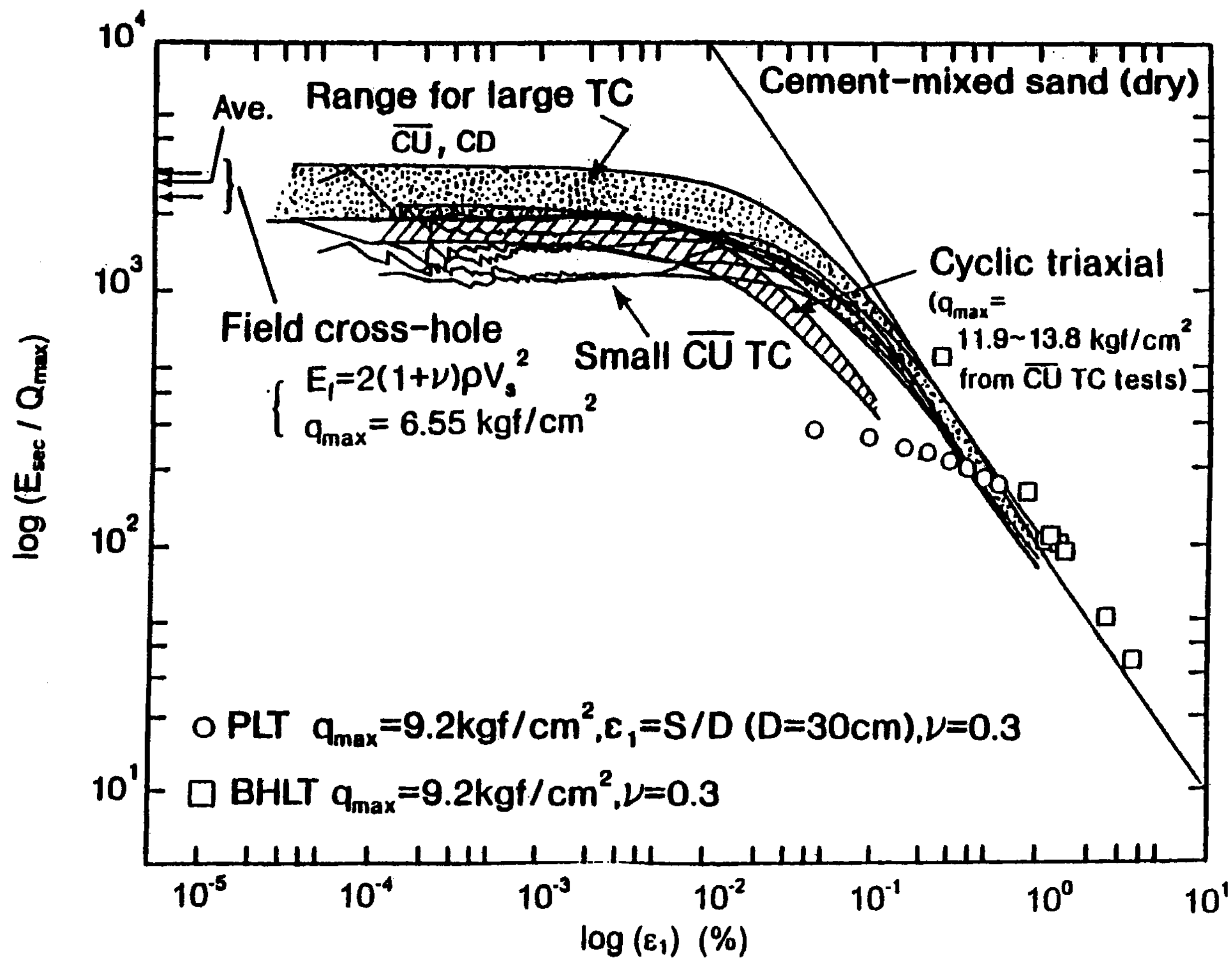


Figure 267. Comparison of Young's moduli vs. strain for treated sandy soil (Kohata et al., 1992).

Figure 268 illustrate that other natural "geo-materials" display similar relationships between modulus and strength, and have ratios of E_{max} to Q_{max} similar to those found for soil-cement.

6.4.3 Other Factors in Treated Soil Modulus Behavior

The dependency of strength on cement factor is well established. Figure 269 shows that shear modulus is also dependent on a_w, with there being a strong effect in the small strain behavior. As the a_w increased from 10 to 30%, the shear modulus was found to increase 5 to 10 times for small strain ranges of tests performed. When larger shear strains were experienced, only about a threefold increase in modulus was observed. These shear modulus values versus a_w data were not reported with unconfined compressive strength, so ratios of E/Q could not be evaluated.

Also, the type of cementing agent has been found to affect the stress-deformation behavior of soil-cement. Figure 109 illustrates that use of a combination of flyash, gypsum, and cement produces lower modulus than just using cement. The ranges in the ratio of E_{50} to unconfined compressive strength were found by Asano et al. (1996), to be two to three times greater when using cement alone. The conclusion reached is that lower modulus can be achieved without significant reduction in strength by substituting flyash and gypsum for some portion of cement. However, the chemistry of the flyash must be thoroughly evaluated, because it is dependent on coal composition and power plant combustion process.

Confining pressure has not been found to have a strong influence on modulus of soil-cement. For cement treated sandy soils, some apparent influence of confining pressure was evident for test samples with "externally" measured strains, but for local measurements, no such relationship was observed. This is reasonable, particularly when effective confining pressure is less than the yield point. At such pressures, the cementation bonds control the stiffness of the soil-cement behavior.

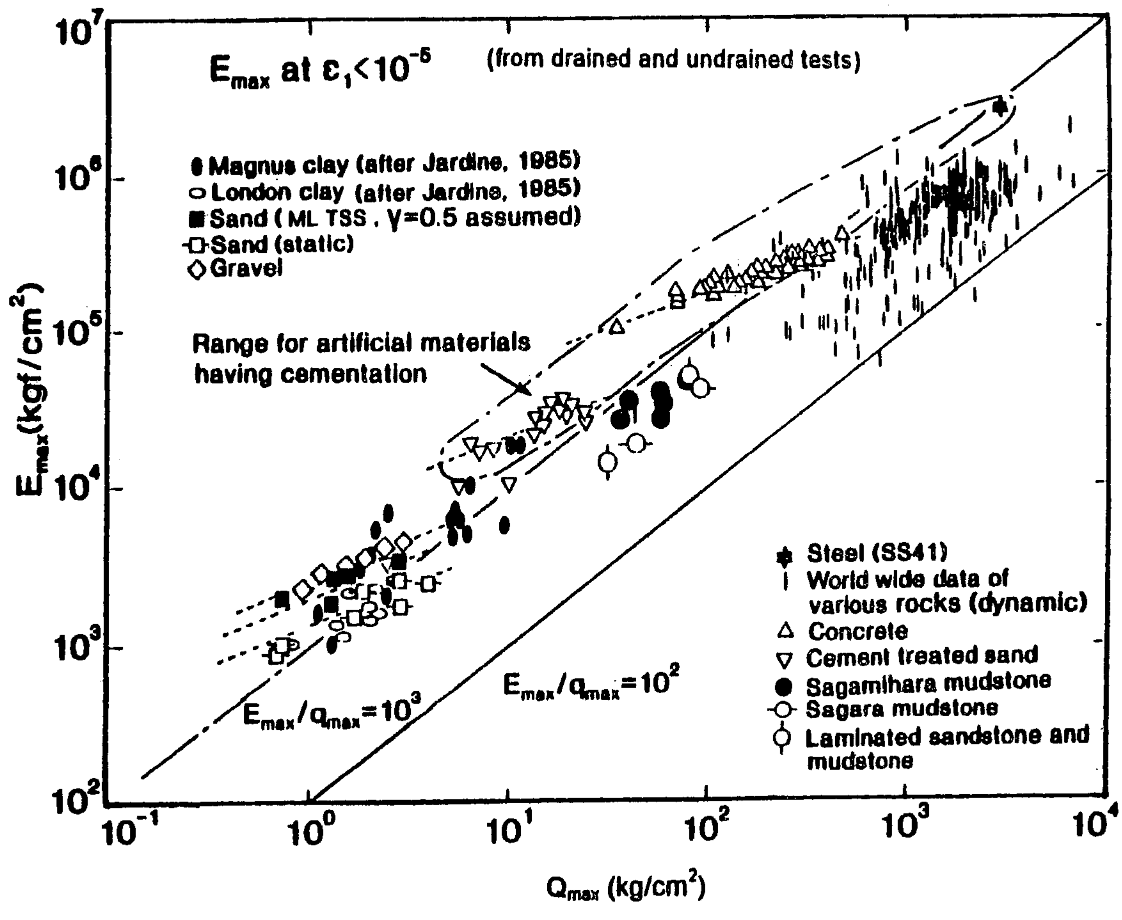


Figure 268. Relationship between E_{max} and Q_{max} obtained from laboratory shear tests for a wide range of geo-materials (Kim et al., 1991).

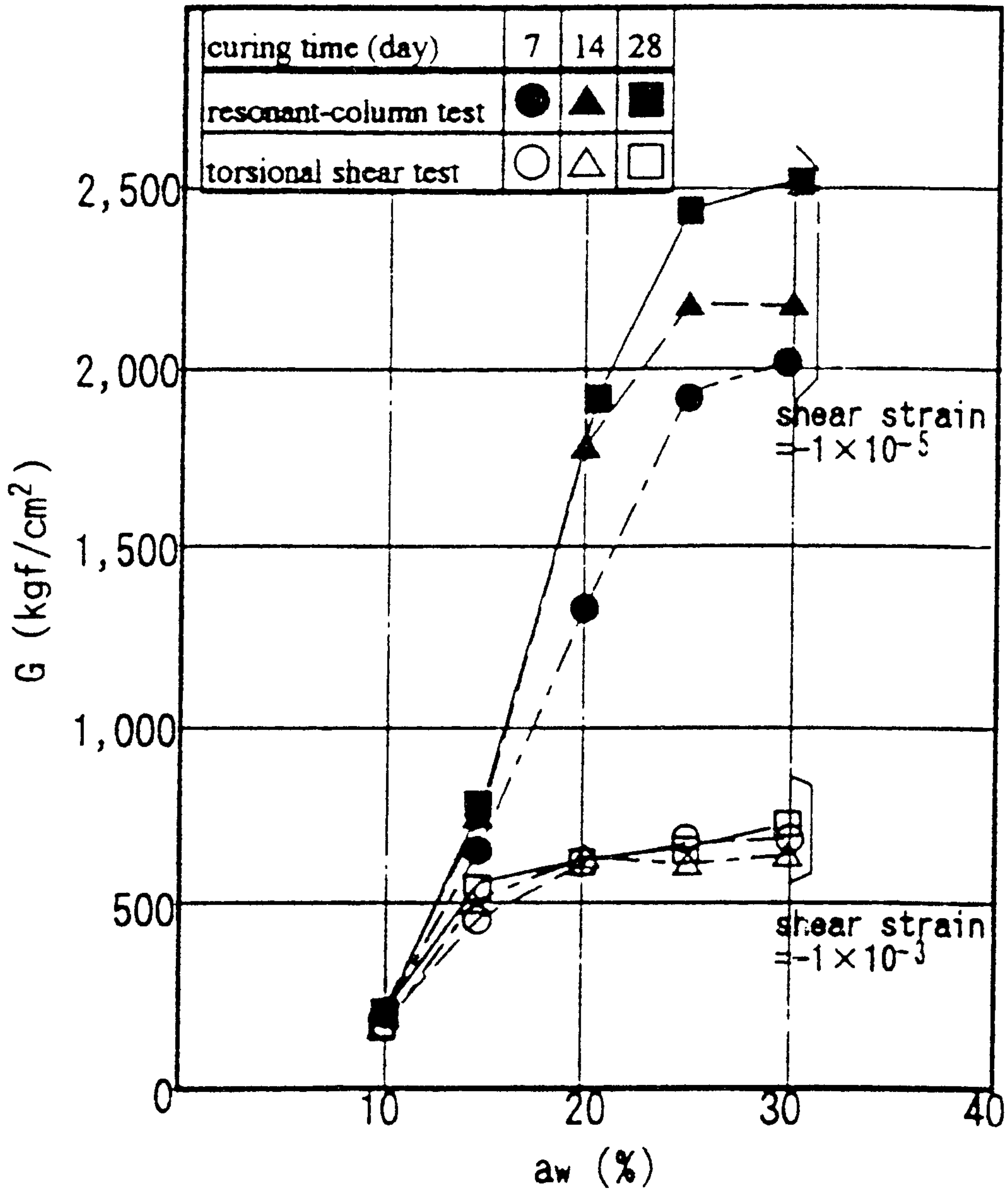


Figure 269. G-a_w relationships (Hirama and Toriihara, 1983).

6.5 Examples in Case Histories

Only a few case histories of soil-cement applications have been published that report both the analysis method predictions and the specific properties used to estimate soil-cement structure deformations. It is instructive to review predictions versus performance to better understand actual in-service soil-cement deformation behavior, as it relates to the assumptions made on modulus. These represent only a few of the numerous case histories reported on soil-cement structure installations, but most do not have sufficient data for assessment of appropriateness of model or soil-cement properties.

6.5.1 Wharf Structure on DMM (Hosomi et al., 1996)

The lateral movements of a block of soil-cement used as foundation for a wharf retaining wall structure are shown in Figure 270. Horizontal movements measured over the 25-m height of the block were less than 30 mm. Although data on assumed soil-cement modulus are not presented, the movements recorded are quite small. However, even at shear strain of about 0.001 (0.1%) over the entire block height, strains are beyond the lower limit of "small" strain.

6.5.2 Excavation Side Slope Stability Using DMM Block (Shiomi et al., 1996)

Construction for expansion of the Tokyo International Airport required a large excavation, for which soil-cement buttresses were installed. Displacements of cement-treated soil block were measured, and are shown in Figure 271 and Figure 272. The larger of these lateral movements are on the order of 100 to 150 mm over a height of 20 to 30 m, a shear strain in the soil-cement of about 0.005 (0.5%), which is again beyond the "small" strain range. The measured and calculated displacements are in close agreement; however, the modulus values used are not reported.

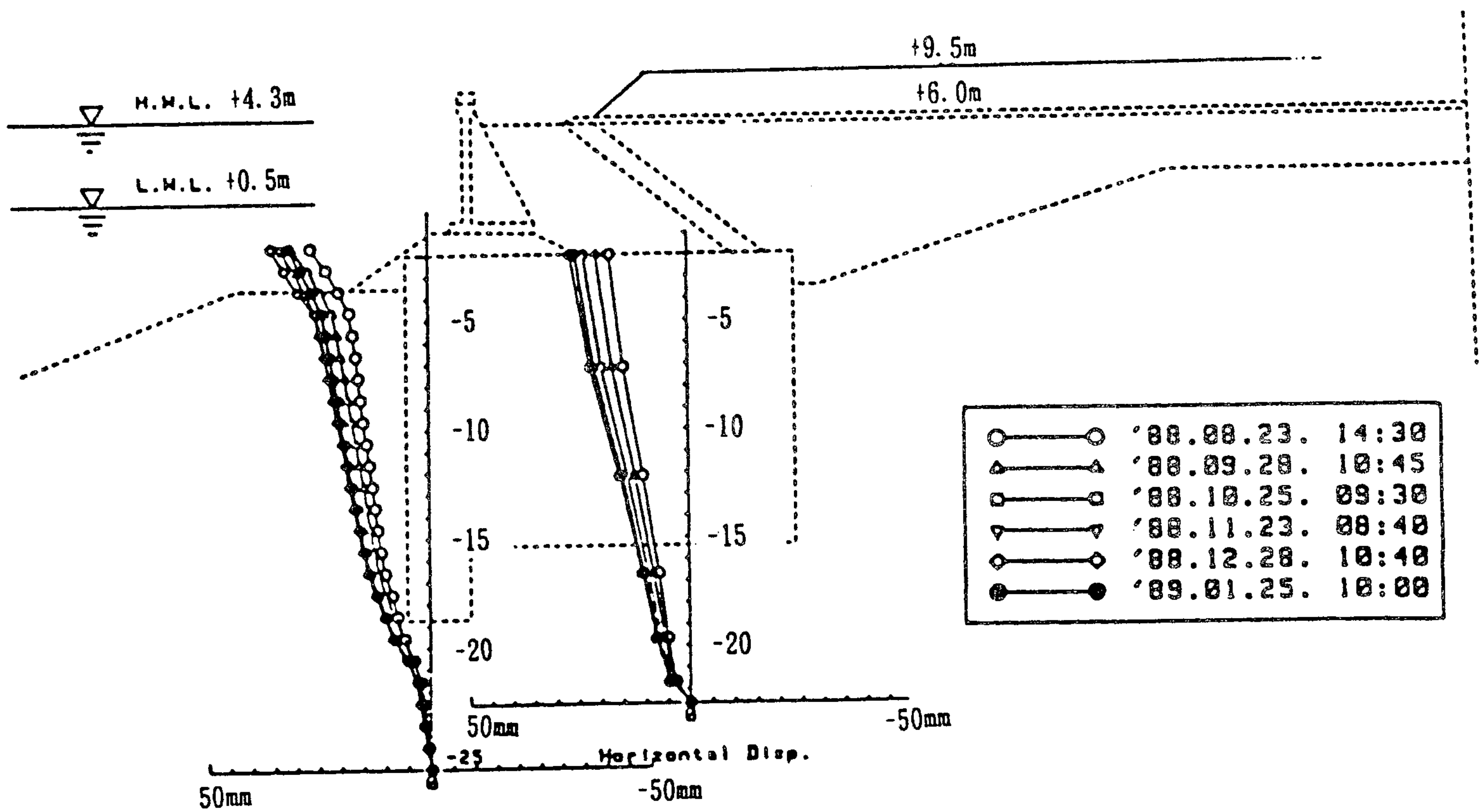


Figure 270. Line A deformation according to field observations (Hosomi et al., 1996).

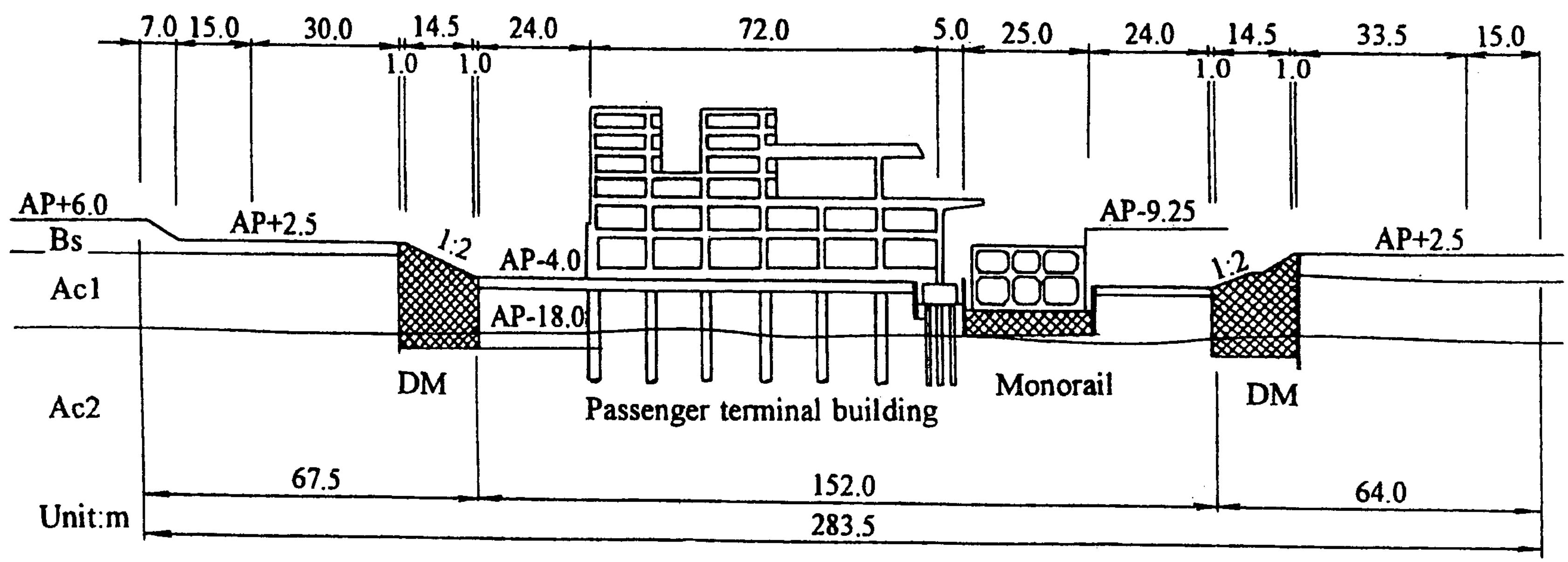


Figure 271. Typical cross section of excavation work (Shiomi et al., 1996).

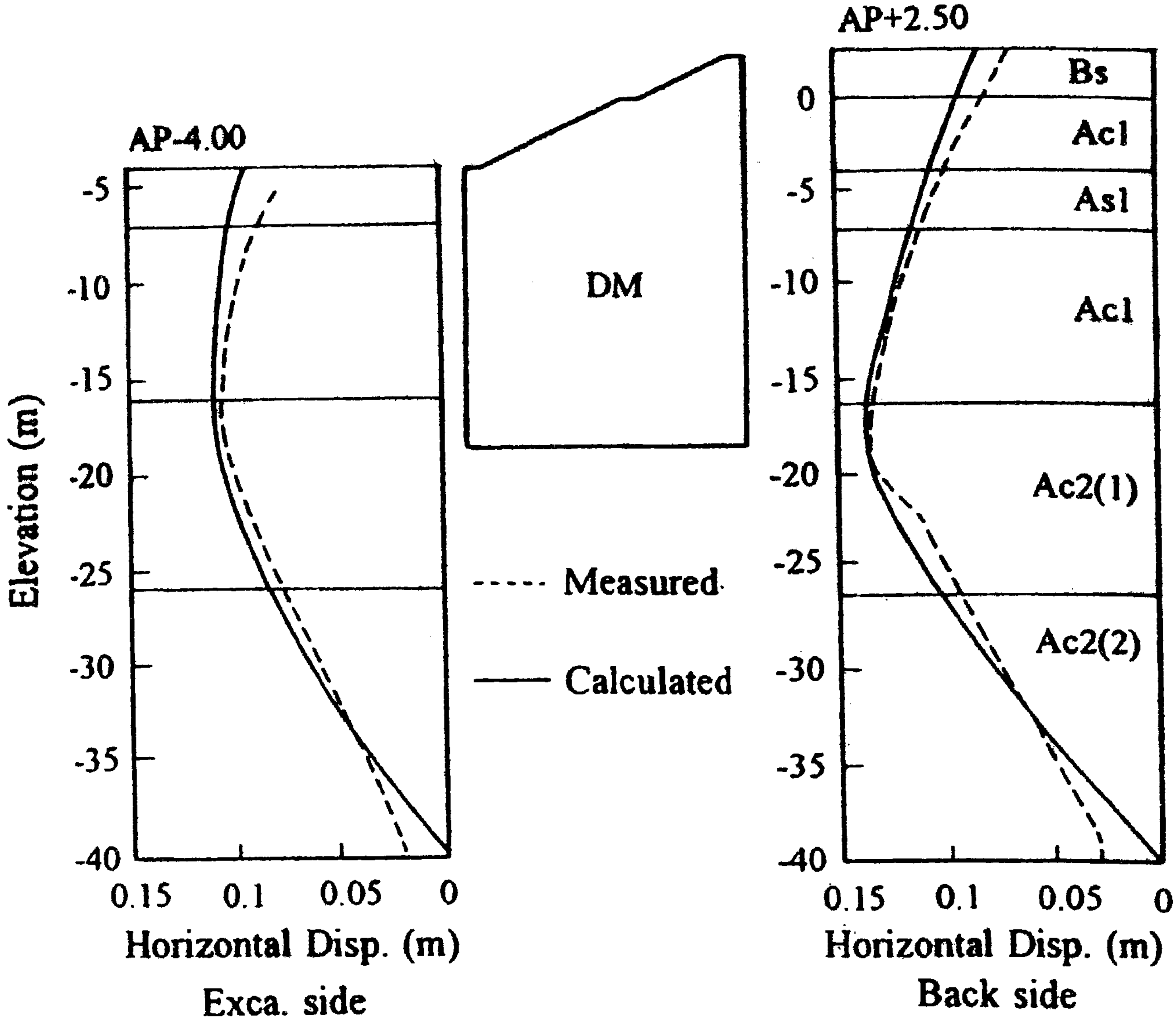


Figure 272. Displacement of the treated soil (Shiomi et al., 1996).

6.6 General Overview

The stress-strain behavior of treated soil depends on the soil type, amount of cement mixed into the soil, mode of shear stress application, drainage conditions, and level of strain that will be experienced. As the cement content increases, the soil-cement becomes stiffer, and the stress level over which elastic strain occurs increases. The effect of confinement makes soil-cement deform with ductility, such that sudden breakage or failure does not occur, but increasing confining pressure can counteract the inherent cementation effects if applied after the soil-cement has solidified. In undrained shear, positive pore pressures are generated that increase to create conditions of essentially no effective confining pressure. Therefore, undrained shear will produce similar strengths regardless of initial confining pressure. The behavior of soil-cement during drained shear depends on the confining pressure. Figure 273 illustrates general stress-strain responses of soil-cement for unconfined, undrained, and drained conditions.

The appropriate modulus to use in modeling soil-cement behavior is highly dependent on the strain level. Peak strength will usually be experienced before reaching about 2% strain. But in most situations, particularly where the soil-cement is to act as a separate, or independent structure, tolerable strains must be held to much smaller levels. For modeling seismic loading conditions where strains are less than 0.01%, small strain modulus values are required, which can be adequately determined from tests using shear wave velocity. As strains become greater than 0.05 %, modulus values decrease. If measured on laboratory compression test specimens, care must be taken to eliminate the effect of load system compliance, which artificially increases the "measured" strain. If "large" strains are expected to occur (i.e., greater than 0.1%), then modulus values will be in the "small" range. It is common to relate modulus to the unconfined compressive strength, and a ratio of 1:1000 may be applicable for strains of 0.05 %, whereas at larger strains of 0.5% the ratio may be 1:150.

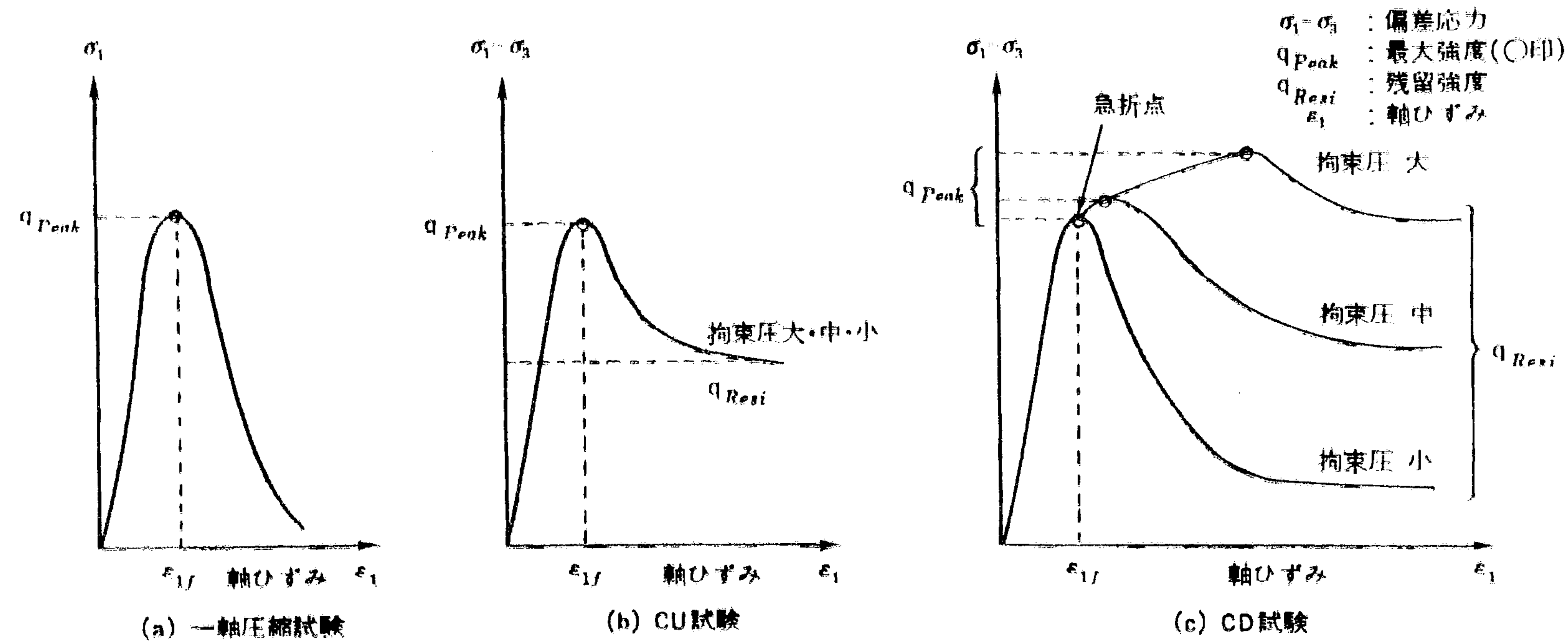


Figure 273. General stress-strain response of soil-cement for unconfined, undrained, and drained conditions (CDM Association, 1994).

7. REVIEW OF JAPANESE GEOTECHNICAL SOCIETY COMMITTEE AND ASSOCIATION REPORTS AND "SPECIAL LECTURES" IN 1996

In the supplementary volume of the 1996 Proceedings of the Tokyo Conference on Grouting and Deep Mixing, a series of Technical Committee Reports was presented. These five subcommittees (or divisions) operate under the Japanese Geotechnical Society Technical Committee on Cement Stabilization Techniques (established 1993) and address specific aspects of DMM technology, as summarized in Table 75. The major objectives of the Committee are to accumulate and evaluate these techniques, knowledge, and experiences to both update the current databases and to investigate new applications and future challenges.

The composition of these subcommittees reflects the close collaboration between contractors, suppliers, owners, consultants, and universities that characterizes Japanese technology. The amount of literature they have reviewed further underlines the intensity of the research conducted over the years. The great bulk of it, however, was published only in the Japanese language until recently.

These subcommittee reports are therefore highly significant in that they provide very valuable and comprehensive summaries, which lend insight, perspective, and cohesion to those data that have been published in English and that have been discussed elsewhere in this report. This chapter therefore provides a synopsis of the key elements of four of these committee reports; design aspects are beyond the scope of this report. In addition, very useful summaries were presented by the CDM Association and Okumura at the 1996 Tokyo Conference. These give valuable and comprehensive overviews and so are also reviewed below.

Equally valuable are the Proceedings of the International Conference on Dry Mix Methods for Deep Soil Stabilization, held in Stockholm, Sweden, in October 1999. However, data from the papers presented are incorporated elsewhere in this report, principally in Chapters 3 and 5, and no "committee report" type format was adopted. For these reasons, the findings of the

Table 75. Nature and composition of various Divisions of the Japanese Geotechnical Society Technical Committee on Cement Stabilization Techniques.

Committee Name and Title of Report	Committee Members		Sources of Data
	Name	Affiliation	
Cement-Treated Soil Properties "Deformation and Strength Properties of DM Cement-treated Soils"	Kohata	Railway Technical Research Institute, Tokyo	290 papers in 10 years
	Muramoto	Railway Technical Research Institute, Tokyo	
	Maekawa	Kanazawa Institute of Technology	
	Yajima	Tekken Corporation, Tokyo	
	Babasaki	Takenaka Corporation, Chiba	
Cement Stabilization Technique "Factors Influencing the Strength of Improved Soil"	Babasaki	Takenaka Corporation, Chiba	231 test results from 69 sites in Japan
	Terashi	Nikken Sekkei Nakase Geotechnical Institute, Kawasaki	
	Suzuki	Hiroshima Institute of Technology	
	Maekawa	Miyoshi Shokai, Yokohama	
	Kawamura	Nihon University, Chiba	
	Fukazawa	Kajima Corporation, Tokyo	
Method Evaluating Strength "An Evaluation of the Strength of Soils Improved by DMM"	Hosoya	Obayashi Corporation, Tokyo	84 papers in 10 years
	Nasu	Onoda Chemico Corporation, Tokyo	
	Hibi	Kisojiban Consultants, Tokyo	
	Ogino	Chuken Consultants, Tokyo	
	Kohata	Railway Technical Research Institute, Tokyo	
	Makihara	Tokyo Soil Research Company	
Design "Japanese Design Procedures and Recent Activities of DMM"	Kitazume	Port and Harbor Research Institute, Ministry of Transportation	21 major papers and reports (1978 to 1994)
	Miyake	Technical Research Institute, Toyo Construction	
	Omine	Kyushu University	
	Fujisawa	East Japan Railway Company	
Methods of Execution "Factors Affecting the Quality of Treated Soil During Execution of DMM"	Yoshizawa	JDC Corporation, Tokyo	About 400 papers
	Okumura	Takenaka Company, Tokyo	
	Hosoya	Obayashi Corporation, Tokyo	
	Sumi	Geotop Corporation, Tokyo	
	Yamada	Konoike Construction Company, Osaka	

Stockholm Conference are not treated independently in this chapter, in the manner of those from the Tokyo Conference.

7.1 Deformation and Strength Properties of DM Cement-Treated Soils

This committee confirmed that the strengths of treated soil depend on a wide range of factors, but that a laboratory test (particularly triaxial compression) had not been standardized. U.C.S. is used since this is a relatively simple test. They have therefore made recommendations for various laboratory testing methods (procedures for making specimens; and methods for uniaxial and triaxial testing) and they discuss deformation and strength properties.

7.2 Factors Influencing the Strength of Improved Soil

These factors, up to 1985, were considered to be those listed in Table 6. The major observations of the committee included:

7.2.1 Types and Characteristics of Binders

Cement and lime are most popular, but "dozens" of materials are available especially for treating clays with high moisture content, or organics, or where "controlled" rates of strength increase are required (as in the FGC-CDM method). They list four subcategories of binders:

1. Lime type: both quick and slaked.
2. Standard cements: ordinary portland and portland blast furnace (OPC and blast furnace slag: facilitating pozzolanic reaction and forming fine C-S-H type hydration products to enhance long-term strength).
3. Special cements: by adjusting chemistry, grain size or using additives for treatment of organics (high Alumina) or low-strength soils (gypsum or lime).
4. Industrial wastes or byproducts: mainly those that induce pozzolanic reaction, e.g., various types of slag and ash.

7.2.2 Characteristics of Treated Soil

Based on 231 test results from 69 Japanese sites, and using ordinary portland cements, 28-day core data were presented, the highlights of which were:

Figure 274: Shows relation of cement content (a_w) and U.C.S. (q_u). For the same cement factor, q_u varies considerably with soil type, the finer grained soils being associated with the lower strengths.

Figure 275: Shows relationship between moisture content (total moisture content, i.e., including porewater, and mixing water) and U.C.S. (q_u) for three different cement factors. For any specific soil, the lower the water content, and the higher the cement content, the greater the strength. However, even with constant mix parameters, strength may vary considerably due to subtle differences in the soil. There appears to be minimal increase in strength in soils with moisture contents greater than 200% with increasing cement factor (sludges, marshy soils, fills). Also some soils with moisture contents less than 200% cannot be improved simply by increasing cement factor (due to presence of organic acidic soils with low pH).

Figure 276: Shows relationship between ignition loss and U.C.S. (q_u) (limited data). Generally for ignition loss greater than 10%, U.C.S. remains low. The circled data have humus contents greater than 0.9% (high).

Figure 277: Shows relationship between pH and U.C.S. Most soils with pH less than 5 show little increase in strength.

In Figures 275 through 277, the "individual characteristics" of the soil control results (especially the characteristics of the fine grained portion – type of clay material, soil consistency, etc. – and not the characteristics of the sand content).

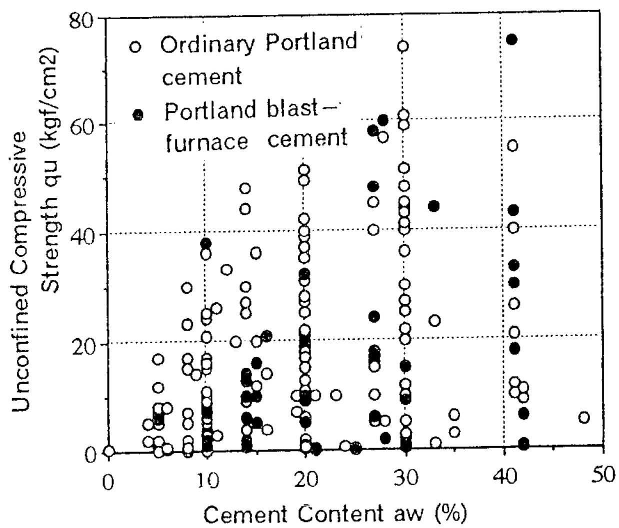


Figure 274. Relationship between unconfined compressive strength, q_u, and cement content, a_w (Babasaki et al., 1996).

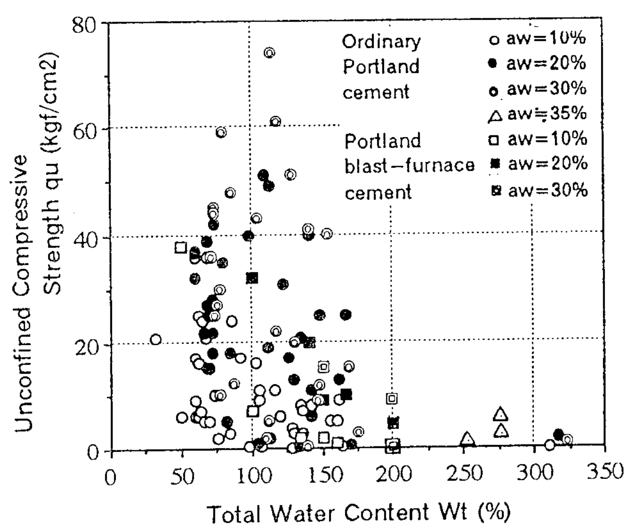


Figure 275. Relationship between unconfined compressive strength, q_u, and water content, wt (Babasaki et al., 1996).

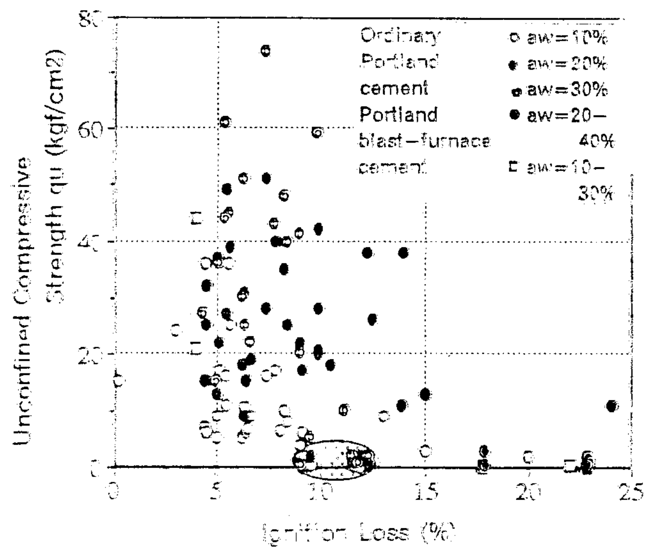


Figure 276. Relationship between unconfined compressive strength, q_u, and ignition loss (Babasaki et al., 1996).

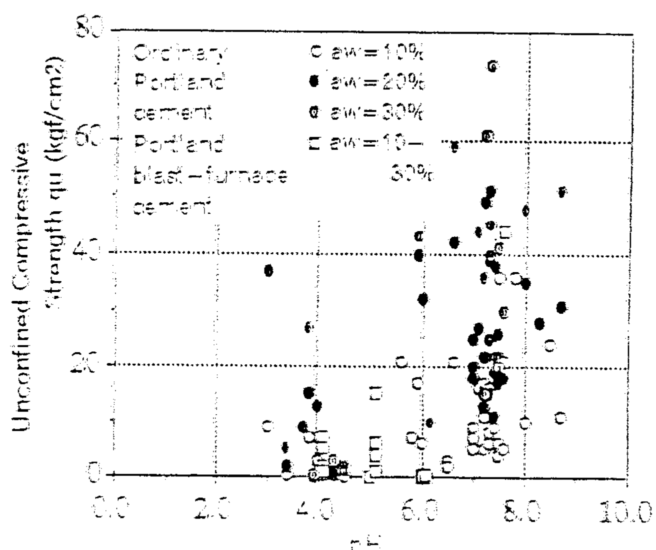


Figure 277. Relationship between unconfined compressive strength, q_u, and pH value (Babasaki et al., 1996).

7.2.3 Curing Conditions

When exposed to water, treated soil may deteriorate in the long term (rate is inversely proportional to log of time). Figure 278 shows the relationship of q_u and curing temperature for the same silty soil mixed by the same DMM process. Given that "maturity" is defined as the curing temperature plus 10°C times the curing time in days, attempts have been made to express the relationship of temperature, time, and strength in different soils (Figure 279). The authors concluded that the temperature does not have a significant effect on long-term strength, but does have a considerable effect on short-term strength. Furthermore, the generation of heat of hydration is significant (Figure 280) – being higher the greater the volume of treated soil. Ohmura et al. (1981) showed the ground temperature to be in excess of 50°C for 100 to 6,000 hours after treatment on one particular site. This scale effect is very significant when considering the relevance of tests on laboratory produced samples.

The authors concluded that "we are not yet at the stage where we can determine a widely applicable formula (to predict U.C.S.) that incorporates all the relevant factors." Therefore, laboratory mold tests are made using site soils, and then field test results are planned based on "past experience." Large projects are often begun with field tests, while smaller projects are designed based on prior experience "in the similar area." There are also cases where a formula can be derived, but it can only be based on test results applicable for that specific site alone.

7.3 An Evaluation of the Strength of Soils Improved by DMM

7.3.1 Influence of Evaluation of U.C.S. Values Based on Cores

The method and skills for obtaining 50- to 100-mm diameter cores from columns of diameters 1 to 2 m are critical and result in a scatter of results, especially since those lengths of cores in "good condition" tend to be chosen. The impact of this selection process on the nature of the resultant data is shown in Figure 281, for a theoretical case.

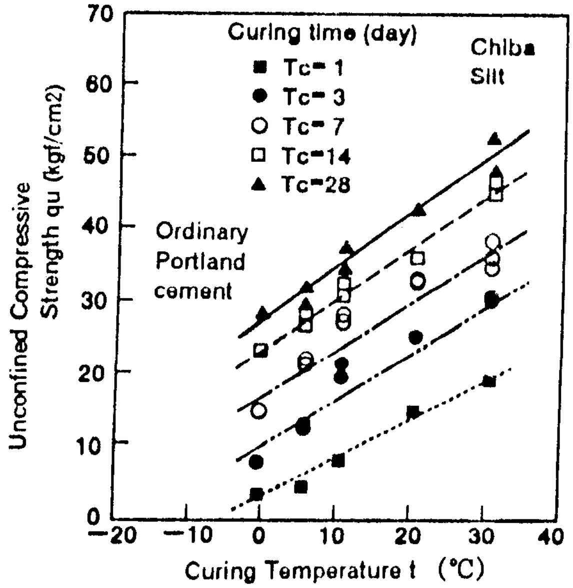


Figure 278. Relationship between unconfined compressive strength, q_u, and curing temperature, t (Babasaki et al., 1996).

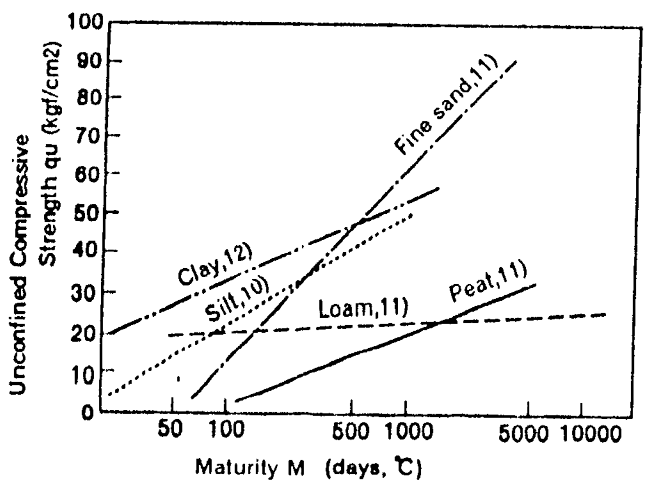


Figure 279. Relationship between unconfined compressive strength, q_u, and maturity, M (Babasaki et al., 1996).

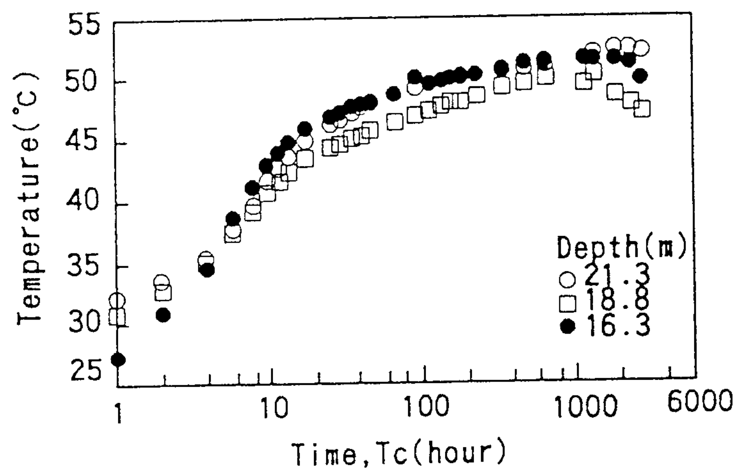


Figure 280. On-site measurements of hydration-generated heat in improved ground (Babasaki et al., 1996).

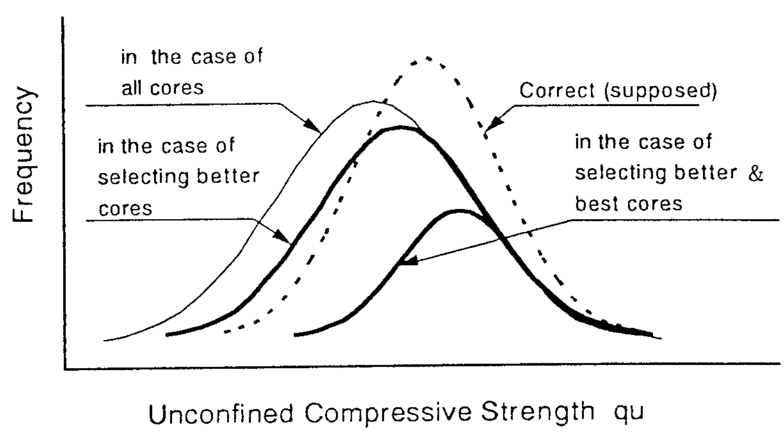


Figure 281. Histogram-image of unconfined compressive strength, q_u, of improved soils (Hosoya et al., 1996).

7.3.2 Influence of Coring Proceedings

Double or triple core barrels are used, and are typically of larger diameter (86, 116 mm) in "irregular" soils. Cracks in the cores may result from borehole deviation (Figure 282), the rigidity of the sampler, locking mechanism of the sampler, and rotation of the core in the sampler. Deviation "strains" the core and "disturbs" the sample. Also, as shown in Figure 283, strength apparently decreases with increasing diameter (up to 150 mm), and the derived relationship is $q_{u400} = 0.87q_{u60}$. "As a result, a sampler with diameter of 150 mm is a criterion for soil sampling." They also concluded that larger diameter samples give better RQD values.

7.3.3 Problems of Laboratory Tests

Figure 284 shows that the values of q_u obtained from unconfined compression testing on in situ samples are highly variable (because of cracks in the samples) whereas the strength q_{max} obtained from triaxial tests (c_u) is generally higher and has less scatter. The latter is therefore preferred as a true indicator of strength.

7.3.4 Review of Different Methods

Table 76 lists the various methods for assessing treated soil. SPT values are considered "coarse" but of "some merits." The committee prefers the Rotary Penetrating Test (RPT), being simpler and easier to use (Figure 285). Figure 286 shows the relationship between q_u and the "factor of drilling energy," E_s, as derived from integrating bit thrust and torque readings.

7.3.5 Future Developments

They refer to the new coring methods developed by Sugawara et al. (1996), which can only reduce coring disturbance at best, although further developments are expected to be announced. They recommended increasing the use of triaxial testing, as well as making better use of the in situ stiffness values (Tatsuoka and coworkers) obtained from field elastic shear wave tests.

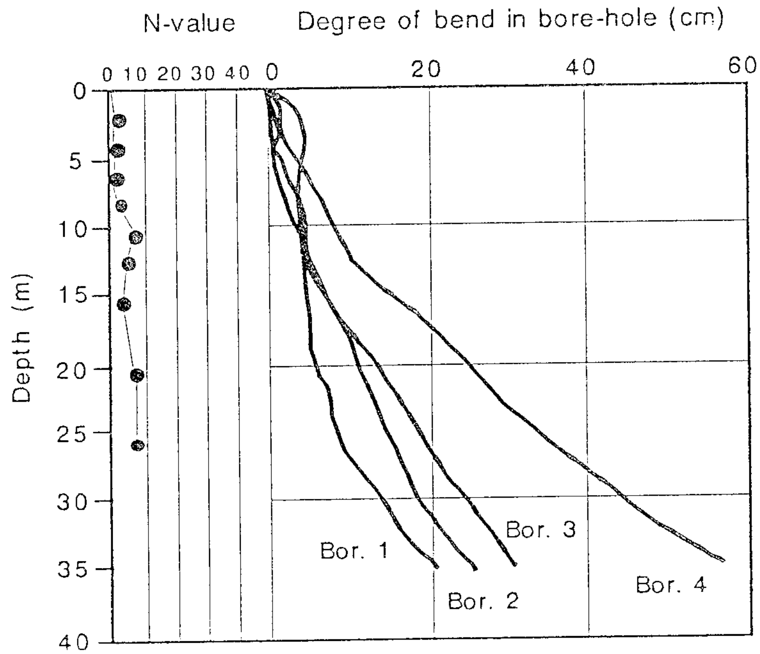


Figure 282. Comparison of degree of bend in boreholes (Suzuki et al., 1992).

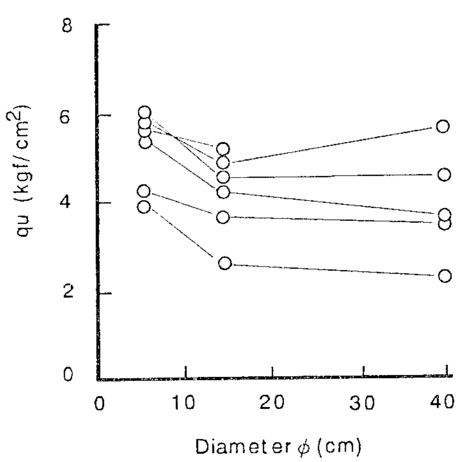


Figure 283. Relationship between diameter of sample, ϕ, and unconfined compressive strength, q_u (Saitoh et al., 1982).

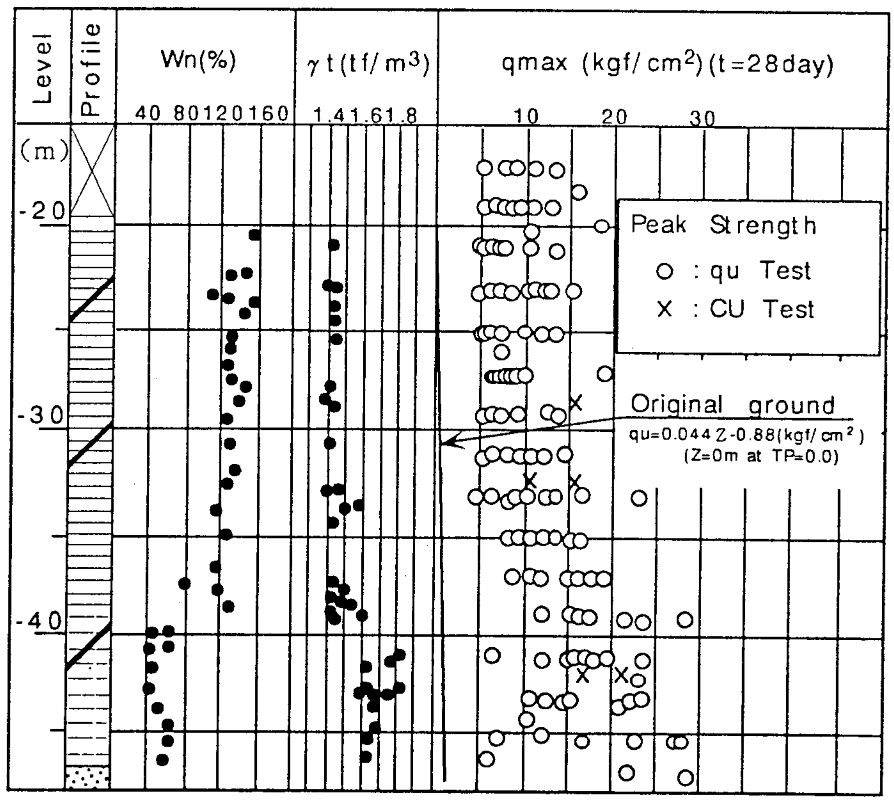


Figure 284. Comparison of unconfined compressive strength and triaxial compressive strength on field improved soil (Uchida et al., 1993).

Table 76. On-site test for evaluating stabilized soil (Hosoya et al., 1996)

Type of test	Method	Test method and results	Comment on quality control method
Sounding test	Standard penetration test	Let 67.5kgf hammer fall free from 75cm height and count the number of strokes (N-value) to penetrate 30 cm.	Most common method on natural soil. However, only a few applications for stabilized soil are available. There is correlation with unconfined compressive strength.
	Dynamic cone penetration test	Let 5kgf hammer fall from 50cm height and count the number of strokes (Nd-value) for cone to penetrate 10cm.	Easy transportation and operation. Practical for unconfined compressive strength of qu=2~5 kgf/cm2. qu=0.29Nd-2.58(kgf/cm2)
	Electric static cone penetration test	Let the cone penetrate at uniform speed and measure the resistance at the end and surrounding surface and pore-water pressure in sequence.	Applicable to measure the improvement of low strength stabilized soil in sequence. However, not applicable for firm stabilized soil.
	Rotary penetrating test	Measure the bit pressure, torque, and muddy water pressure by the sensor at the end of the boring rod to observe the soil strength in sequence.	Greater mobility compared with core sampling and in-situ strength can be measured. However, correlation with the unconfined compressive strength must be compared from site to site.
Test of utilizing bore-hole	PS logging	Measure the velocity of P and S waves. Then calculate rigidity and Poisson's ratio of stabilized soil. There are two testing methods, Down hole method and Suspension method.	There is some correlation with unconfined compressive strength although it is not so uniform. Suspension method is better to evaluate the stabilized soil.
	Electrical logging	Supply electricity to stabilized soil and measure electric current and voltage through an electrode. Then calculate the specific resistance. Lately, analysis is made by tomography in some cases.	The correlation with unconfined compressive strength is low.
	Density log	Measure the gamma rays emitted from a probe inserted into the hole by the detector installed at a certain distance. Then convert the data into density.	Since it is influenced by hole diameter and water inside the hole, calibration is important. There is no correlation with unconfined compressive strength.
Loading test	Bore-hole lateral load test	Press rubber tube toward the bore-hole wall in stages and measure the strength and deformation modulus of stabilized soil. Mesurement apparatus is Pressio metre and LLT.	Deformation modulus rather than strength is often the objective of the tests. Vertical measurement is costly so it is used as representative value of stabilized soil.
	Plate loading test	Place a loading plate (round plate of 30cm diameter) on the stabilized soil and put on load in stages. Bearing capacity and deformation characteristics can be obtained directly from the load and settlement curves.	Bearing capacity and deformation characteristics can be obtained directly. However, the evaluation of stabilized soil is possible only down to the depth of 2 to 3 times of(load) plate diameter.
	Stabilized pile loading test	Place the (load) plate of the same diameter as the stabilized pile on the surface of the leveled stabilized pile. Then put on load in stages. This is a sort of vertical load test of a pile where bearing capacity characteristics are obtained from the load and settlement curves.	Bearing capacity characteristics of a stabilized pile can be directly obtained. However,testing equipment is costly and the number of tests available is limited.
Non-destructive test	Integrity test	Safely stroke the surface of a stabilized pile with a hammer and measure the reflected wave of the vibration by the accelerometer installed on top of the pile. Length and discontinuity of the stabilized pile is measured.	Simple method. However, evaluation standard for a stabilized pile has not been established yet.
	Elastic wave exploration	Emit P and S waves to measure the velocity distribution of stabilized soil. In the case of stabilized soil, measurement of S wave is preferred.	Stabilized condition is measured by velocity distribution of the S wave. The measurement is made in the bore-hole and on the ground surface. Tomography is used to improve accuracy of the test.
Other test	Penetration test	Use pocket type pin penetration apparatus and measure the penetration resistance of stabilized soil on the job site. Then estimate the unconfined compressive strength.	Easy and simple method. A lot of tests can be done. However, only the surface of the stabilized soil can be tested.

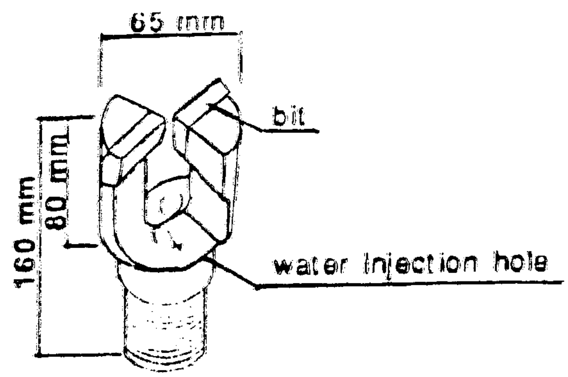


Figure 285. Drilling bit of RPT (Shimatsubo et al., 1992).

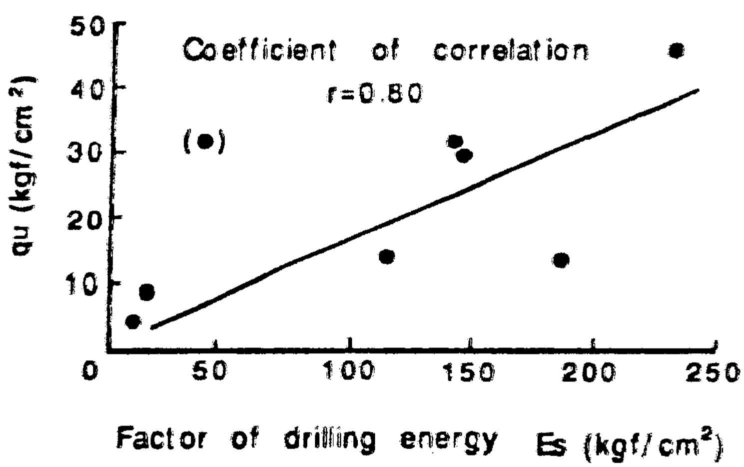


Figure 286. Relationship between q_{u28} and E_s of RPT (Shimatsubo et al., 1992).

7.4 Factors Affecting the Quality of Treated Soil During Execution of DMM

Regarding the factors influencing the resultant properties during execution, this committee considered two groups: "insertion conditions" and "mixing conditions."

7.4.1 Insertion Conditions

1. Type of cement. Figure 287 illustrates for a sandy soil (less than 5% fines) the benefit of using blast furnace cement. However, this benefit is not always achieved, depending on the soil and the type of DMM (slurry or dry).

2. Water cement ratio. Figure 288 shows a marine clay with a moisture content of 100%. As water/cement ratio increases, the average U.C.S. drops and the coefficient of variation increases, both indicative of a reduction in quality. Water/cement ratio is usually limited to 1.0 as a result.

3. Volume ratio. Figure 289 shows an increase in strength (for a laboratory test) and decrease in variability with increasing volume ratio. As a result, values of 20 to 30% are most common for "practical applications."

7.4.2 Mixing Conditions

1. Number of mixing shafts. Figure 290 shows that there was less variation and higher strength with four shafts than with one, due to opposite rotation enhancing the efficiency of mixing.
2. Mixing blades (Figure 291 and 292). Experiments confirm that the orientation and geometry of the blades can markedly affect strength, especially in "highly viscous ground." Various mechanical developments being experimented with to improve quality include anti-rotation vanes, counter-rotating blades, and "share blades."

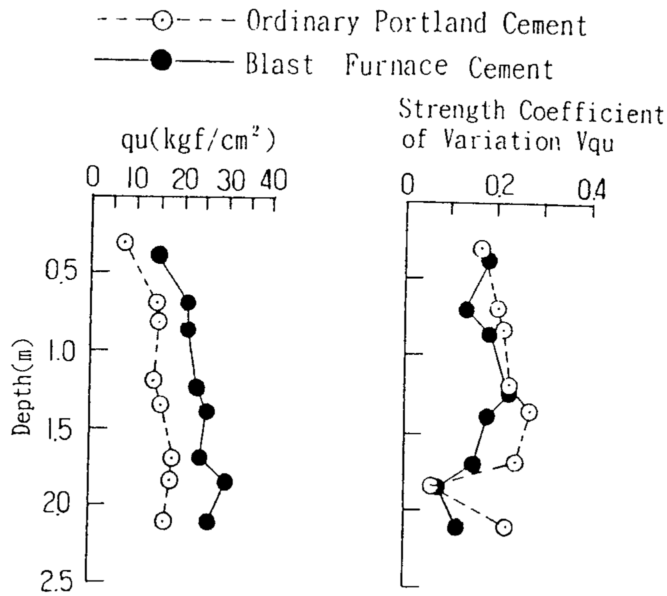


Figure 287. Variation in strength with type of cement (Yoshizawa et al., 1996).

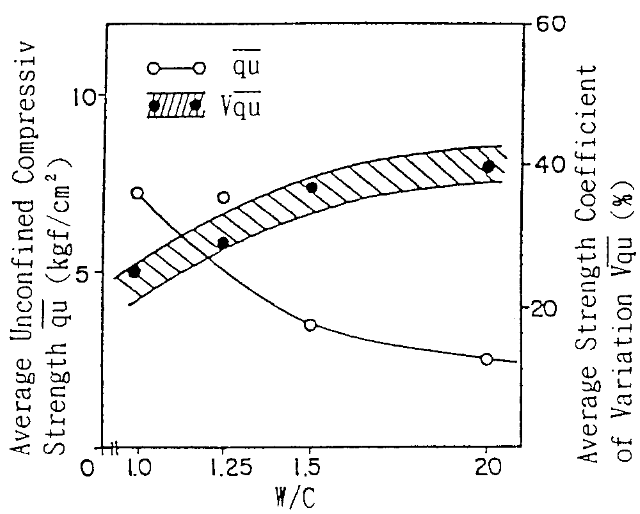


Figure 288. Variation of average strength and coefficient of variation of treated soil with water/cement ratio (Yoshizawa et al., 1996).

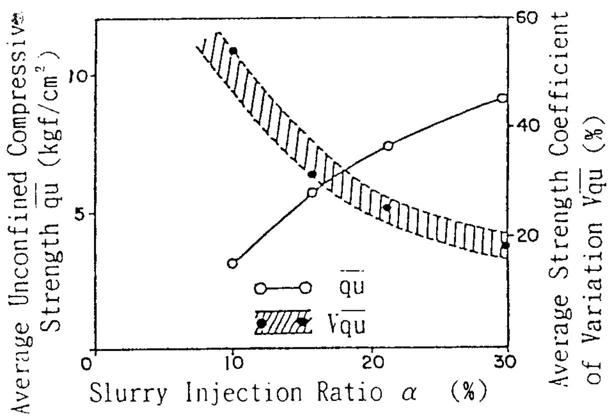
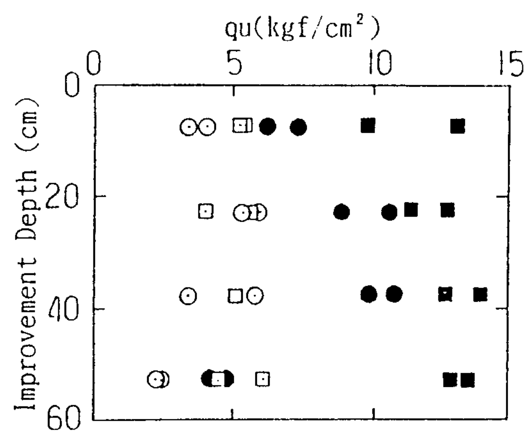


Figure 289. Variation of average strength and coefficient of variation of treated soil with α (Yoshizawa et al., 1996).



Number of Mixer Shaft	7 day	28day
1–Shaft	⊙	●
4–Shaft	⊡	■

Figure 290. Comparison of strength achieved using 1-shaft and 4-shaft models

(Yoshizawa et al., 1996).

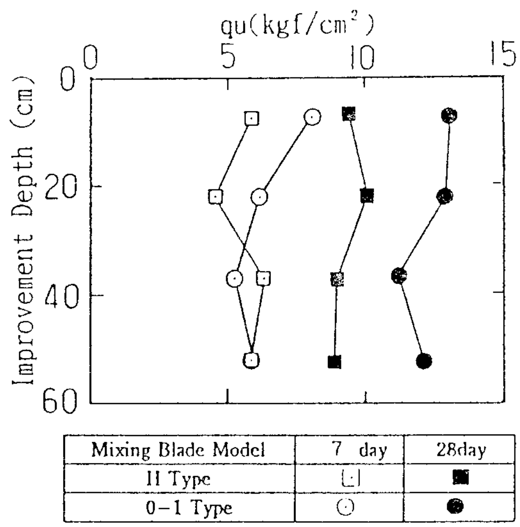


Figure 291. Variations in strength of treated soil with blade configuration (Yoshizawa et al., 1996).

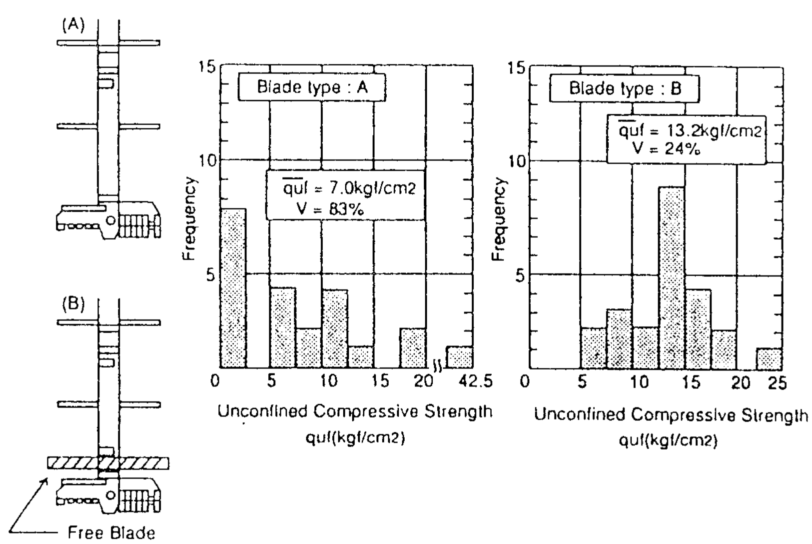


Figure 292. Effect of soil anti-rotation vanes (Yoshizawa et al., 1996).

3. Rotational speed. Figure 293 shows that higher strength results from faster rotation.
4. Binder injection method. Although better mixing performance is obtained when injecting during penetration, depth and soil type may prevent this from being practically possible. Also, slurry may be more easily lost during injection than during penetration, creating a reduction in strength (Figure 294) as a result of the lower actual cement factor in situ.
5. Penetration/withdrawal velocity. Figure 295 shows increased strength and smaller variation with reduction in penetration rate (withdrawal rate constant at 1 m/min).
6. Effects of time on overlap performance. Up to 1 hour there is no problem, but at 24 hours the strength of the overlap drops, and at 48 hours it is 60% that of the main body.
7. Degree of mixing indicator. Expressed as the number of blade rotations over 1 m of penetration or withdrawal, following binder injection. For any given site and method, a certain "mixing indicator" value can be established, above which efficient mixing occurs.

Regarding the future, the authors looked at three categories of potentially fruitful lines of research:

1. Mixing Techniques: As shown in Table 77, research continues, especially to improve quality in "sticky clays or humic soils."
2. Monitoring and Quality Control Methods: Table 78.
3. New Applications: Table 79 is particularly applicable for liquefaction, industrial wastes, and reduced spoil applications.

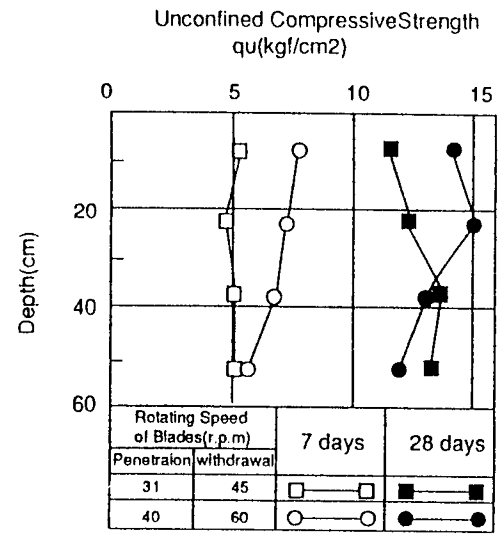
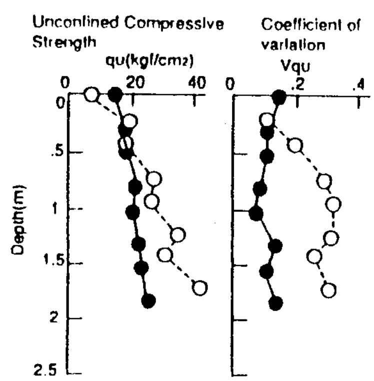


Figure 293. Variation of strength of treated soil with rotational speed (Yoshizawa et al., 1996).



type / Item	Injection during penetrating	Injection during Withdrawing
	---●---	---○---
qu (kgf/cm2)	20.1 (78)	25.7 (78)
$\overline{V}$qu (each layer)	0.103 (9~10)	0.222 (9~10)
Vqu (total layer)	0.200 (78)	0.486 (78)

※ ( ) means numbers of Specimens

Figure 294. Comparison of improvement strength achieved using penetration and withdrawal injection (Yoshizawa et al., 1996).

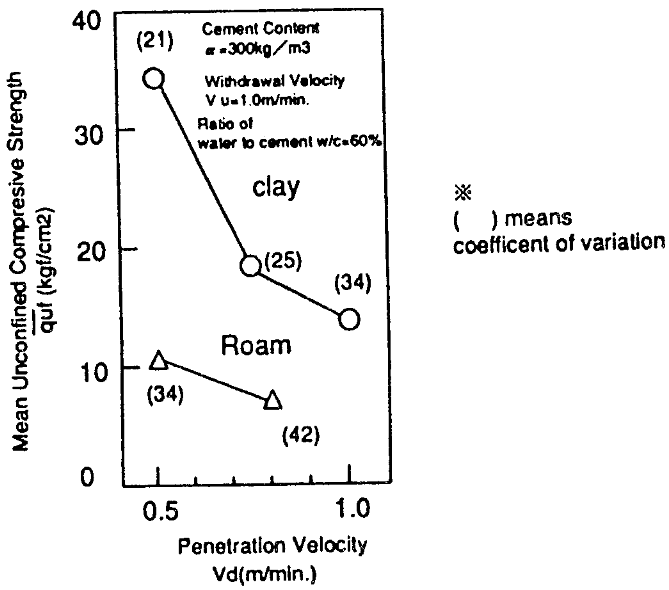


Figure 295. Effect of penetration velocity on treatment effectiveness (Yoshizawa et al., 1996).

Table 77. Diversity of mixing methods (Yoshizawa, et al., 1996)

Objective	Measures
Improvement in uniformity and agitation	(a) Wings to prevent retained lumps (b) Wings rotating in reverse direction. (c) Mixing within the ground (d) Rotating and revolution wings
Improvement of mixing efficiency	(a) Agitation in vertical direction (b) Installation of rectangular box (c) Use of high pressure water jet (d) Use of high pressure air jet
Reduction of the residual soil	(a) Expand mechanically at a depth (b) Use of spiral auger (c) Boring with the use of casing
High level of quality control	(a) Mixing at a plant on the ground and them filling back

Table 78. Improvement monitoring (Yoshizawa et al., 1996).

Objective	Measures
Process Monitoring	(a) Automation and robotics (b) Automatic positioning system (c) Global positioning system
System Monitoring	(a) Rotary sounding method (b) Use of neutron sensor (c) Use of parameters such as work efficiency in controlled installation

Table 79. Development of new applications (Yoshizawa et al., 1996).

Objective	Measures
Response to new needs	(a) Installation to greater depth (b) Reduction of environmental effects (c) Countermeasure against liquefaction (d) Application to basement construction (e) Down-sizing construction equipment (f) Barrier against industrial waste flow
New applications	(a) Application as earth anchors (b) Application to underground structures

They closed by noting that the original market drivers of production speed and lowest possible cost have been supplemented by the need for reliability, value, and quality. "A system of real-time monitoring that would enable prediction of final treated soil quality is desirable."

7.5 Overview by CDM Association (1996)

They summarized the main factors influencing strength of treated marine soil (Figure 296). The two chemical mechanisms are defined as:

1. Hydration of cement and water.
2. Pozzolanic reaction between clays, and calcium hydroxide produced through cement hydration reaction.

The factors influencing the properties of treated soil were summarized as follows:

a. Water content: Generally lower in treated soil, although the effect is smaller for relatively dry soils. Cement factor will also influence (Figure 297).

b. Specific Density: Typically (Figure 298) a little higher for treated soil but is often regarded for design purposes as the same.

c. Permeability: Reduced considerably (Figure 299), and reduces with increasing cement factor.

d. U.C.S.: Depends on soil, cement factor, and cement type, inter al., as shown in Figures 300 and 301.

e. Elastic modulus: For soils with less than 10 to 15% sand content, the approximate relationship is E_{50} = (400 to 600) x q_u although the data of Figure 302 show a wider range, with a wide range of soils (350 to 1000 times).

f. Relationship of U.C.S. field and laboratory: Figure 303 shows that field strengths may often be higher, but this does depend on a variety of factors.

g. Undrained shear strength: Figure 304 (laboratory tests) shows an initial ratio of 50% of U.C.S., decreasing with increasing U.C.S. to about 30%.

h. Tensile strength: Figure 305 shows relationship for both direct tensile tests (28 to 8% with increasing U.C.S.) and splitting tests (13 to 8% with increasing U.C.S.).

i. Rate of gain of strength: Figures 306 and 307 show data on laboratory tests for different sites. Figure 307 shows a strength relationship of 28-day strength = 1.44 times 7-day strength.

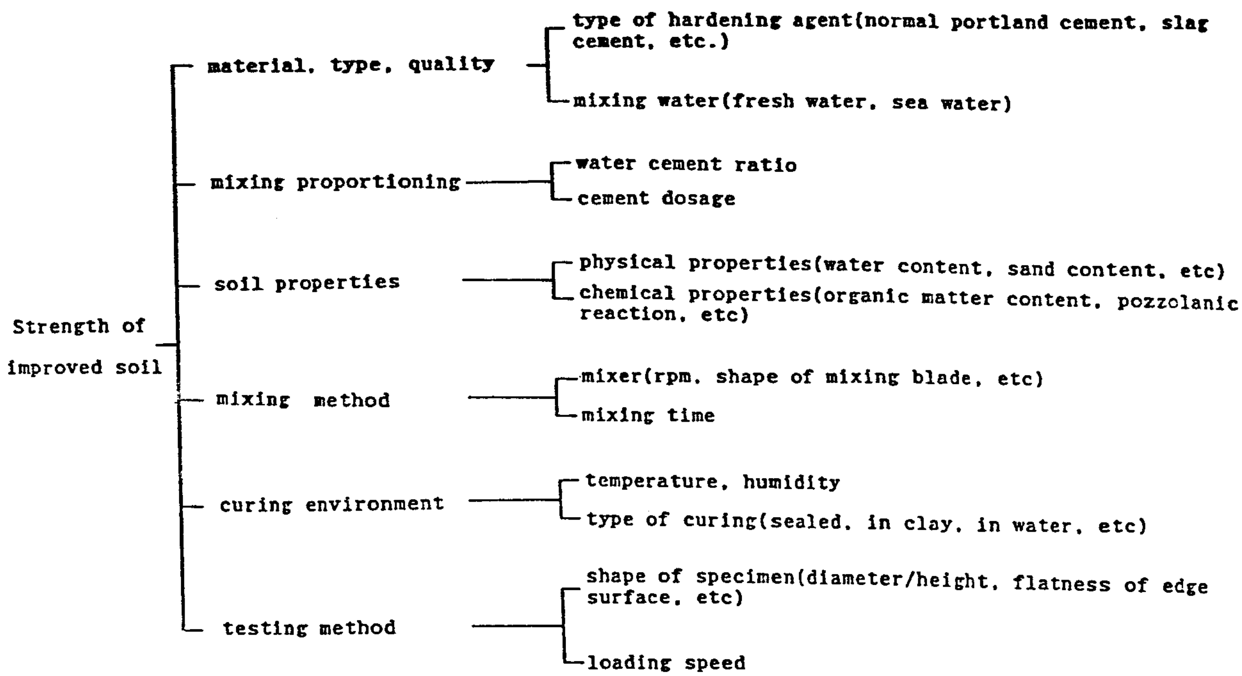


Figure 296. Main factors influencing the strength of improved soil (CDM Association, 1996).

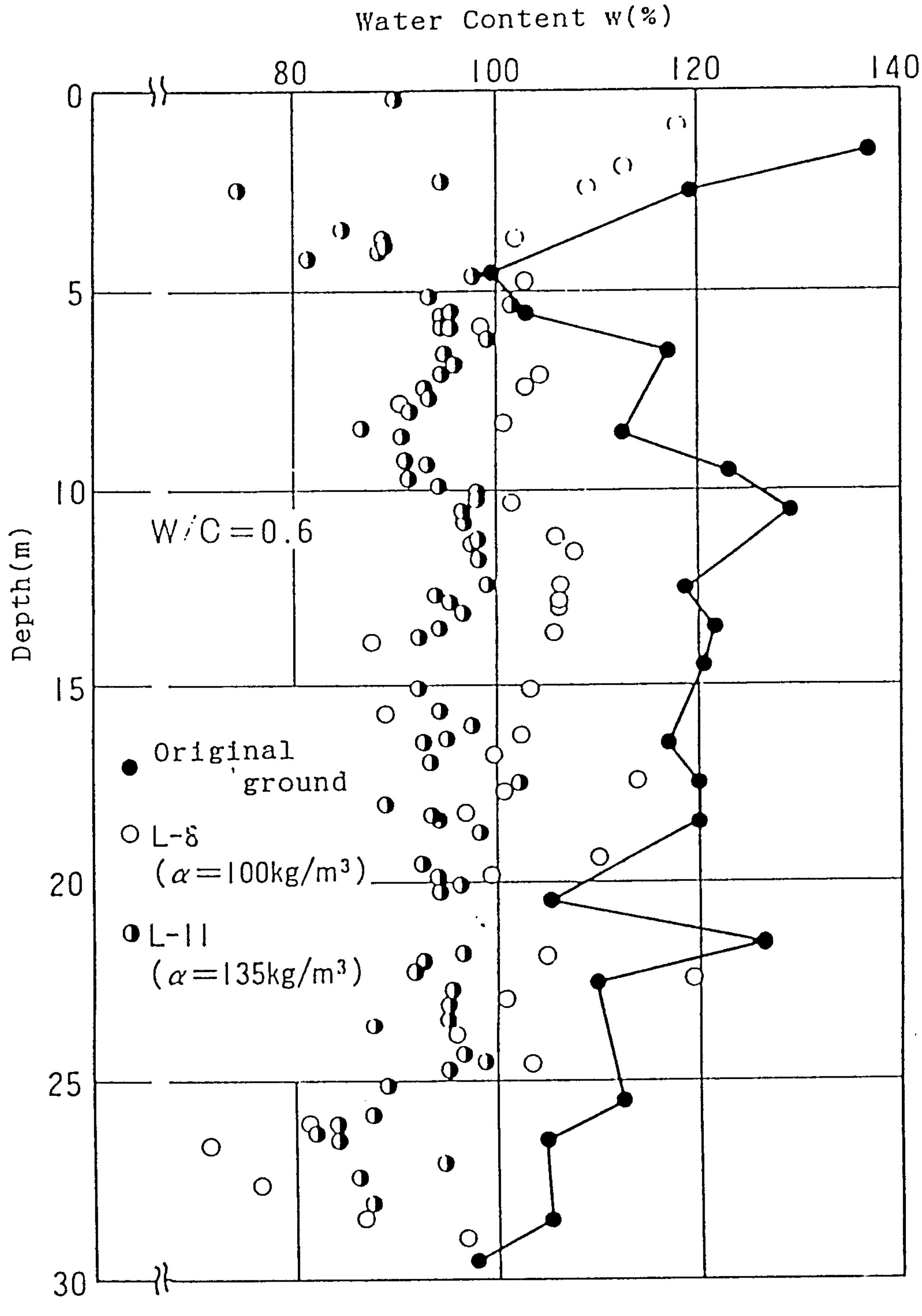


Figure 297. An example of distribution of water content of in situ improved soil (Tokyo Port) (CDM Association, 1996).

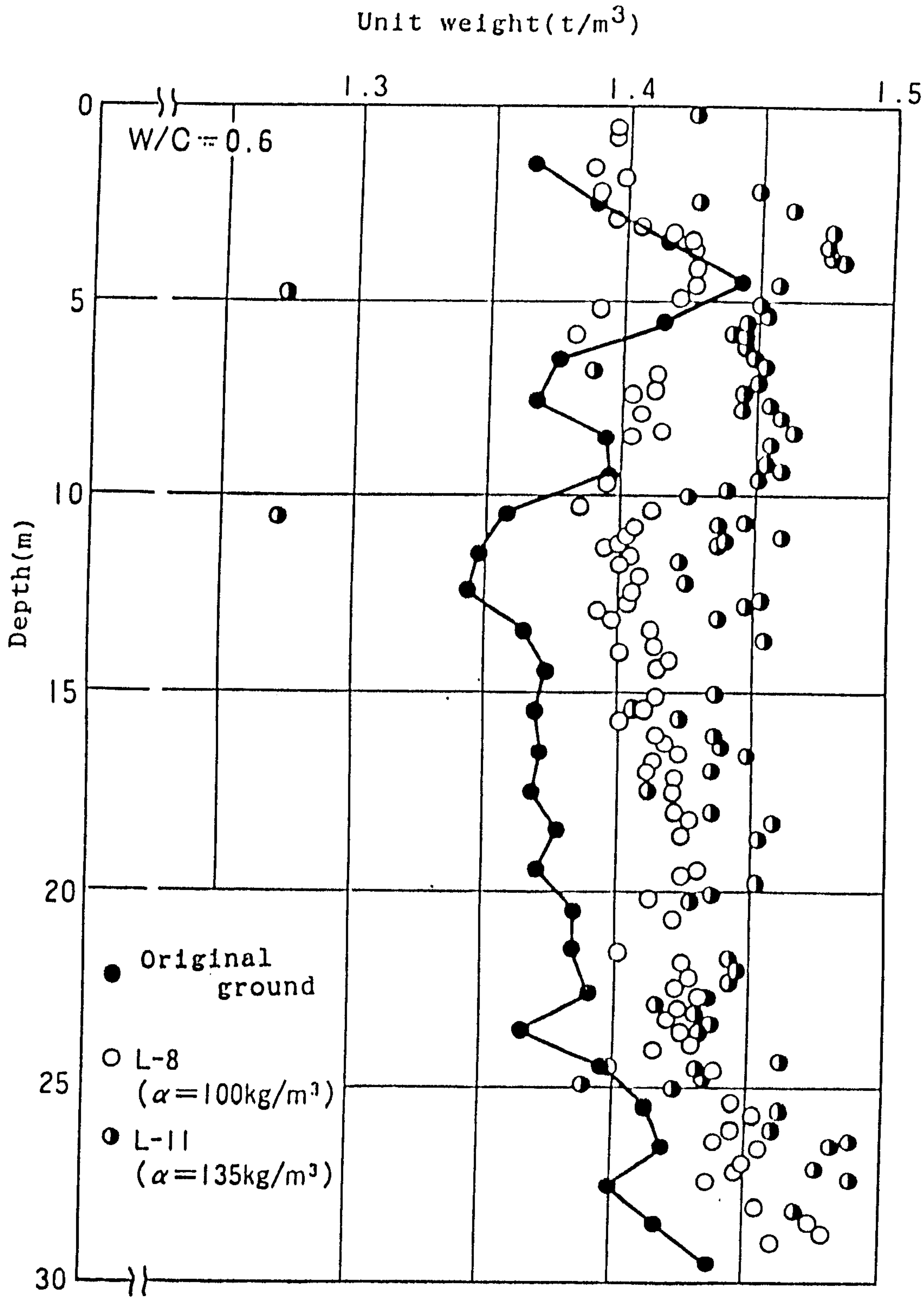


Figure 298. An example of unit weight distribution of in situ improved soil (Tokyo Port) (CDM Association, 1996).

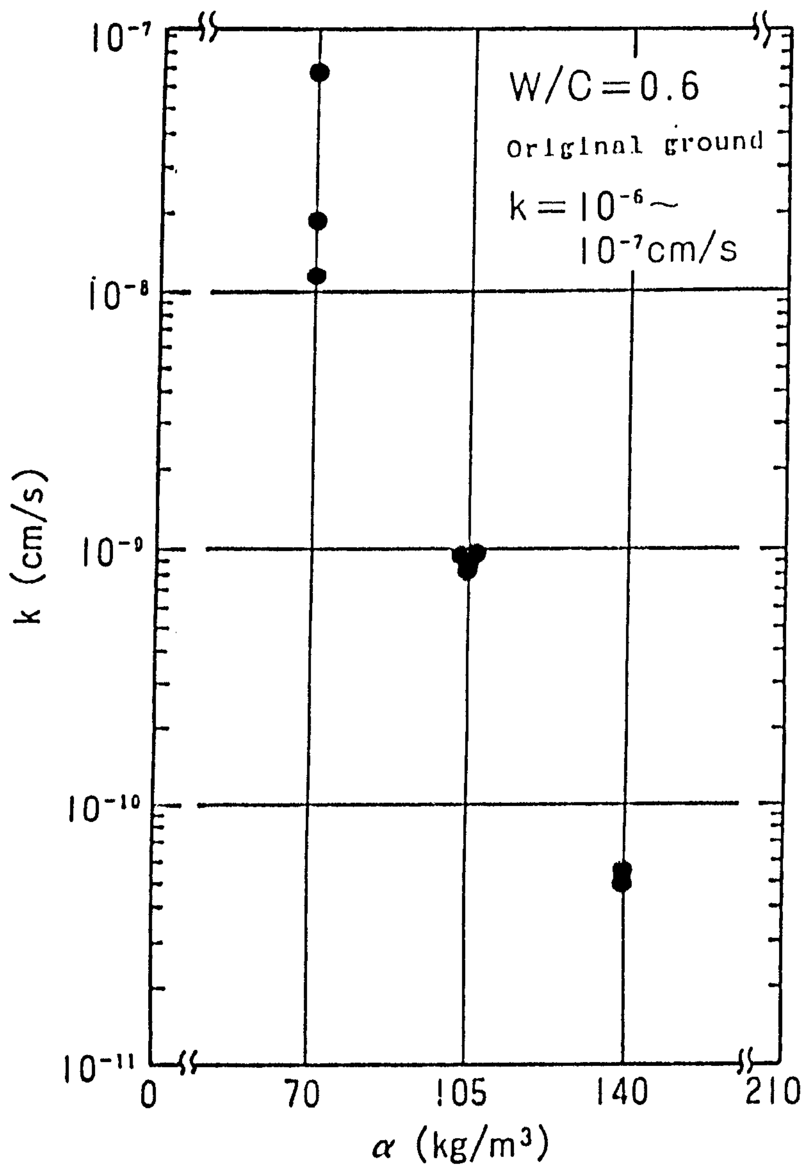
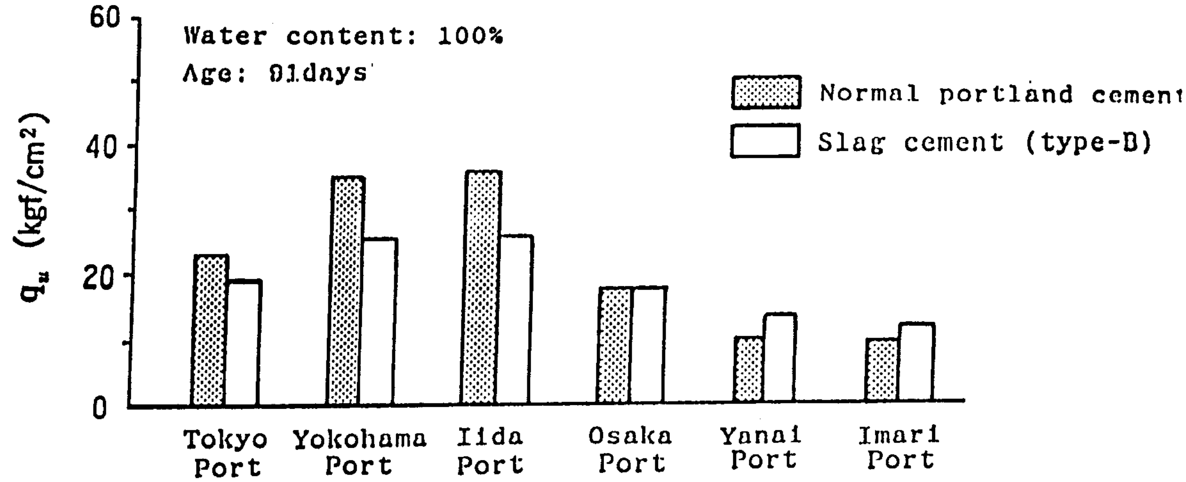
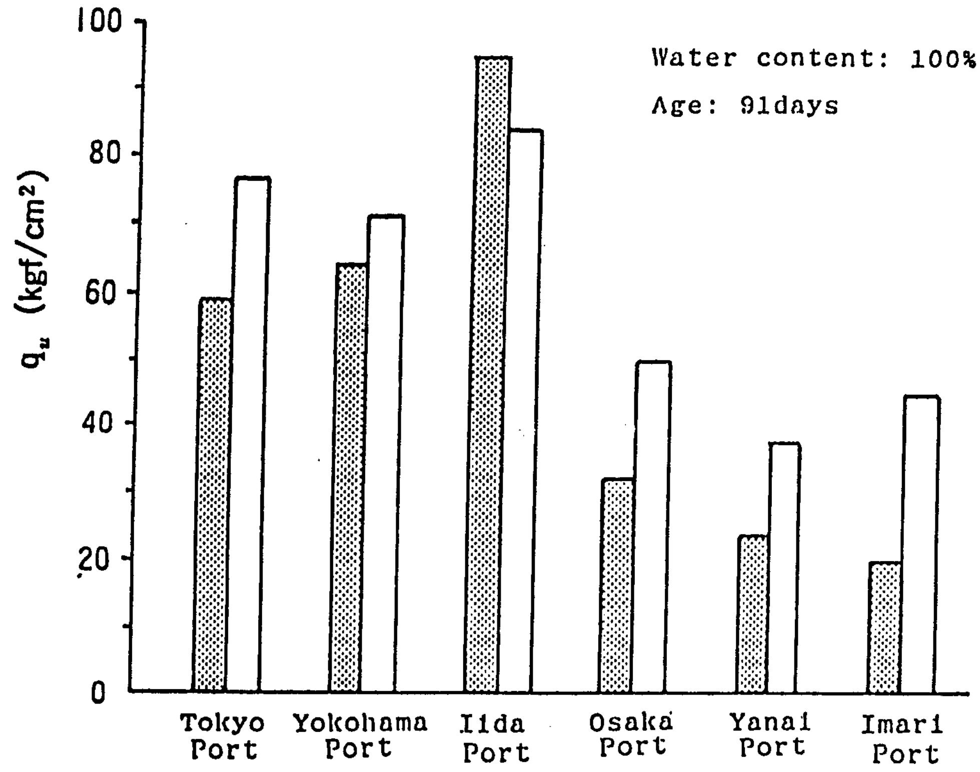


Figure 299. Relationship between coefficient of permeability and cement content for laboratory improved soils (clay in Tokyo Port) (CDM Association, 1996).

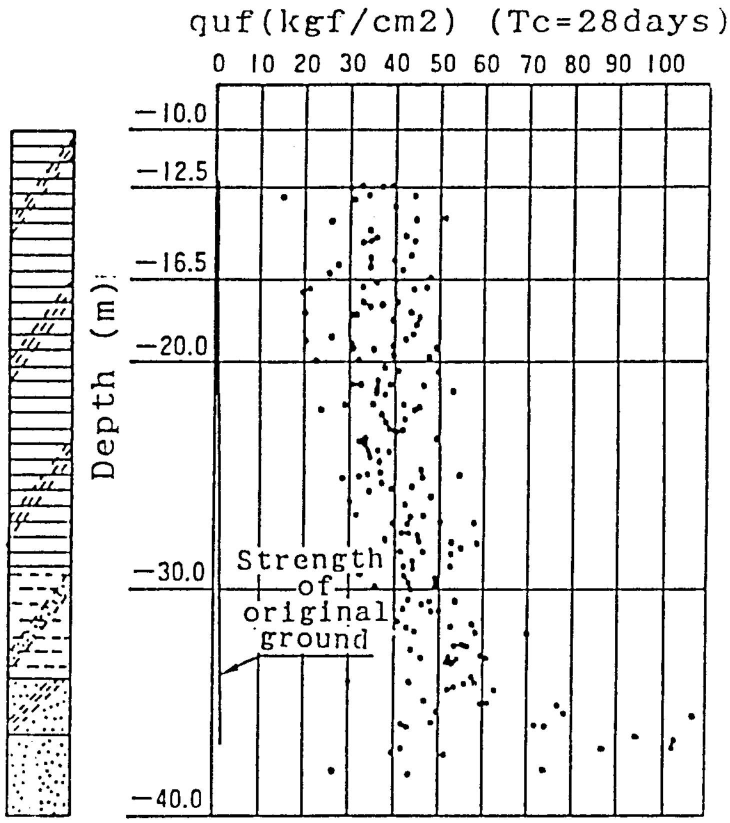


(a) for cement content of 100 kg/m3



(b) for cement content of 200 kg/m3

Figure 300. Comparison of improving effects for different types of soils (CDM Association, 1996).



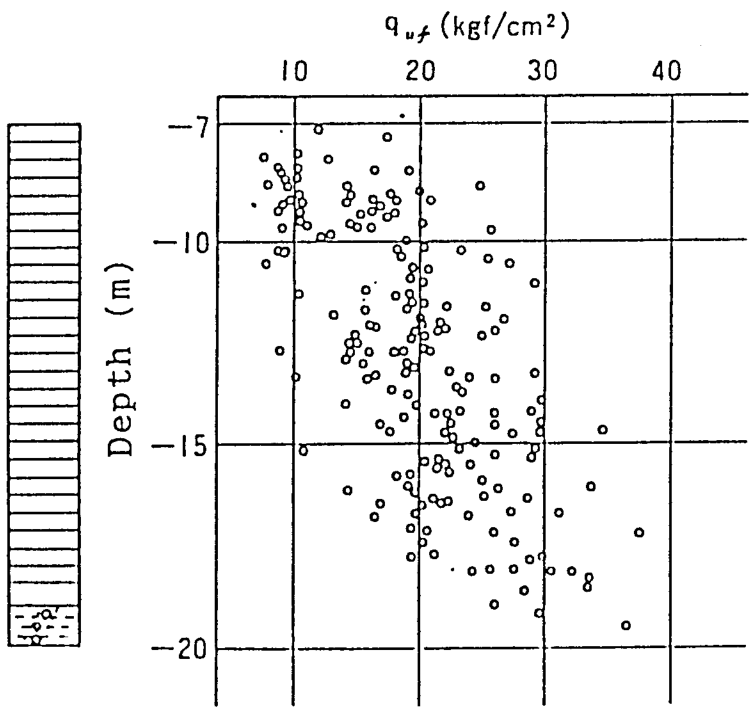


Figure 301. Distribution of unconfined compressive strength of in situ improved soil (CDM Association, 1996).

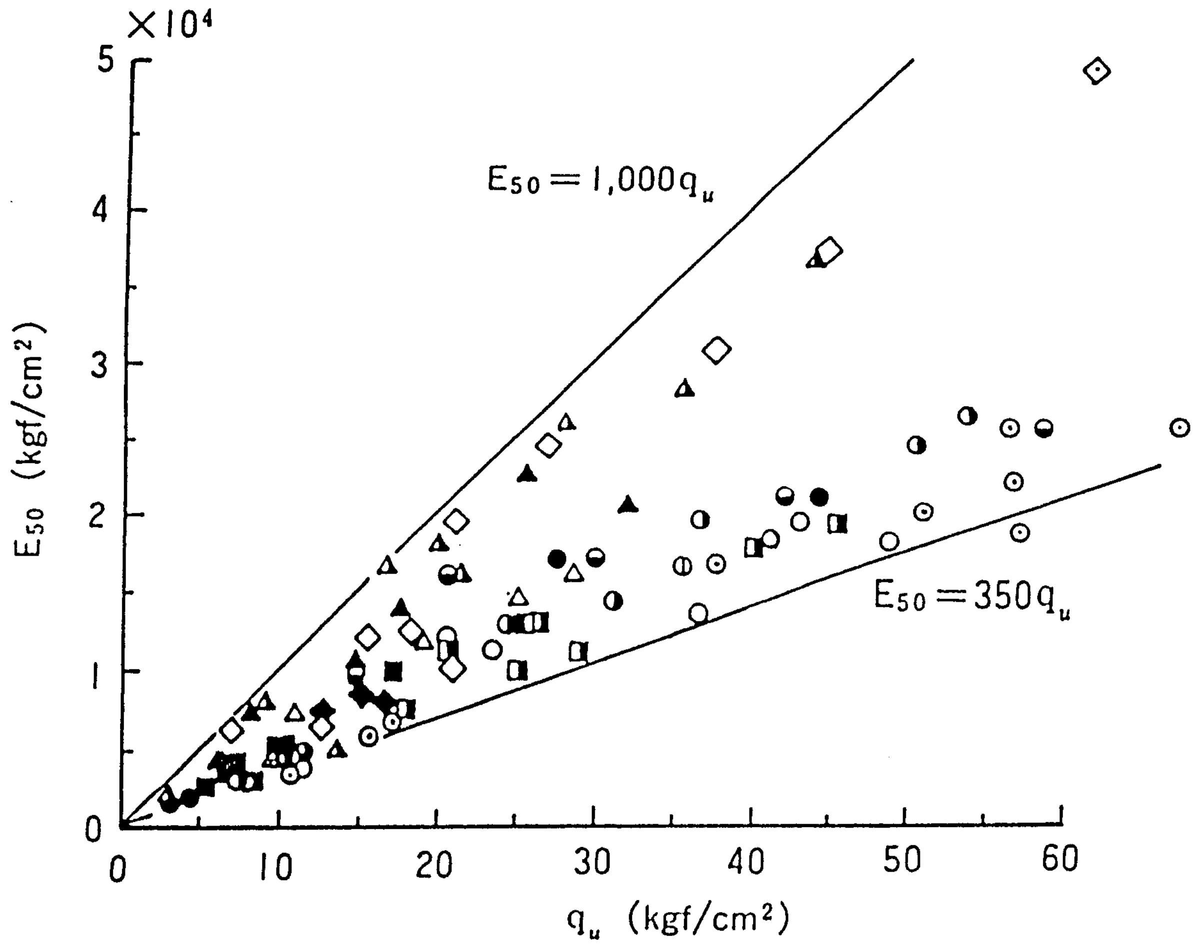


Figure 302. Relationship between unconfined compressive strength, q_u, and modulus of linear deformation, E_{50}, for laboratory improved soil (CDM Association, 1996).

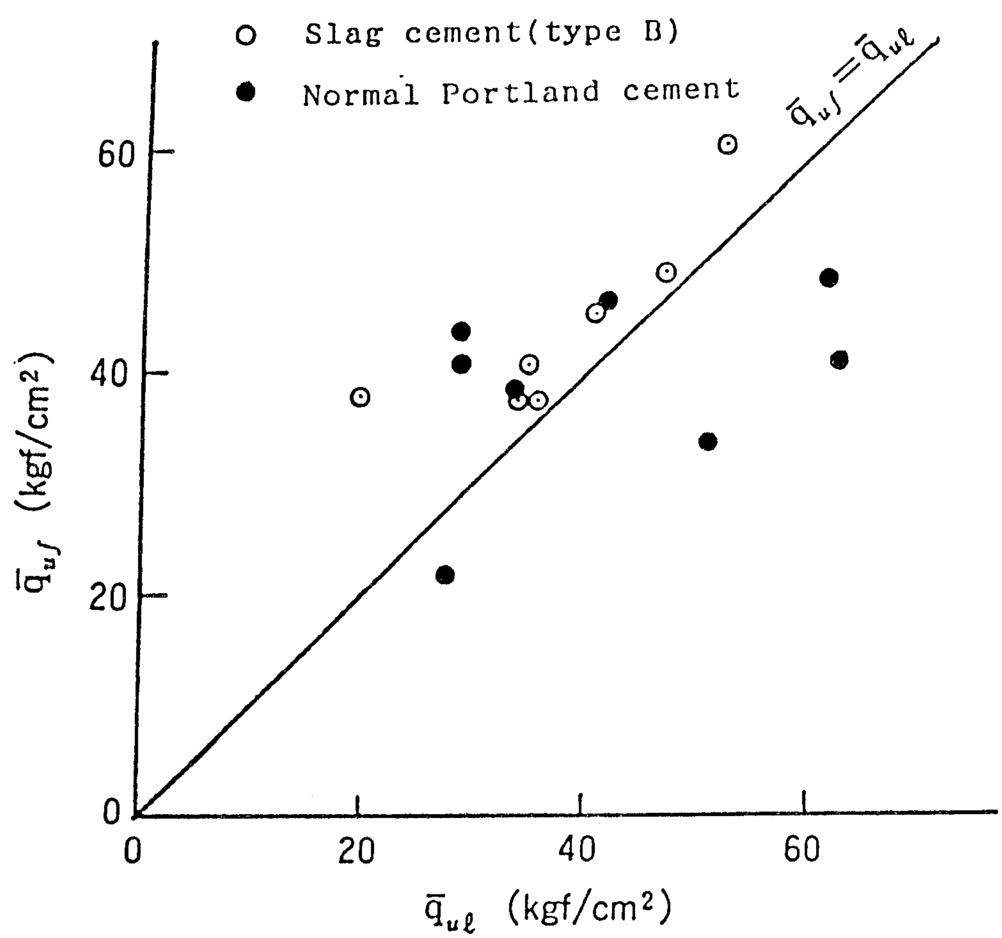


Figure 303. Relationship between unconfined compressive strength of in situ improved soil, q_{uf}, and that of laboratory improved soil, q_{ul} (CDM Association, 1996).

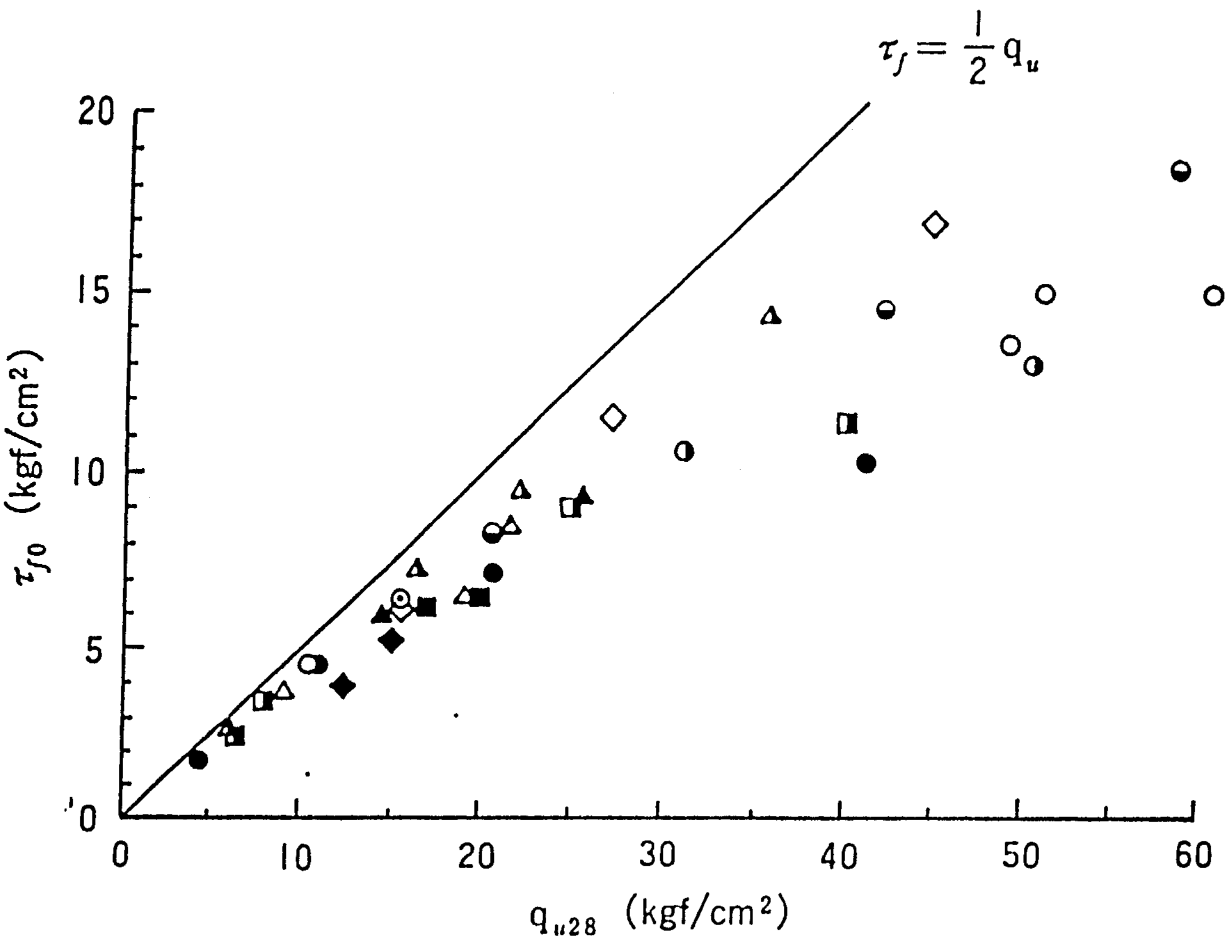


Figure 304. Relationship between shear strength, τ_{of},and unconfined compressive strength, q_{u28}, for laboratory improved soils (CDM Association, 1996).

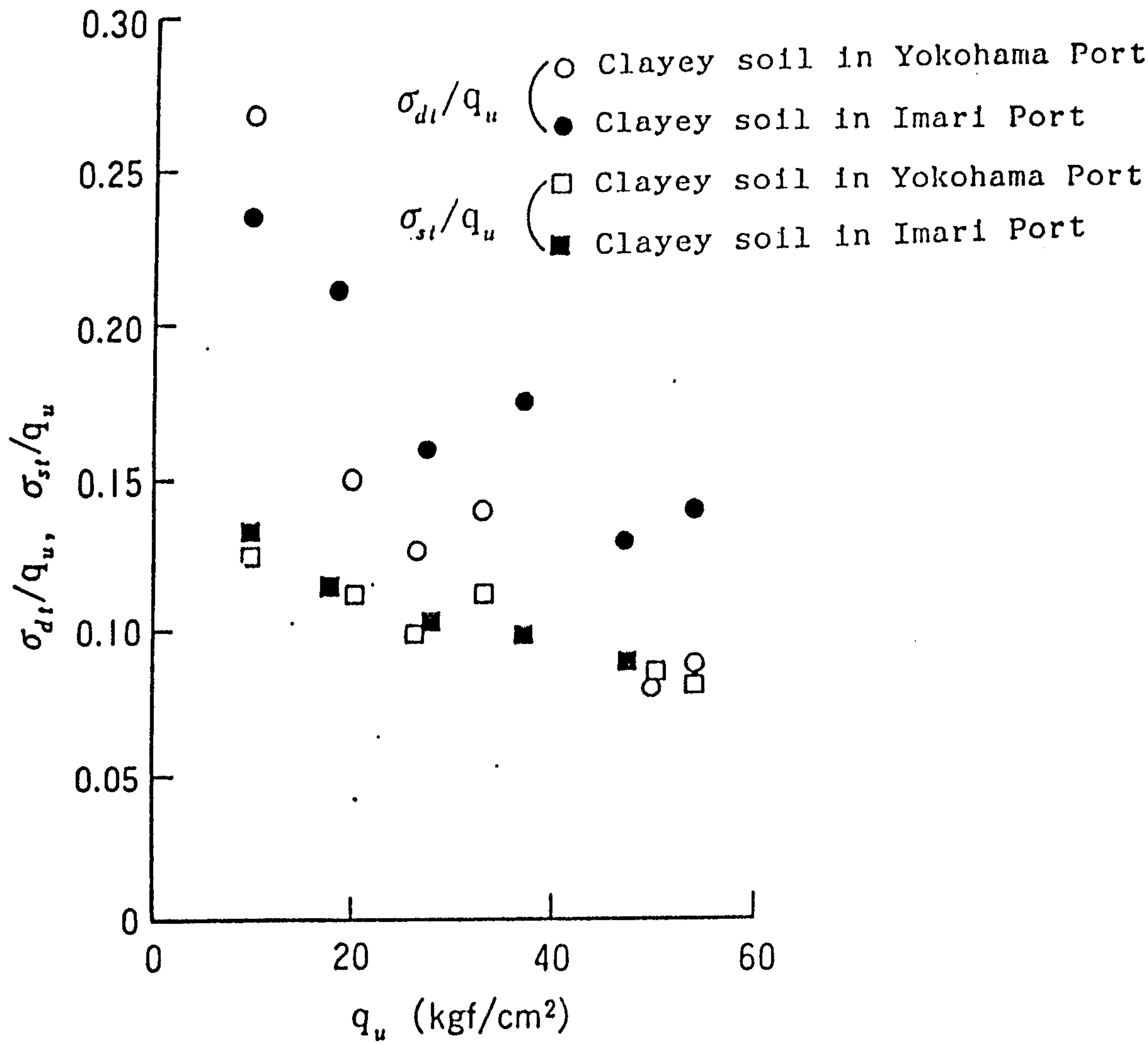


Figure 305. Relationship between unconfined compressive strength and tensile strength (CDM Association, 1996).

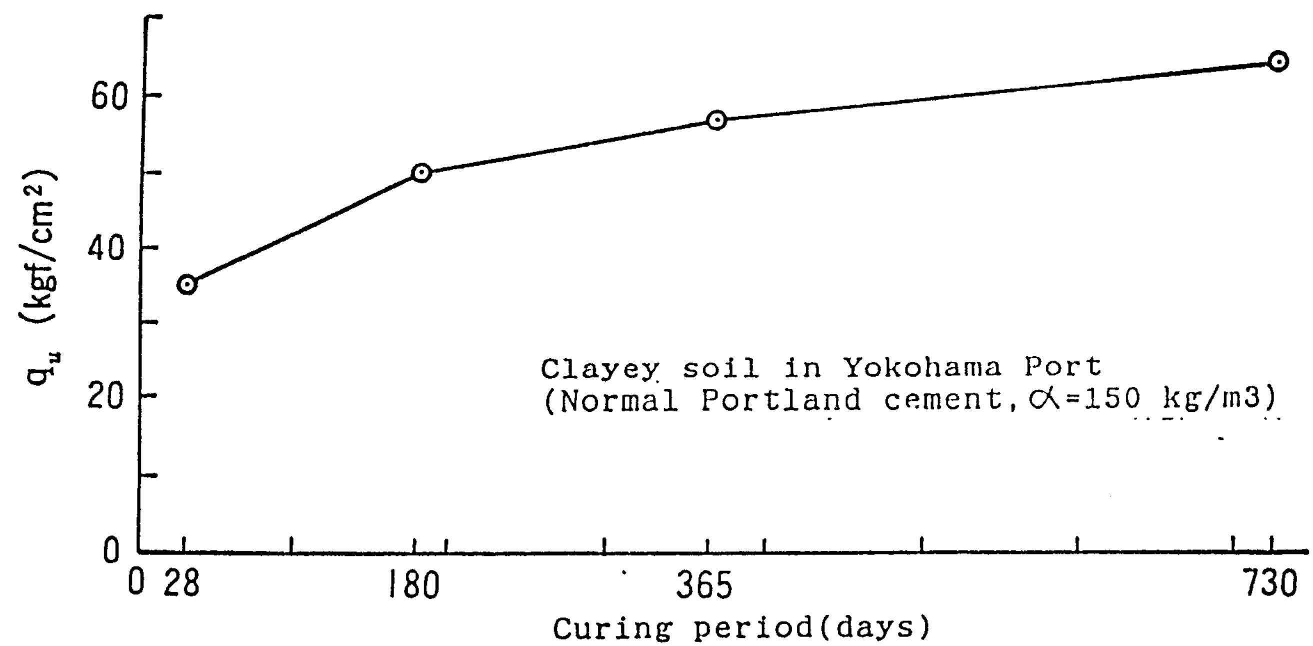


Figure 306. Relationship between unconfined compressive strength, q_u, and curing period for laboratory improved soils (sealed curing) (CDM Association, 1996).

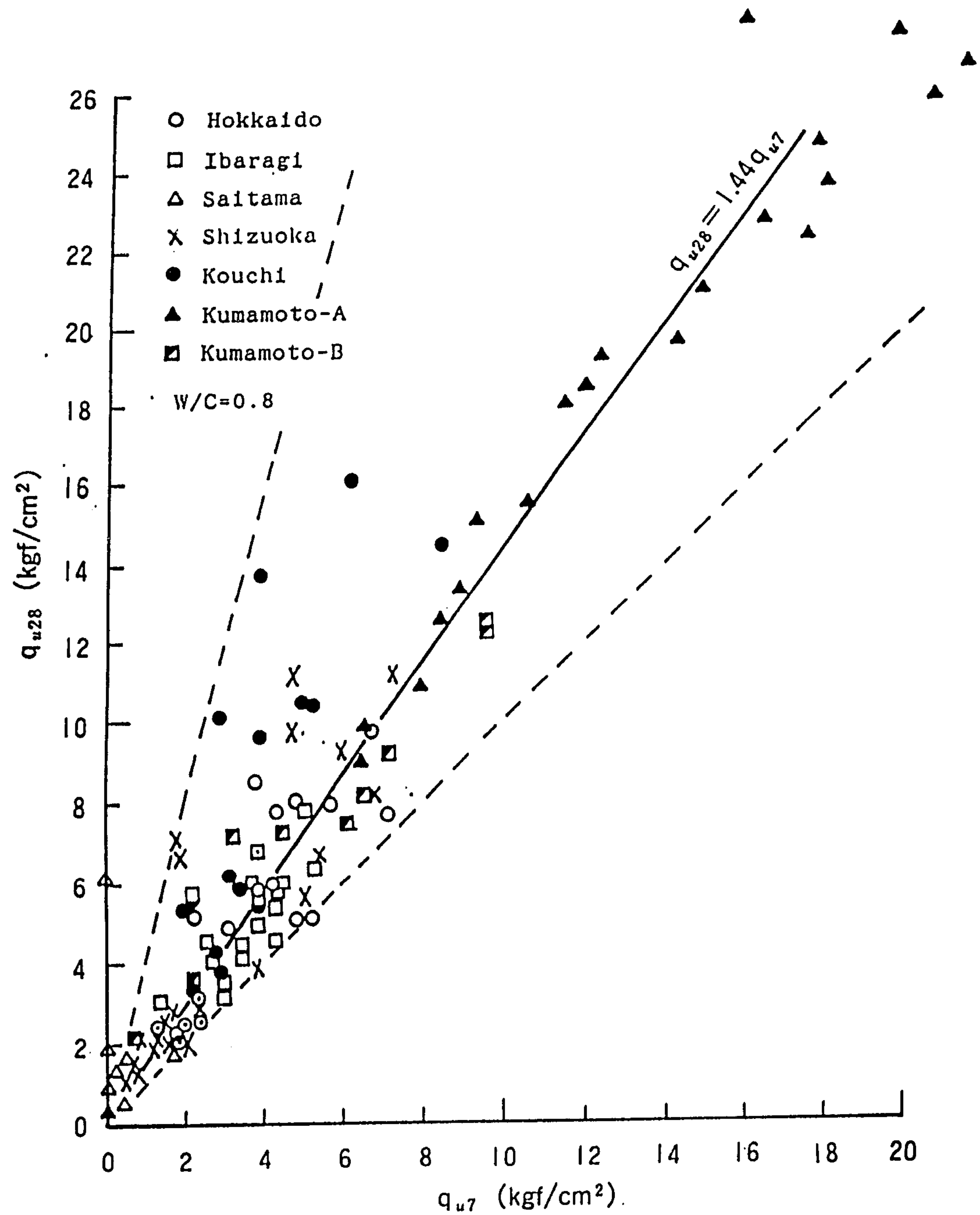


Figure 307. Relationship between q_{u7} and q_{u28} for laboratory improved soil by special cement (organic soils) (CDM Association, 1996).

7.6 Overview by Okumura (1996)

Okumura summarized that the characteristics of treated soil can be said to be similar to those of overconsolidated, cemented, or aged soil. Figure 308 shows an example of triaxial test results (Terashi et al., 1983). It may be said that the strength of treated soil is constant under the confining pressure up to the yield stress and proportional to the consolidation pressure over the yield stress. Tensile strength of treated soil is said to be 10 to 20% of compressive strength (Terashi et al., 1983).

The data shown in Figure 309 suggest that the ratio of E_{50}:U.C.S. changes from about 140 (U.C.S. < 1.5 MPa) to about 800 (U.C.S. of 1.5 to 3.5 MPa).

Consolidation yield stress or pre-consolidation pressure of treated soil is about 1.3 times the unconfined compressive strength (Okumura and Terashi, 1975). Examples of coefficient of volume compressibility are shown in normalized form in Figure 310, where m_v and m_{vr} are the coefficient of volume compressibility of treated and untreated remolded soil, respectively, and p_λ is the consolidation yield stress. It may be said that the compressibility of treated soil is very small under pressures below the consolidation yield stress. Coefficient of consolidation of treated soil is about 10% that of remolded soil below the yield stress, but decreases rapidly with consolidation pressure over the yield stress as shown in Figure 311 (Okumura and Terashi, 1975). Coefficient of permeability of treated soil is lower than that of untreated soil, and hence it is not suitable to constitute a vertical drain (Okumura, 1977; Terashi and Tanaka, 1983).

Deterioration under the conditions of exposure to seawater increases with logarithm of elapsed time (Terashi et al., 1983).

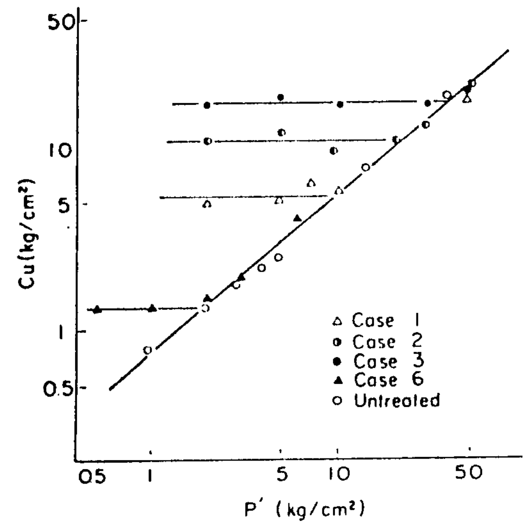


Figure 308. Consolidated undrained triaxial test results (Okumura, 1996).

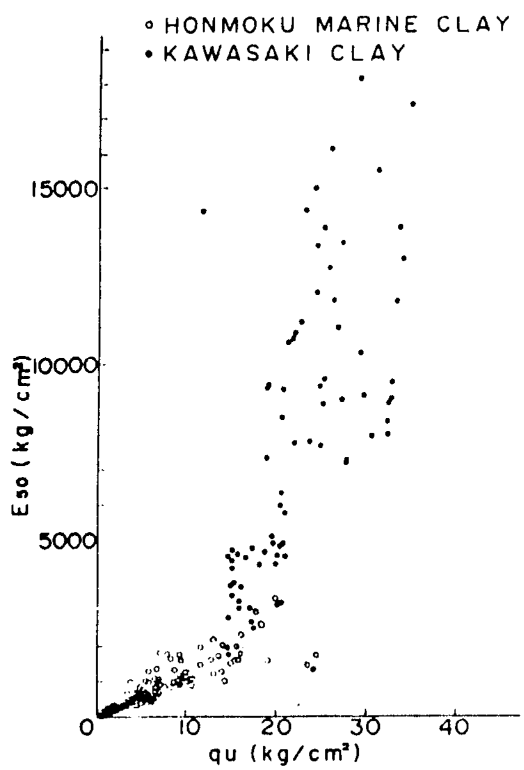


Figure 309. Relationship between unconfined compressive strength and modulus of deformation (Okumura, 1996).

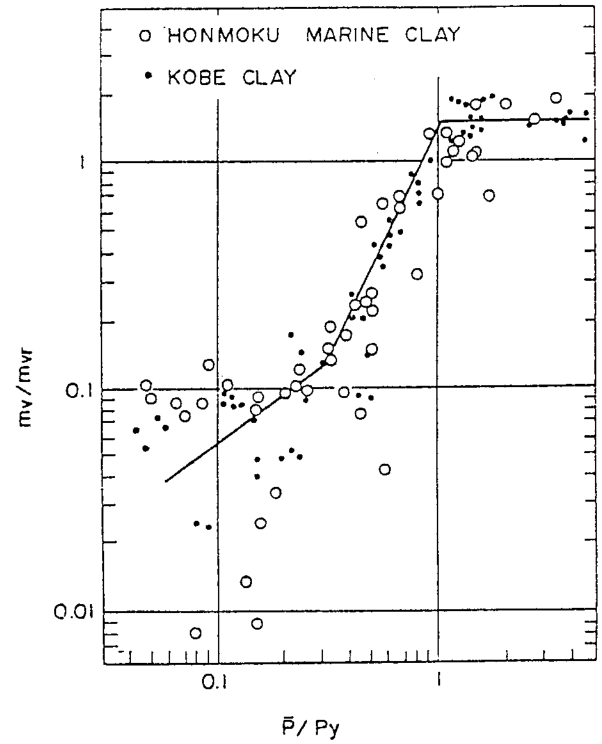


Figure 310. Coefficient of volume compressibility of lime treated soils (Okumura, 1996).

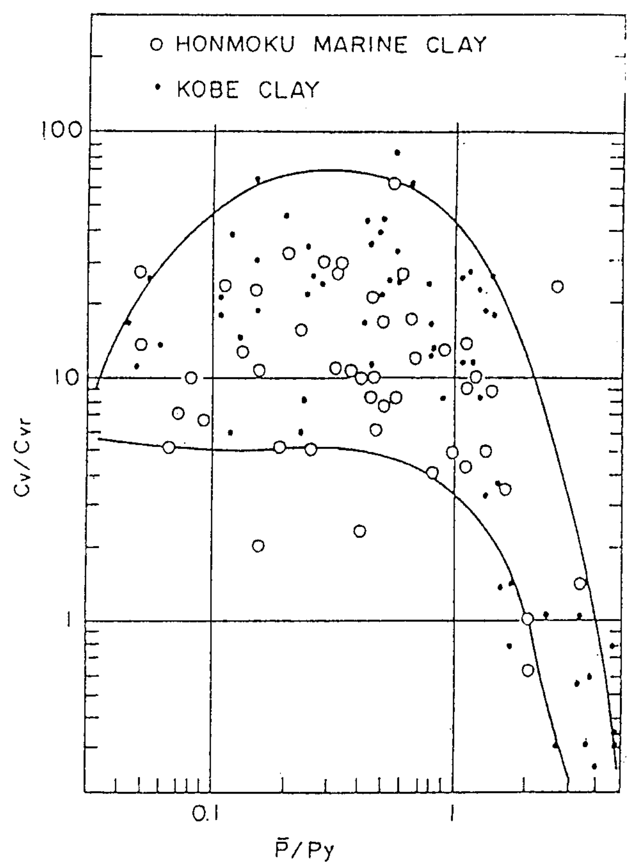


Figure 311. Coefficient of consolidation of lime treated soils (Okumura, 1996).

7.7 General Overview

The data contained in the four committee reports reviewed in this chapter arguably represent the most authoritative collection of data on the subject in the English language. Hundreds of papers have been reviewed, and test results from scores of sites have been collated and analyzed. These reviews often include reference to papers discussed separately, elsewhere in this report, and so further serve to reinforce key observations and conclusions previously drawn. Few of the points in the following summary are therefore introduced for the first time, but they do reinforce the details of Chapters 3, 4, and 5 in particular.

1. Report on "Deformation and Strength Properties of DM Cement-Treated Soils."
   Recommendations are made for standardizing and optimizing laboratory testing methods.

2. Report on "Factors Influencing the Strength of Improved Soil."
   a) Four subcategories of binders are used:
      - Lime (quick and slaked).
      - Standard cements (ordinary and blast furnace).
      - Special cements (for organics (high alumina) or low strength (gypsum or lime)).
      - Industrial byproducts (e.g., slags and ash).

   b) Characteristics of treated soil (mainly unconfined compressive strength)
      U.C.S. is controlled by:
      - Cement factor and soil grain size.
      - Moisture content (up to 200%: thereafter minimal increase).
      - Organic content (>15% ignition loss, or 0.9% humus, or pH < 4).
      - Curing conditions (temperature, scale, water).

3. <u>Report on "Evaluation of the Strength of Soils Improved by DMM."</u>

It notes that evaluation by crushing core samples yields variable results due to the nature of the sampling. A 150-mm-diameter core is recommended, as is triaxial as opposed to uniaxial testing. Developments continue with coring equipment. Various in situ categorized as follows

a) Sounding tests (SPT, cone (dynamic and static), drill parameters).
b) Borehole testing (PS, electrical, density, logging).
c) Load testing (lateral, plate load, pile).
d) NDT (integrity, elastic wave).
e) Other (penetration).

4. <u>Report on "Factors Affecting the Quality of Treated Soil During Execution of DMM."</u>

Two main groups were considered: "insertion conditions," and "mixing conditions."

a) Insertion conditions

The committee confirmed that the following controls are exercised:

- Type of cement (blast furnace usually superior).
- w/c ratio (usually 1.0).
- Volume ratio (usually 20 to 30%).

b) Mixing conditions

The Japanese have conducted quite fundamental research into the mixing mechanisms, and have quantified the impact of:

- Number of mixing shafts.
- Orientation and geometry of blades.
- Rotational speed.
- Binder injection method.
- Penetration/withdrawal rates.
- Overlap timing.
- "Degree of mixing" indicator.

In summary, it is clear that the more intense and efficient the blending operation, the higher the resultant treated soil strengths. This is logical, and largely relates to the energy per unit soil volume imparted by the process.

Future research topics will focus on mixing techniques (especially in "sticky clays or humic soils"), monitoring, QC methods, and new applications.

5. Overview by CDM Association and Okumura

They summarized data on the roles of soil characteristics and binder types and concentrations on the full range of treated soil parameters.

8. FINAL REMARKS

From the mid 1990s onwards there has been a tremendous increase in the volume of technical papers dealing with the testing and properties of treated soil *published in the English language*. The results of fundamental studies conducted in Japan, the Nordic countries, the United States, and England prove an immensely informative and insightful pool of knowledge. Given the intensity and energy with which the research continues to be conducted in Japan and Scandinavia in particular, it may only be expected that the pool will continue to grow.

This is clearly beneficial to the industry in general, although it does pose simple logistical problems to individual practitioners: it is virtually impossible to keep abreast of all these new – let alone recent – developments in practice. This problem is compounded by the fact that the pool is not always clear in its entirety. The properties of treated fine-grained soils are dependent on a large number of interacting factors both natural and induced; the complexities, for example, of the hydration and pozzolanic reactions, especially in highly organic cohesive soils, are still being researched. Due to the subtle influences of various factors, and even differences in sample preparation and testing methods, data are produced that may seem widely different from those of another source. Occasionally, these may even be contradictory with respect to the conclusions drawn between the various groups of researchers. The reader must therefore be very careful when drawing from any given source – the details of soil, treatment, and testing must be carefully evaluated to ensure that the conclusions drawn by the author in question are truly apposite to the reader's particular goals and needs.

Amid all the details contained in this report, there are a number of very simple statements that should prove to be both a guide and encouragement to practitioners.

1. There are a large number of proprietary DMM techniques, but only four basic mixing methodologies. The properties of the treated soil are highly sensitive to the details of the particular method used (e.g., mixing method and energy, binder types, and quantity).

2. The properties of the native soil are equally powerful controls over the properties of the treated soil. In particular the moisture content, fines content and composition, organic content and composition, and in situ stress conditions are especially important to evaluate.
3. Levels of quality control and assurance are extremely high in the various techniques originating in Japan and Europe, and are improving steadily in the processes born in the United States. Real-time display of relevant construction parameters permits logical changes to be made in response to any variations in ground conditions, and otherwise confirms that the work is being executed in conformance to pre-established criteria.
4. There exists a vast store of data obtained on properties of soils treated (and tested) in both the laboratory and the field. With appropriate engineering judgment, such data can be used as a starting point when planning a new project. However, the actual performance achieved (or achievable) on any given site must be verified *on that site*, as a true reflection of the interaction between local ground conditions and the construction materials and methodologies adopted. Clearly the extent to which this can be accomplished will reflect the nature of each specific project.
5. Verification methods in different continents have traditionally followed different paths: coring in Japan, probe testing in the Nordic countries, and wet grab sampling in the United States. Depending on the anticipated properties of the treated soil, however, there is a wide range of techniques, some modified from natural soil testing, others "imported" from geophysical or piling practice, which can be employed in addition or instead. The selection of specific testing methods should follow careful study of their relevance, accuracy, applicability, and cost in particular.
6. Research and development remains ongoing into various aspects of DMM, but of particular relevance are those studies that relate to new binder types, more efficient mixing systems, QA and QC techniques, improved and standardized sampling and testing methods, and the challenges posed by soils with high moisture contents and/or high organic contents. Current studies seem to indicate practical limits on the viability of DMM as an economic ground treatment methodology in such extreme soils: such soils may pose chemical conditions that defeat effective hydration and pozzolanic activity, and physical conditions (e.g., peat) that are not amenable to uniform blending. Extreme care must be exercised under such

conditions to ensure that unrealistic performance goals are not set, which may lead to later technical, financial, and contractual problems.

REFERENCES

Åhnberg, H., G. Holm, L. Holmqvist, and C. Ljungcrantz. (1994). Deep stabilisation of soft soils. Seminar on ground improvement methods, Hong Kong, May 1994, Proceedings, pp. 75-83.

Åhnberg, H., S.E. Johansson, A. Retelius, C. Ljungkrantz, L. Holmqvist, and G. Holm. (1995). "Cement och kalk för djupstabilisering av jord – en kemisk/fysikalisk studie av stabiliseringseffekter, Swedish Geotechnical Institut, Report No. 48.

Åhnberg, H. (1996). "Stress dependent parameters of cement and lime stabilised soils." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 387-392.

Åhnberg, H., and G. Holm. (1999). "Stabilization of some Swedish organic soils with different types of binders." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 101-108.

Al-Tabbaa, A. (1999). "Soil mixing of stratified contaminated sands." Accepted in the Institution of Civil Engineers, Journal of Hazardous Materials.

Al-Tabbaa, A., and C.W. Evans (1999a). "Pilot in situ auger mixing treatment of a contaminated site – Part 3: time-related performance." Accepted in the Proceedings of the Institution of Civil Engineers, Journal of Geotechnical Engineering.

Al-Tabbaa, A., and C.W. Evans (1999b). "Laboratory-scale soil mixing of a contaminated site." *Ground Improvement*, No. 3, pp. 119-134.

Al-Tabbaa, A., and C.W. Evans. (1998). "Pilot in situ auger mixing treatment of a contaminated site – Part 1: treatability study." Proceedings of the Institution of Civil Engineers, Journal of Geotechnical Engineering, No. 131, January, pp. 52-59.

Al-Tabbaa, A., C.W. Evans, and C.J. Wallace. (1998). "Pilot in situ auger mixing treatment of a contaminated site – Part 2: site trial." Proceedings of the Institution of Civil Engineers, Journal of Geotechnical Engineering, No. 131, April, pp. 89-95.

American Society of Civil Engineers. (1997). *Ground Improvement, Reinforcement, and Treatment: Developments 1987-1997*, Proc. of Sessions Sponsored by the Committee on Soil Improvement and Geosynthetics of the Geo-Institute. Logan, UT, July 17-19, Ed. by V.R. Schaefer, Geotechnical Special Publication No. 69, 616 p.

Asano, J., K. Ban, K. Azuma, and K. Takahashi. (1996) "Deep Mixing Method of soil stabilization using coal ash." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo '96, the Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp 393-398.

Ashby and Jones. (1986).

Axelsson, M., and S.E. Rehnman. (1999). "Field methods for quality control at the dry jet mixing method." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 303-310.

Babasaki, R., M. Terashi, T. Suzuki, A. Maekawa, M. Kawamura, E. and Fukazáwa. (1996). "JGS TC Report: Factors Influencing the Strength of Improved Soil." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 913-918.

Bachy. (1996). Promotional information.

Bachy.(1992). Promotional information.

Bachy.(1994). Promotional information.

Bahner, E.W., and A.M. Naguib. (1998). "Design and construction of a deep soil mix retaining wall for the Lake Parkway Freeway Extension." *Soil Improvement for Big Digs*," Geo-Institute of the American Society of Civil Engineers, Geotechnical Special Publication No. 81, pp. 41-58.

Balossi Restelli, A., M. Bertero, and E. Lodigiani (1991). "Deep mixing technology to improve the bearing capacity of a very soft clayey soil under an earth embankment." 4th International Conference on Piles and Deep Foundations, Stresa, Italy, 7-12 April, pp. 171-179.

Barker, P., M. Wyllie, and A. Esnault. (1996). "Investigation, planning and execution of the remediation of Ardeer Landfill, Scotland." International symposium on major recent case histories, Paris, Paper 19, February, 18 p.

Bogue, R.H. (1955). "Calculation of the compounds in portland cement." Ind. Engng. Chem. (Anal.) 1 (4), pp. 192-197.

Braaten, A., R. Aabøe, and F. Oset. (1999). "Development of in-situ control methods for lime/cement columns." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 295-301.

British Standard 12. (1996).

Broms, B.B. (1991). "Stabilization of soil with lime columns." *Foundation Engineering Handbook,* 2nd Ed. Ed. Hsai-Yang Fang. Van Nostrand Reinhold, New York, NY, pp. 833-855.

Broms, B.B., and P. Boman. (1979). "Lime Columns – A New Foundation Method. *Journal of the Geotechnical Engineering Division*. American Society of Civil Engineers, Vol. 105, No. GT4, pp. 539-556.

Broms, B.B. (1999). "Keynote lecture: Design of lime, lime/cement and cement columns." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 125-153.

Burke, G.K. (1997). Personal communication.

Burke, G.K., A. Furth, D. Rhodes. (1996). "Site remediation of hexavalent chromium in a plater's sump: heavy metal chemical fixation - a case history." Proceedings of the 17th Annual AESF/EPA Conference on Pollution Prevention and Control, Orlando, FL, February, 10 p.

Bye, G.C. (1999). *Portland Cement – composition, production, and properties*. Thomas Telford, Ltd., London, 2nd Ed., 225 p.

Catalano, N. (1998). "Trevimix." Presented at the University of Wisconsin Milwaukee Short Course on Deep Mixing Methods, Milwaukee, WI, August 27-28.

CDM Association of Japan. (1994). *CDM Manual*. Cement Deep Mixing Association of Japan. In Japanese, 194 p.

CDM Association. (1996). Promotional information.

Chen, X.L., Y.H. Liu, and S.D. Zhang. (1996). "Design methods of the cement-soil retaining wall." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 475-480.

Condon Johnson Associates. (1998). "Geo-Jet." Presented at the University of Wisconsin Milwaukee Seminar, Deep Mixing Methods, Milwaukee, WI, August 27-28.

Condon Johnson Associates. (1999). "Geo-Jet." Presented at the University of Wisconsin Milwaukee Short Course on Deep Mixing Methods, Anaheim/Orange County, CA, April 26-27.

Das, B.M. (1983). *Advanced Soil Mechanics*. McGraw-Hill Book Company, Hemisphere Publishing Corporation.

Day, S.R., and C.R. Ryan. (1995). "Containment, stabilization, and treatment of contaminated soils using in situ soil mixing." American Society of Civil Engineers, Specialty Conference, New Orleans, LA, February 22-24, 17 p.

DJM Association of Japan. (1993). *DJM Manual*, Dry Jet Mixing Association of Japan. In Japanese, 75 p.

DJM Association. (1994). Promotional information.

DJM Association. (1996). Promotional information.

Dong, J., K. Hiroi, and K. Nakamura. (1996). "Experimental study on behavior of composite ground improved by Deep Mixing Method under lateral earth pressure." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 585-590.

Druss, D.L. (1998). "Quality assurance/quality control and verification of deep mixing methods." Presented at the University of Wisconsin Milwaukee Seminar, Deep Mixing Methods, Milwaukee, WI, August 27-28.

Edstam, T., and P. Carlsten. (1999). "A new method for laboratory preparation of stabilized clay." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 315-319.

Esrig, M. (1997). Presentation on Stabilator. La Guardia Marriott, May.

Esrig, M. (1999). "Lime-Cement Columns." Presented at the University of Wisconsin Milwaukee Short Course on Deep Mixing Methods, Anaheim/Orange County, CA, April 26-27.

European Pre-Standard ENV197.

Federal Highway Administration. (2000). "An introduction to the deep mixing method as used in geotechnical applications." Prepared by Geosystems, L.P., Document No. FHWA-RD-99-138, March, 143 p.

Federal Highway Administration. (2000). "Supplemental reference appendices for an introduction to the deep mixing method as used in geotechnical applications." Prepared by Geosystems, L.P., Document No. FHWA-RD-99-144, December, 295 p.

Finnish Technical Development Center. (1996). Tekes/Ympäristögeotekniika, Stabiloinnin markkinet-selvitys, Markku Tuhola OL - Consulting Oy, October 11.

Fujita, T. (1996). "Application of DJM under special conditions." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo '96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 591-594.

Futaki, M., K. Nakano, and Y. Hagino. (1996). "Design strength of soil-cement columns as foundation ground for structures." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 481-484.

Gause, C. (1998). "Principles of grout design." Presented at the University of Wisconsin Milwaukee Seminar, Deep Mixing Methods, Milwaukee, WI, August 27-28.

Geo-Con, Inc. (1996). Promotional information.

Geo-Con, Inc. (1998). Promotional information.

GeoLogan. (1997). *Grouting – Compaction, Remediation, and Testing*, Proc. of Sessions Sponsored by the Grouting Committee of the Geo-Institute of the American Society of Civil Engineers, Logan UT, Ed. by C. Vipulanandan, Geotechnical Special Publication No. 66, July 16-18, pp. 212-229.

Gotoh, M. (1996). "Study on soil properties affecting the strength of cement treated soils." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 399-404.

Greenwood, D.A. (1987). "Simple techniques of ground improvement with cement." *Proceedings International Conference on Foundations/Tunnels,* Vol. 2, pp. 8-19.

Hagblom, J. (1997). Personal communication

Haley and Aldrich, Inc. (1991). "Final geotechnical data report, Central Artery (I-93)/Tunnel (I-90) Project, Design Section D007A, Boston, MA," prepared for the Massachusetts Department of Public Works, File No. 10360-54, Vols. 1-2, Boston, MA.

Haley and Aldrich, Inc. (1995). "Summary report of April, 1995 fact finding trip on soil mixing methods," Excavation Support/I-90 Marine Tunnels, submitted by Maguire Group, Inc. and Frederic R. Harris, Inc. to Bechtel/Parsons Brinckerhoff, Boston, MA.

Halkola, H. (1999). "Keynote Lecture: Quality control for dry mix methods." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 285-294.

Hampton, M.B., and T.B. Edil. (1998). "Strength gain of organic ground with cement-type binders." *Soil Improvement for Big Digs.* Proceedings of Sessions of Geo-Congress 98, American Society of Civil Engineers, Geotechnical Special Publication No. 81, Boston, MA, October 18-21, pp. 135-148.

Hane, E., and H. Saito. (1996). "Quality assessment of cement-mixed soil ground by S-wave tomography." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 129-132.

Harnan, C.N., and Y.U. Iagolnitzer. (1992). "COLMIX: the process and its applications." *Grouting in the Ground.* Institution of Civil Engineers, Thomas Telford, London. pp. 511-525.

Hartlen and Holm. (1995). "Deep stabilization of soft soils with lime-cement columns." Proceedings of the Bengt B. Broms Symposium on Geotechnical Engineering, Singapore, December, pp. 173-179.

Hayashi, H. and J. Nishikawa. (1999). "Mixing efficiency of dry jet mixing methods applied to peaty soft ground." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the

International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 333-338.

Hayward Baker, Inc. (1997). Promotional information.

Hercules. (1999). Promotional information.

Hibino, S. (1989). "Summary and histories of Tenocolumn Method." *Foundation Engineering and Equipment (Kiso-Ko)*, pp. 90-96.

Hirai, T., J. Ise, T. Kusakari, M. Gotou, and Y. Hibi. (1996). "Development and application of Deep Mixing Soil Stabilization method to control displacement of adjacent ground." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 485-490.

Hiraide, A., K. Baba, and H. Azuma. (1996). "Quality assessment of cement stabilized soil by S-wave logging." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 603-606.

Hirama, K., and M. Torrihara. (1983). "Dynamic properties of cement-mixed soils – influences of cement content and duration of curing," Report of Ohbayashi Corporation Technical Institute, No. 26, In Japanese, pp. 92-96.

Hoboya, Y., T. Nasu, Y. Hibi, T. Ogino, Y. Kohata, and Y. Makihara. (1996). "JGS TC Report: An evaluation of the strength of soils improved by DMM." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 919-924.

Holm, G. (1994). "Deep stabilization by admixtures, 13th International Conference on SMFE, Proceedings, Vol. 3, New Dehli, pp. 1123-1126.

Holm, G.H. (1997). Personal communication.

Holm, G., H. Bredenberg, and B.B. Broms. (1981). "Lime columns as foundations for light structures." Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering, Vol. 3, Stockholm, Sweden, pp. 687-694.

Holm. G., R. Tran, and A. Ekstorm. (1983). "Improving lime column strength with gypsum." Proceedings of the 8th ECSMFE, Helsinki.

Holm, G., M. Ruin, and S. Hakansson. (1999). "Column penetration tests and extraction of lime/cement columns." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 311-314.

Hosomi, H., S. Nishioka, and S. Takei. (1996). "Method of Deep Mixing at Tianjin Port, People's Republic of China." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 491-494.

Hosoya, Y., T. Nasu, Y. Hibi, T. Ogino, Y. Kohata, and Y. Makihara. (1996). "JGS TC Report: An evaluation of the strength of soils improved by DMM." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 919-924.

Huiden, E.J. (1999). " Soil stabilization for embedment of Botlek Railwaytunnel in the Netherlands." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 45-49.

Huttenen, E. and K. Kujala. (1996). "Assessment of the quality of stabilized peat and clay." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 607-612.

Huttenen, E., K. Kujala, and H. Vesa. (1996). "Assessment of the quality of stabilized peat and clay." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 607-612.

Imamura, S., A. Nakajima, and S. Mitsuishi. (1996). "Evaluation of soil improvement using borehole resistivity profiler." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 613-618.

INQUIP Associates. (1998). Promotional information.

Isobe, K., Y. Samaru, C. Aoki, K. Sogo, and T. Murawaki. (1996). "Large scale deep soil mixing and quality control." *Grouting and Deep Mixing,* Proceedings of IS-Tokyo '96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 619-624.

Jameson, R. (1996). "Mechanical Properties of Soil Cement and Application in Excavation Support," M.S.Thesis, Massachusetts Institute of Technology, Cambridge, MA.

Jasperse, B.H. (1989). "In Situ stabilization using shallow soil mixing and deep soil mixing," Hazardous Materials Management Conference, HazMat Central, Rosemont, IL, March.

Jean Lutz. (1998). Promotional information.

Kamon, M. (1996). "Effect of grouting and DMM on big construction projects in Japan and the 1995 Hyogoken-Nambu Earthquake." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, Vol. 2, May 14-17, pp. 807-823.

Kawasaki, K., H. Kotera, K. Nishida, and T. Murase. (1996). "Deep Mixing by Spreadable Wing Method." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 631-636.

Kim et al. (1991).

Kivelö, M. (1997). "Undrained shear strength of lime-cement columns." Proceedings of the 14th International Conference on Soil Mechanics and Foundation Engineering, Hamburg, Vol. 2, pp. 1173-1180.

Kivelö, M., and B.B. Broms. (1999). "Mechanical behaviour and shear resistance of lime/cement columns." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 193-200.

Kobayashi, A. and F. Tatsuoka. (1982). Mechanical Properties of Saturated Soft Clay Improved by Cement-Mixing VI, Effects of Long-Term Curing Seisan-Kenkyu (*Jour. of IIS*, Univ. of Tokyo), Vol. 34, No. 11, pp. 32-35.

Kohata, Y., Muramoto, K., Maekawa, H., Yajima, J., and Babasaki, R. (1996). "JGS TC Report: Deformation and Strength Properties of DM Cement-Treated Soils." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 905-911.

Kujala, K. (1982). "The use of gypsum in deep stabilization." Oulu University of Technology, Oulu. (in Finnish).

Kujala, K., M. Mäkikyrö, and O. Lehto. (1996). "Effect of humus on the binding reaction in stabilized soils." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 415-420.

Kukko, H., and J. Ruohomäki. (1995). "Stabilization of clays with various binders." VTT Research Notes, 1682. Espoo Technical Research Centre of Finland. In Finnish.

Lahtinen, P., H. Jyrävä, and K. Kuusipuro. (1999). "Development of binders for organic soils." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 109-114.

Lambrechts, J.R. (1999). "Properties of mixed soil." Presented at the University of Wisconsin Milwaukee Seminar, Deep Mixing Methods, Anaheim/Orange County, CA, April 26-27.

Larsson, S. (1999). "The mixing process at the dry jet mixing method." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 339-346.

Lin et al. (1997).

Lin Q.K., and H.I. Wong. (1999). "Use of deep cement mixing to reduce settlement at Bridge Approaches." *Journal of Geotechnical and Environmental Engineering*, American Society of Civil Engineers, April, pp. 309-320.

Markteknik. (1999). Promotional information.

Master Builders Technologies, Inc. (1998). "Principles of grout design," Presented at the University of Wisconsin Milwaukee Seminar, Deep Mixing Methods, Milwaukee, WI, August 27-28.

Matsuo, O., T. Shimazu, Y. Goto, Y. Suzuki, R. Okumura, and M. Kuwabara. (1996). "Deep Mixing Method as a liquefaction prevention measure." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 521-526.

Millgard Corporation. (1998). Promotional information.

Min, T. (1996). "Application of offshore cement Deep Mixing Method in the construction of Yantai Port." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 527-532.

Mitchell, J. (1993). *Fundamentals of Soil Behavior*, 2nd Edition, John Wiley & Sons, Inc. New York, N.Y.

Mitchell, J.K. (1986). "Practical problems from surprising soil behaviour." Journal of Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, Vol. 12, No. SM3, pp. 259-289.

Miyoshi, A., and K. Hirayama. (1996). "Test of solidified columns using a combined system of mechanical churning and jetting." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 743-748.

Mizutani, T., S. Kanai, and M. Fujii. (1996). "Assessment of the quality of soil-cement columns of square or rectangular shapes formed by a Deep Mixing Method." *Grouting and Deep Mixing,* Proceedings of IS-Tokyo '96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 637-642.

Nicholson, P.J., and E.K. Chu. (1994). "Boston Central Artery Tunnel excavation support remediation by ground treatment." Presented at American Society of Civil Engineers Conference on Soil Structure Interaction and Environmental Issues, Hershey, PA, September 12-14.

Nicholson, P.J., B.H. Jasperse, and M.J. Fisher. (1997). "Economical alternative for containment barriers." International Containment Technical Conference and Exhibit, St. Petersburg, FL, February.

Nicholson, P.J., and B.H. Jasperse. (1998). "Hydraulic and mechanical cement soil mixing." Presented at University of Wisconsin-Milwaukee Short Course, Deep Mixing Methods, Milwaukee, WI, August 27-28, 20 p.

Nicholson, P.J., J.K. Mitchell, E.W. Bahner, and Y. Moriwaki. (1998). "Design of a soil mixed composite gravity wall." *Soil Improvement for Big Digs*, Proceedings of Geo-Congress 98, American Society of Civil Engineers, Geotechnical Special Publication No. 81, Boston, MA, October 18-21, pp. 27-40.

Nishida, K., Y. Koga, and N. Miura. (1996). "Energy consideration of the Dry Jet Mixing method." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 643-648.

Nishikawa, J., S. Tada, and S. Yamaguchi. (1996). "Strength evaluation based on logging in ground stabilized by Deep Mixing Method." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 649-652.

O'Rourke, T.D., A.J. McGinn, J. Dewshap, and H.E. Stewart. (1997). "Performance of excavations stabilized by deep soil mixing." Report prepared for Bechtel Parsons Brinckerhoff, Cornell University, Ithaca, NY, June, 214 p.

O'Rourke, T.D., and A.J. McGinn. (1998). "Parametric study of proposed excavation for Ramp D Sta 29+00." Report prepared at Cornell University for Bechtel/Parsons Brinckerhoff, Massachusetts Highway Department, and Federal Highway Administration.

O'Rourke, T.D., and A.J. McGinn. (1999). "Study of stability and soil mix performance during dredging adjacent to Ramp D." Report prepared at Cornell University for Bechtel/Parsons Brinckerhoff, Massachusetts Highway Department, and Federal Highway Administration.

Ohmura, T., M. Murata, and N. Hirai. (1981). "Site measurement of hydration-generated temperature in ground improved by Deep Mixing Method and effect of curing temperature on improved soil." 36th Annual Meeting of Japan Society of Civil Engineering, Section III, pp. 732-733.

Okumura, T. (1977). "Discussion on Broms' and Boman's "Lime column, new type of vertical drain." Proceedings of the 9th ICSMFE, Vol. 3, p. 387.

Okumura, T. (1996). "Deep Mixing Method of Japan." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, Vol. 2, May 14-17, pp. 879-887.

Okumura, T., and M. Terashi. (1975). "Deep-Lime-Mixing method of stabilization for marine clays." Proc. 5th Asian Regional Conference on SMFE, Vol. 1, pp. 69-75.

Ou, C.Y., and T.S. Wu. (1996). "Evaluation of material properties for soil improvement in excavation." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 545-550.

Pagliacci, F., and Pagotto, G. (1994). "Soil Improvement Through Mechanical Deep Mixing Treatment in Thailand." Proc. 5th DFI International Conference, Bruges, Belgium, June 13-15, pp. 5.11 – 5.17.

Pavianni, A., and G. Pagotto. (1991). "New technological developments in soil consolidation by means of mechanical mixing, implemented in Italy for ENEL Power Plant at Pietrafitta." 10th International Conference on Soil Mechanics and Foundation Engineering, Florence, pp. 511-516.

Pile Dynamics, Inc. (1999). Promotional information.

Porbaha, A. (1998). "State of the art in deep mixing technology: part I, Basic concepts and overview." *Ground Improvement*, Journal of the International Society of Soil Mechanics and Geotechnical Engineering (TC-17), Thomas Telford, Vol. 2, No. 2, pp. 81-92.

Queiroz de Carvalho, J.B. (1981). "Amorphous Materials and Lime Stabilized Soils." *Proceedings of the 10th International Conference on Soil Mechanics and Foundation Engineering*, Stockholm, Vol. 3, pp. 761-764.

Rajasekaran, G., K. Murali, B. Dhanaseelan, and R. Srinivasaraghavan. (1996). "Influence of some pollutants on lime treated marine clay." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 437-442.

Rathmayer, H. (1996). "Deep mixing methods for soft subsoil improvement in the Nordic Countries." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, Vol. 2, May 14-17, pp. 869-877.

Reavis, G., and F.C. Freyaldenhoven. (1994). "GEOJET Foundation System." *Geotechnical News*. Vol. 12, No. 4, "Geosystems Issue." Ed. M.E. Caves, December. pp. 56-60.

Rodio, Inc. (1990). Technical Information.

Ryan C.R, and B.H. Jasperse. (1989). "Deep Soil Mixing at Jackson Lake Dam." Proceedings of the ASCE 1989 Foundation Engineering Congress, *Foundation Engineering: Current Principles and Practices*, Vol. 1 and 2, Evanston, Illinois, June 25-29, 1989, pp 354-367.

Ryan C.R. and B.H. Jasperse. (1992). "Stabilization and fixation using soil mixing." *Grouting Soil Improvement and Geosynthetics,* American Society of Civil Engineers, Geotechnical Special Publication No. 30, New Orleans, LA, pp. 1273-1284.

Ryan C.R., and Walker, A.D. (1992). "Soil Mixing for Soil Improvement - Two Case Studies." Ohio River Valley Geotechnical Seminar, XXIII, October 16, Louisville, KY.

Saito, S., K. Babasaki, Y. Hichida, T. Haruki, Y. Momokura, and T. Seien. (1982). "Studies of the soils treated by DCM method." Takenaka Technical Research Report, No. 28, In Japanese, pp. 1-10.

Saito, S., T. Kawasaki, N. Niina, R. Babasaki, and T. Miyata. (1980). "Research on Deep Mixing Method using cementitious agents (Part 10) – Engineering properties of treated soils (3)," *15th Soil Engineering Research Conference*, pp. 717-720.

Saitoh, S., Y. Suzuki, S. Nishioka, and R. Okumura. (1996). "Required strength of cement improved ground." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 557-562.

Shibuya, S., F. Tatsuoka, S. Teachavorasinskun, X.J. Kong, F. Abe, Y.S. Kim, and C.S. Park. (1992). "Elastic deformation properties of geomaterials." *Soils and Foundations*, Vol. 32, No. 3, pp. 26-46.

Shibuya, S., Y.S. Kim, F. Tatsuoka, T. Arata, H. Kurihara, T. Sato, M. Kimura, and T. Nakagaki. (1989). "Stiffness of Cement-Treated Soils at Low Strain Levels." Proceedings of the 24th Japan National Conference on Soil Mechanics and Foundation Engineering, JGS, pp. 653-656.

Shimatsubo, K., S. Naemura, and Y. Shin. (1992). "Evaluation of effectiveness of cement treated columns by rotary penetration test." Proceedings of the 27th Japan National Conference on SMFE, JSSMFE, No. 1, pp. 193-194. (In Japanese).

Shiomi, M., K. Ito, D. Nishimura, H. Tanaka, and K. Tanaka. (1996). "Slope Stability Using the Admixture Method." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 563-568.

Silvester, T. (1999). "Deep soil mixing." *Ground Engineering*. Vol. 32, No. 9, September, pp. 29-32.

Stabilator (1996). Promotional information.

Stabilator. (1997). Promotional information.

Stabilator. (1998). "Lime cement columns: a new soil stabilization technique." Presented at University of Wisconsin - Milwaukee Short Course, Deep Mixing Methods, Milwaukee, WI, August 27-28, 129 p.

Sugawara, N., Y. Ito, and M. Kawai. (1996). "New core sampler with planet gear for investigating the cement-mixed ground." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 653-658.

Sugimura, Y. (1997). "Guideline of design and quality control of ground improvement for buildings – deep mixing system by cement reagent." Japan Building Center.

Suzuki, Y. (1982). "Deep Chemical Mixing Method using cement as a hardening agent." Proc. Symp. on Rock and Soil Improvement Techniques, AIT, Bangkok, November 29 – December 3, Paper B-1.

Suzuki, K., T. Nasu, and K. Adachi. (1992). "Sampling of soil stabilized by deep mixing method with triple-tube core sampler, Proceedings of Sampling symposium, JSSFME, pp. 107-114. (In Japanese).

Takenaka Civil Engineering (also referred to as Takenaka and Takenaka). (1995). "Deep chemical mixing method using cement as hardening agent." Published by Takenaka Corporation, Tokyo

Taki, O., and D.S. Yang. (1989). "Excavation support and groundwater control using soil cement mixing wall for subway projects," Proceedings, Rapid Excavation and Tunneling Conference, Los Angeles, CA, June 11-14, pp. 156-175.

Taki, O., and D.S. Yang. (1990). "The emergence of the soil-cement mixed wall technique and its application in the 1990s." Deep Foundation Institute Annual Meeting, Seattle.

Taki, O., and D.S. Yang. (1991). "Soil-cement mixed wall technique." American Society of Civil Engineers, Proceedings, Geotechnical Engineering Congress, Denver, CO. pp. 298-309.

Taki, O., and R.A. Bell. (1997). "Booklet on Soil-cement pile/column." SCC Technology, Inc., 23 p.

Taki, O., and R.A. Bell. (1998). "Soil-Cement Pile/Column – A system of deep mixing." *Soil Improvement for Big Digs*, Proceedings of Geo-Congress 98, American Society of Civil Engineers, Geotechnical Special Publication No. 81, Boston, MA, October 18-21, pp. 27-40.

Tamura, M., M. Futaki, and A. Abe. (1996). "Use of non-destructive method for the evaluation of reclaimed soil column." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 659-664.

Tateyama, K., R. Fukagawa, and T. Tsujii. (1996). *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 665-668.

Tatsuoka, F. and A. Kobayashi. (1983). "Triaxial Strength Characteristics of Cement Treated Soft Clay." Proceedings of the 8th European Confrence on Soil Mechanics and Foundation Engineering, Helsinki, Vol. 1, pp. 421-426.

Tatsuoka, F., and S. Shibuya. (1992). "Deformation characteristics of soils and rocks from field and laboratory tests." Report of the Institute of Industrial Science, The Unviersity of Tokyo, Vol. 38, No. 5.

Tatsuoka, F., Y. Kohata, T. Tsubouchi, K. Murata, K. Ochi, and L. Wang. (1995). "Disturbance in rotary core tube sampling in soft rock." Proc. Intl. Conf. on Advances in Site Investigation Practice, Institution of Civil Engineers, London, March.

Tatsuoka, F., Y. Kohata, K. Uchica, and K. Imai. (1996). "Deformation and strength characteristics of cement-treated soils." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 453-459.

Terashi, M., and H. Tanaka. (1983). "Settlement analysis for Deep Mixing Method." Proc. 8th European Conference on SMFE, Vol. 2, pp. 995-960.

Terashi, M., H. Tanaka, T. Misumoto, T.S. Honma, and T. Ohashi. (1983). "Fundamental properties of lime and cement treated soils." (3rd Report). Report of Port and Harbour Research Institute, Vol. 22, No. 2, pp. 69-96.

Terashi, M. (1997). Personal communication.

Terra Constructors. (1998). Promotional information and personal communication.

Terzaghi, K., and R.B. Peck. (1948). *Soil Mechanics in Engineering Practice.* John Wiley & Sons, Inc., 729 pp.

Thompson, M.R. (1966). "Lime Reactivity of Illinois Soils," *Journal of the Soil Mechanics and Foundations Division, ASCE*, SM5, pp. 67-92.

Toa Corporation. (undated, late 1980s).

Trevi-ICOS Corporation. (1998). "The "Trevimix" method for deep soil mixing." Presented at the University of Wisconsin Milwaukee Short Course on Deep Mixing Methods, Milwaukee, WI, August 27-28.

Uchida, K. Y., Shioi, H. Hirukawa, and F. Tatsuoka. (1993). "The Trans-Tokyo Bay Highway Project – A huge project currently under construction." Proceedings of the International Conference on Transportation Facilities through Difficult Terrain, Balkema, pp. 57-87.

Uchida, K., K. Imai, F. Tatsuoka, Y. Kohata. (1996). "Ground improvement by cement-treatment in Trans-Tokyo Bay Highway Project." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 669-674.

Ueki, H., K. Hasegawa, K. Suzuki, and M. Bessho. (1996). "Development of a high-pressure Jet Mixing Method for displacement-reducing." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 767-772.

Unami, K., and M. Shima. (1996). "Deep Mixing Method at Ukishima site of the Trans-Tokyo Bay Highway Project." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 777-782.

Walker, A.D. (1992). "Soil mixing and jet grouting on a hazardous waste site, Pittsburgh, PA, USA," *Grouting in the Ground.* Institution of Civil Engineers, Thomas Telford, London. pp. 473-486.

Walker, A.D. (1994). "A deep soil mix cut-off wall at Lockington Dam in Ohio." *In Situ Deep Soil Improvement*, American Society of Civil Engineers, Geotechnical Special Publication No. 45, pp. 133-146.

Watanabe, T., S. Nishimura, M. Moriya, and T. Hirai. (1996). "Development and application of new Deep Mixing Soil Improvement Method to form a rectangular stabilized soil mass." *Grouting and Deep Mixing*, Proceedings of IS-Tokyo'96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 783-786.

Watn, A., S. Christensen, A. Emdal, and S. Nordal. (1999). "Lime-cement stabilization of slopes – Experiences and a design approach." *Dry Deep Mix Methods for Deep Soil Stabilization*, Proceedings of the International Conference on Dry Deep Mix Methods for Deep Soil Stabilization, Ed. by H. Bredenberg, G. Holm, and B. Broms, Stockholm, Sweden, October 13-15, pp. 169-176.

Yang, D.S. (1997). "Chapter 2.5: Deep Mixing." *In Situ Ground Improvement, Reinforcement and Treatment: A Twenty Year Update and a Vision for the 21st Century*, Ground Reinforcement Committee, American Society of Civil Engineers, Geo-Institute Conference, Logan, UT, July 16-17, pp. 130-150.

Yang, D.S., and S. Takeshima. (1994). "Soil mix walls in difficult ground." In Situ Ground Improvement Case Histories, American Society of Civil Engineers, National Convention, Atlanta, GA. October. pp. 106-120.

Yang, D.S., J.N. Yagihashi, and S.S. Yoshizawa. (1998). "Dry Jet Mixing for stabilization of very soft soils and organic soils." *Soil Improvement for Big Digs*. The Geo-Institute of the American Society of Civil Engineers, Geotechnical Special Publication No. 81, pp. 96-110.

Yano, S., S. Tokunga, M. Shima, and K. Kamimura. (1996). *Grouting and Deep Mixing*, Proceedings of IS-Tokyo '96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17. pp. 681-687.

Yoshida, S. (1996). “Shear strength of improved soils at lap-joint-face.” *Grouting and Deep Mixing*, Proceedings of IS-Tokyo ’96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17. pp. 461-466.

Yoshizawa, H., R. Okumura, Y. Hosoya, M. Sumi, and T. Yamada. (1996). “JGS TC Report: Factors affecting the quality of treated soil during execution of DMM.” *Grouting and Deep Mixing*, Proceedings of IS-Tokyo’96, The Second International Conference on Ground Improvement Geosystems, Tokyo, May 14-17, pp. 931-937.

Zen, K., H. Yamazaki, and Y. Sato. (1990). "Strength and Deformation Characteristics of Cement Treated Sands Used for Premixing Method." *Report of the Port and Harbor Research Institute*, Vol. 29, No. 2, pp. 86-118.