Levee Stability Application for Deep Mixing (1) – Design for Full Scale Test Section Using Dry Mixed Soil Cement Columns

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The New Orleans District of the U.S. Army Corps of Engineers is considering Deep Mixing to improve the foundation beneath a planned earthen levee. Dry mix soil cement columns were selected as the ground improvement technology best suited to the site conditions. Experimental full-scale columns were installed in Phase I of the test section using data obtained from a bench-scale design mix study. Variables tested included lime and cement delivery rate, mixing energy, column installation methods, and in-situ shear strength test methods. Data and conclusions from the bench scale test, Phase I design mix evaluation, and design and construction of the Phase II test sections are presented in this paper.

INTRODUCTION AND BACKGROUND

The city of New Orleans, Louisiana is located along the Mississippi River, approximately 100 miles from its mouth. The city is surrounded on all sides by flood protection levees and walls. To the south of the city's downtown business district is the Mississippi River, and to the north is Lake Pontchartrain, which is open to the Gulf of Mexico and to hurricane storm surge. To the west is the Bonnet Carre Spillway, a flood control feature that discharges Mississippi River floodwater into Lake Pontchartrain, and to the east is the Inner Harbor Navigation Channel (IHNC), which locks shipping from the Mississippi River to the Mississippi River Gulf Outlet, a shortcut to the Gulf of Mexico (Figure 1).

The IHNC is important to the economic vitality of the Port of New Orleans, but the lock is nearly 100 years old and is obsolete. Replacement of the IHNC Lock would require that the tie-in flood protection be relocated and raised 2.6m from hurricane protection grade to the higher mainline Mississippi River flood control grade.

A replacement ship lock was authorized in 1956 and has been in the planning stages for many years. Two significant obstacles to its construction have been inadequate funding and neighborhood opposition to the project. Funding is becoming available and neighborhood opposition could be partially overcome by raising the replacement lock forebay flood protection, (Figure 2), within the existing right-of-way using a more aesthetically pleasing earthen levee rather than a concrete floodwall.

Because of low slope stability factors of safety and tight right-of-way constraints, the tie-in protection would ordinarily have to be constructed using a pile-supported concrete wall. Alternatively, the factors of safety for the earthen levee could be upgraded by improving the overall foundation competency through deep mixing. All-earth levees are preferred over structural options for flood control levees that are susceptible to vessel allision. For the IHNC Lock forebay levees, the risk of direct ship impact is slight, but nonetheless requires design consideration. However, at the IHNC site, slope stability concerns do not permit the

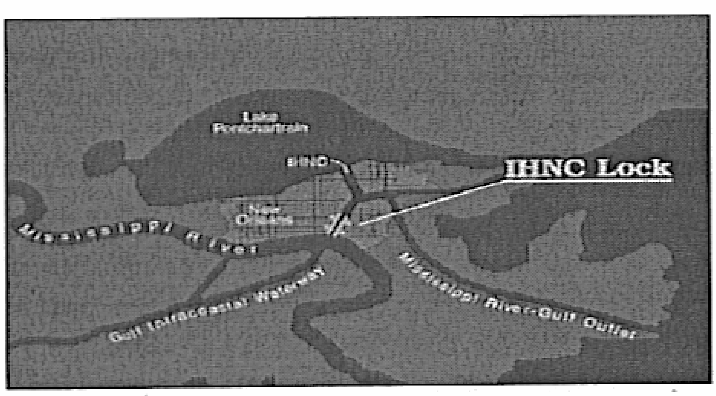


Figure 1 Project Location Map: Inner Harbor Navigation Channel Lock

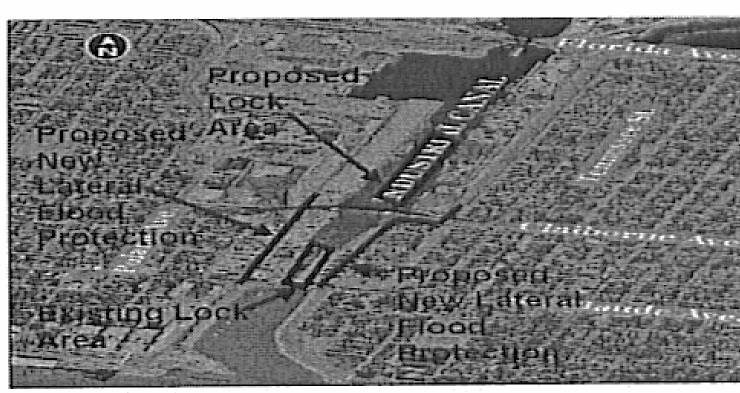


Figure 2 New IHNC Lock Site

construction of a conventional full earthen levee section within the existing right of way. A solution to this problem is to strengthen the foundation soil to yield the required 1.30 slope stability factor of safety for Mississippi River levees.

PRELIMINARY DESIGN

To provide a factor of safety of 1.30 for the design levee, the shear strength of the upper 26 feet* of the foundation soil must be improved from an average value of 350 psf to 2,000 psf. This could be achieved by replacement of 30% of the soil beneath the levee footprint with columns having an average shear strength of 6,060 psf, or 42.1 psi, as indicated in equation 1 (Broms, 1999).

$$c_{sv} = c_{ns} \, (1 - A_c) + c_{lc} \, A_c \qquad (1)$$

where c_{sv} is the shear strength of the total soil volume,
c_{ns} is the shear strength of the natural soil,
A_c is the part of the total shear surface covered by the columns, and
c_{lc} is the shear strength of the columns

For a 30% replacement ratio,

$$2{,}000 \text{ psf} = 260 \text{ psf} \, (1 - 0.30) + c_{lc} \, (0.30)$$

$$c_{sv} = 6{,}060 \text{ psf} = 42 \text{ psi} = 290 \text{ kPa}$$

As a next step toward evaluation of deep mixing as a possible construction method for the project an objective assessment of the applicability of deep mixing technologies to the IHNC levee site was made. The provisional viability was driven by the strength required and the elimination of possibly contaminated spoil disposal. In reconnaissance scope, it was recommended to install approximately 10,000 columns each 33-ft deep beneath the 2,000-ft long reach of levee. The cost was estimated to be $3.2 million, including mobilization and demobilization, installation of the columns, and construction of the earthen levee, compared to $6.2 million for a structural alternative. Ground improvement would therefore be economically competitive if design and QA/QC concerns could be satisfactorily addressed.

A test section was a prerequisite for this type of construction since the U.S. Army Corps of Engineers has never used deep mixing for such an application. The test section was necessary to optimize the design (and cost) based on the site-specific conditions, to demonstrate that columns of the target design strength can be routinely formed in these site conditions, to gain a better understanding of the complex column-soil interaction for a slope stability application, and to establish QA/QC procedures for the construction contract. The site selected was adjacent to the existing lock on U.S. Government property.

**(Original project data were in imperial units, and for historical continuity these units are used where appropriate.)*

SITE CHARACTERIZATION

The area consists mainly of fill and recent Holocene Age soils consisting of swamp/marsh deposits, deltaic plain deposits, beach ridge sand deposits, and near shore Gulf deposits, to approximate El. –65 ft. The swamp/marsh deposits consist of interbedded very soft to stiff, organic fat clay with occasional layers and lenses of peat and silt and lenses of soft to medium lean clay. They average 14 ft. thick and range from approximate El. –5 to approximate El. –18. Intermediate and Interdistributary deposits underlie the swamp deposits and consist of interbedded very soft to medium fat clays with occasional layers and lenses of silt and soft lean clays and lenses of silty sand. They average 34 ft. thick and range from approximate El. –18 to approximate El. –52. Beach, nearshore gulf, and prodelta deposits underlie the interdistributary deposits. Beach deposits consist predominantly of interbedded sand and silty sand with shell fragments and occasional layers and lenses of stiff to medium fat clay and soft to medium lean clay. Nearshore gulf deposits consist of interbedded sand, silty sand, silt, with some shell fragments and occasional lenses of very soft and soft, fat and lean clays. The prodelta deposits consist of homogeneous soft to medium fat clay with occasional lenses of soft lean clay. These types of soils range from approximate El. –44 to approximate El. –65. Pleistocene Age soils are present from approximate El. –65 to the deepest boring termination depth at approximate El. –140. Pleistocene deposits consist of stiff to very stiff oxidized clays interbedded with layers and lenses of silts and sands

Boring TS-1U, a 5-inch diameter, 100 foot deep undisturbed boring, was made at the test section site. The plotted boring log is presented as Figure 3. A generalized view of the stratification is presented in Figure 4

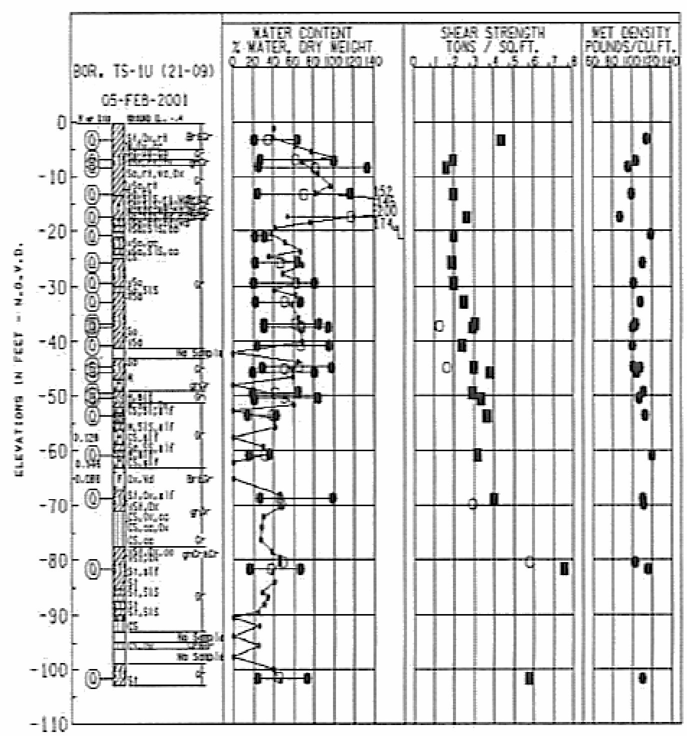


Figure 3 TS-1U Boring

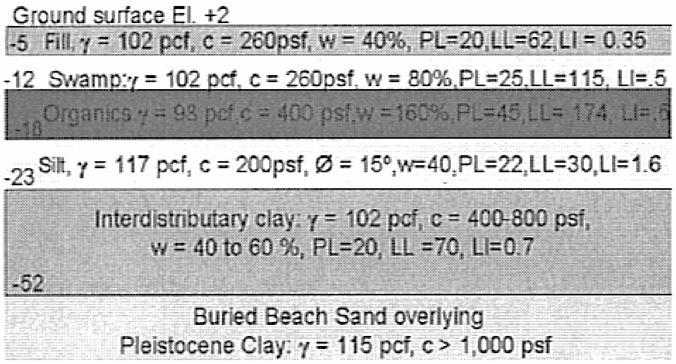


Figure 4 Generalized Site Statigraphy and Soil Properties

TEST PROGRAM GOALS

The test program was planned for three phases: a bench scale test and a full-scale test section in two phases, the specific objectives of which were:

1) Bench Scale Test and Phase I Test - To obtain comparative data regarding the in-situ relationship between column shear strength and column design parameters, such as design mix, loading rate, and mixing energy.
2) Bench Scale Test and Phase I Test – From full-scale column data, adjust the initial design for the Phase II test section so that loading to failure could be achieved.
3) Phase II Load Test - To verify column/soil interaction assumptions made for infinite levee slope stability analyses upon which the actual flood protection levee design would be based.
4) Phases I and II Test - To study the construction methods, quality test methods, and intangible aspects of construction using lime cement columns.

To accomplish the stated goals, a full-scale test section was to be loaded to failure in Phase II.

BENCH SCALE TEST

A bench scale test was conducted to evaluate the effect of binder component proportion and loading rate on shear strength for each distinct soil type for the study depth. Bench scale test data are not to be relied on for design, but are reliable indicators of strength trends and good predictors of full-scale test data.

Samples from the four different types of soil from the undisturbed boring were mixed in the laboratory with five different mixtures and dosages of binder. They were then pressed and compacted into plastic molds to form samples 50mm in diameter and 100 mm high. Two chemical additives were added as binder: Portland Type II cement and high calcium quicklime. Four specimens of each combination were made so that unconfined compressive strength tests could be performed after 7-day, 14-day, 28 day, and 56-day curing periods. Unconfined compression test were run on all eighty specimens. The specimens were tested in an electronically controlled loading frame and loaded to failure by deformation control at a strain rate of 1.5% per minute. After compressive strength testing, water content and density were measured for the test specimens. The design mixtures and unconfined shear strength results are reported in Figure 5.

In general, all four-soil types confirmed that the shear strength was dependent on binder content and age. Also (with the exception of the Intermediate Deposit), samples mixed with 100% cement binder yielded higher strengths than those mixed with lime and cement. A continuing strength increase beyond 56 days for all soil types was observed, except for the Organic Deposit.

Swamp Deposit. As shown in Figure 5, only the 100% Cement, 200-kg/m^3 mix exceeded the 28-day criterion of 290 kPa shear strength, although the 100% Cement mix exceeded the criterion in the interval 28 to 56 days.

Organic Deposit. It was expected that the organic deposit would require greater cement loading rate for strength improvement. Figure 5 indicates that no specimen exceeded 211 psi even at 56 days curing time. A substantially higher strength was observed for the 100% Cement, 200-kg/m^3 mix over the 100% Cement, 150-kg/m^3 mix (i.e., about 60 % increase at 28 to 56 days). A binder dosage of at least 200-kg/m^3 and a binder content of 100 % Cement would be required to meet the load test design requirement

Deposit	Mix Cement/Lime	Dosage kg/m^3	Shear Strength (kPa) Days 7	14	28	56
Swamp	50/50	100	97		144	297
	50/50	150	86	89	161	
	100/0	100	159	229		189
	100/0	150	233	221	273	306
	100/0	200	293		376	435
Organic	50/50	100	85	84	103	
	50/50	150	104	114	115	
	100/0	100	94	105	116	
	100/0	150	124	118		125
	100/0	200	129	192	211	
Inter-mediate	50/50	100	110	165	296	552
	50/50	150	229	329	582	1134
	100/0	100	162	198	304	313
	100/0	150	257	296		581
	100/0	200	393	506	669	982
Interdis-tributary	50/50	100	159	204	334	399
	50/50	150	198	283	369	686
	100/0	100	274	307	382	392
	100/0	150	340	480	520	655
	100/0	200	35	54	67	

Intermediate and Interdistributary Deposits All design combinations exceeded the 28-day compressive strength criterion, as shown in Figure 5.

Figure 5. Bench Scale Test Data

It was concluded from this bench test that the only way to achieve the design strength in the organic deposit was to use 100% cement. However lime application adds greater column ductility. So, lime cement columns were also included in Phase I of the test section.

PHASE I TEST

One objective of Phase I was to obtain comparative data regarding the in-situ relationship between column shear and column design parameters (design mix, loading rate, and mixing energy). Triplicate sets of the eight combinations of design parameters shown in the matrix in Figure 6 were installed outside of the footprint of Phase II load test cells. All columns were 10-m deep and 800 mm in diameter.

An important objective of Phase I was to gain confidence in a QA/QC test method that would be immediate enough to evaluate column quality during a construction contract without delaying the installation. This would be a requirement for acceptance of deep mixing technology as a means of improving slope stability for flood protection projects. For the triplicate columns, the specifications called for pressuremeter and Reverse Column Penetration (RCPT) testing to be performed for each type of column. Of the eight remaining columns, the four 100 % cement columns were to be exposed for coring and laboratory testing while the four remaining 3:1 cement/lime columns were to have field axial compression tests performed. However as discussed below and presented in Figure 7, the testing program was modified during construction in response to actual circumstances.

Actual strength was to be measured by pulling a probe, installed with the column, through the column at prescribed column ages. These data would then be correlated to their equivalent bench scale test results, providing a valuable link for establishing the design mixture for the project. This would also provide the basis for QA/QC testing for the production columns. However, for the first three attempts at probe testing after five days curing time, the probe could not be pulled through the columns.

Figure 6. Phase I Column Design

Mix Composition	Cement factor	Mixing Method
100% Cement	150 kg/m^3	Uniform for all soils
75% Cement/25% Lime	200 kg/m^3	50 % increase in Organic Layer

Figure 7 As-Built Phase I Columns and Testing

	Mix Composition	Cement Factor	Mix Method	Testing Type
1	100% C	150 kg/m^3	1	RCPT
2	100% C	150 kg/m^3	1	PM
3	100% C	150 kg/m^3	1	EXP
4	100% C	150 kg/m^3	1	RCPT
5	25%L/75%C	150 kg/m^3	1	PM
6	100% C	134 kg/m^3	1	RCPT
7	100% C	200 kg/m^3	1	RCPT
8	100% C	130 kg/m^3	1	PM
9	100% C	200 kg/m^3	1	EXP
10	100% C	139 kg/m^3	1	PM
11	25%L/75%C	200 kg/m^3	1	PM
12	100% C	136 kg/m^3	1	RCPT
13	100% C	200 kg/m^3	1	RCPT
14	100% C	130 kg/m^3	2	PM
15	100% C	150 kg/m^3	1	EXP
16	100% C	144 kg/m^3	1	PM
17	25%L/75%C	150 kg/m^3	2	PM
18	100% C	153 kg/m^3	1	RCPT
19	100% C	200 kg/m^3	1	RCPT
20	100% C	200 kg/m^3	2	PM
21	100% C	200 kg/m^3	1	EXP
22	100% C	154 kg/m^3	1	PM
23	25%L/75%C	200 kg/m^3	2	PM
24	100% C	150 kg/m^3	1	CPT

RCPT=Reverse Column Penetration Test
PM=Pressuremeter
EXP=Exposed for 6' Coring and UU testing
Columns 2,5,8,11,14,17,20 and 23 were also bored using 3inch sampler and UU testing
Mix Method 1 – Injection of binder during both penetration and withdrawal
Mix Method 2 – Same as Mix Method 1 with a remix in Organic layer

Problems were encountered when attempting to conduct each of the three RCPT. In the first instance, the probe cable may have become kinked during the double mixing process the contractor elected to perform; for the other two attempts, the probe could not be mobilized beyond a few inches under the applied maximum tensile capacity of the 0.5-inch diameter steel cable. Because of this unanticipated development, pressuremeter testing was conducted on the next eight columns installed. The eight columns subjected to pressuremeter testing were cored using a variety of different samplers to include a triple tube core barrel, Shelby tubes, and a Pitcher sampler.

The upper 5.2m of four columns were excavated (Figure 8), sealed in plastic wrap, and transported to the New Orleans District reservation for visual inspection. Additional testing was performed using an instrumented and calibrated hydraulic press capable of applying 900 KN of axial load, or approximately 1725 kPa applied compressive stress to the column, see Figure 9. Additionally, the column sections were cored axially and

150 mm diameter specimens were tested at the Waterways Experiment Station in Vicksburg, MS. There, controlled unconsolidated-undrained triaxial compression tests were conducted to measure the shear strength of the columns.

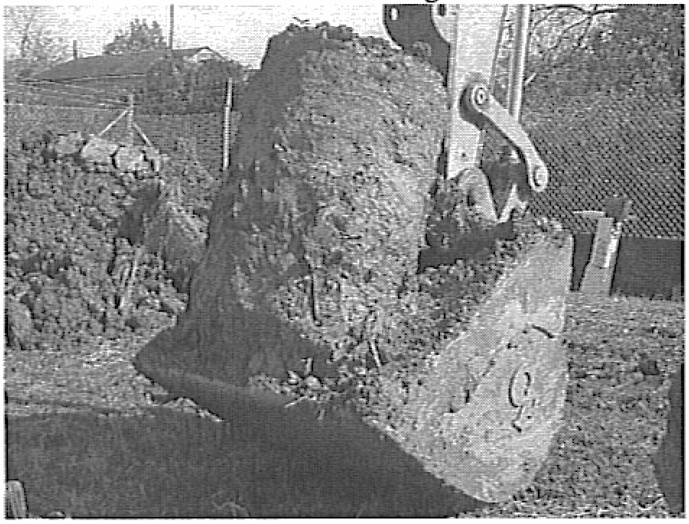


Figure 8 Column Extraction

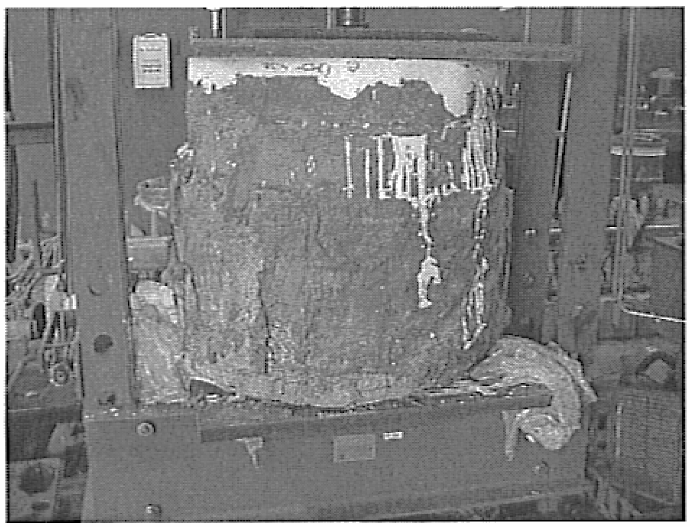


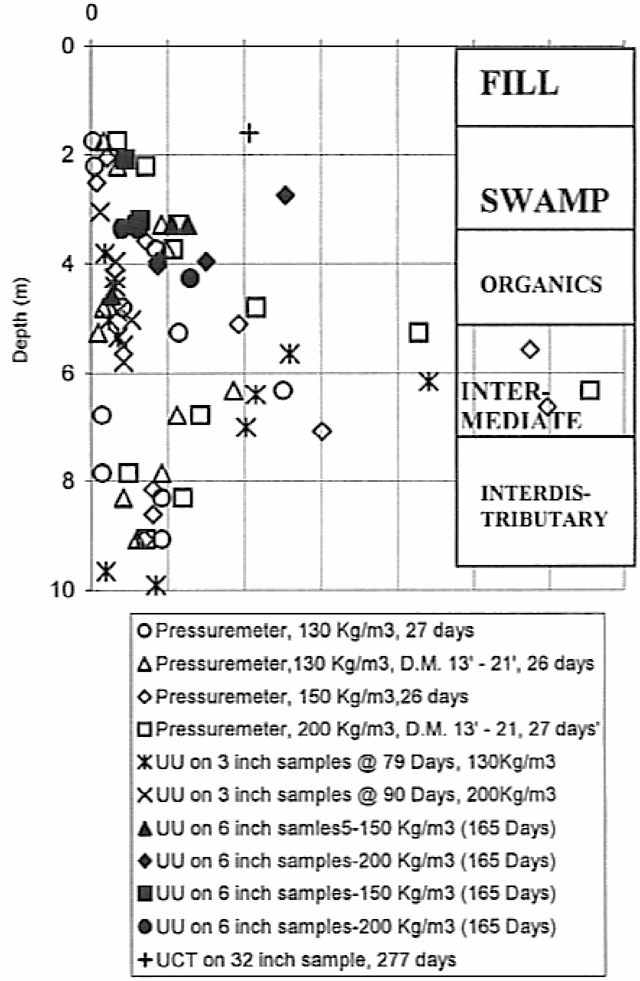
Figure 9 Full Scale Compression Test

Test results from the pressuremeter tests, the 6-inch UU tests, the 3-inch UU test and the full-scale compression test on the 100% Cement columns are presented in Figure 10.

As shown in Figure 11, pressuremeter tests conducted on 23-day old columns constructed using a the 150 kg/m^3 and 200 kg/m^3, with 3:1 ratio of cement to lime, recorded strength values below the target of 290 kPa shear strength in the organic deposit, while tests exceeded the target for the silt and interdistributary deposits. In the 100% cement columns, Figure 10, the strength values in the silt and interdistributary deposits were even greater and tests for the organic layer averaged 290 kPa. It was at this point that the decision was made to use only cement for the test section columns.

Field pressuremeter test results and laboratory bench scale data for binder dosages of 150 kg/m^3 and 200 kg/m^3 cement are presented in Figures 12 and 13. Except for the obvious strength gain with time, no direct correlation between the bench scale and in-situ pressuremeter tests was evident.

Figure 11 Pressuremeter Test Results on 75% Cement / 25% Lime Columns



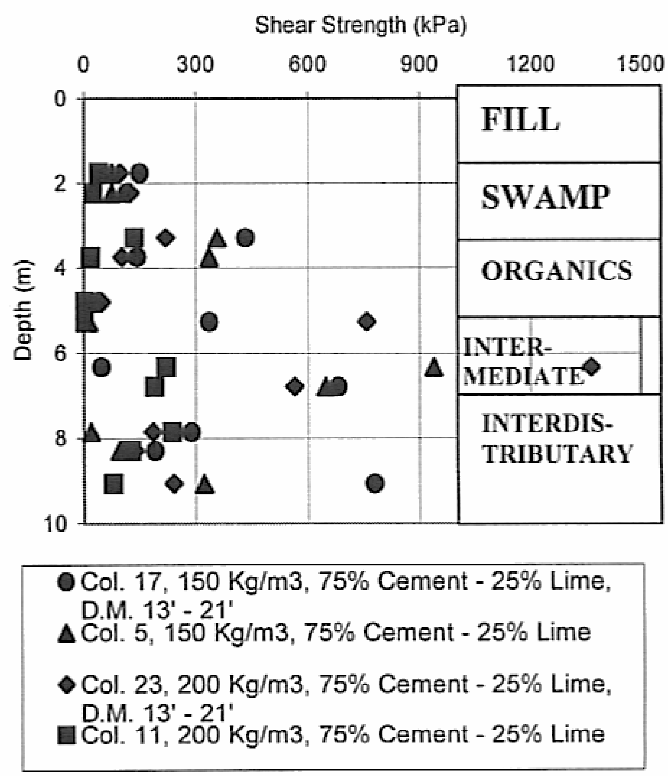


Figure 10 Test Results on 100% Cement Columns

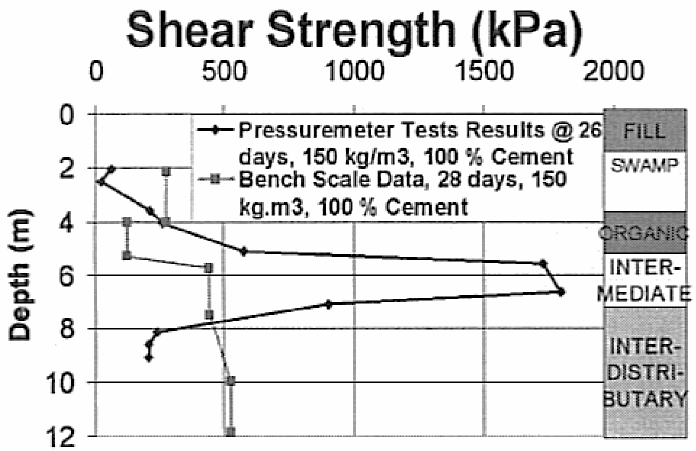


Figure 12 Bench Scale vs. In-situ Pressuremeter Test, 150 kg/m^3 cement

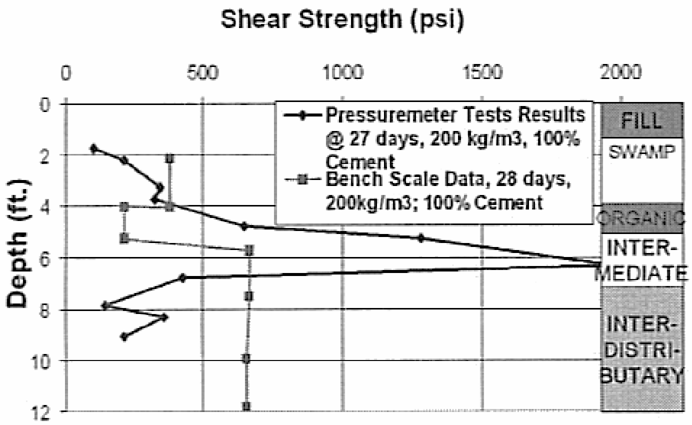


Figure 13 Bench Scale vs. In-situ Pressuremeter Test, 200 kg/m^3 cement

The remaining eight columns installed in Phase I were installed using a design mix of 100% cement, single-mixed, and applying a binder dosage of 100 kg/m^3 for all of the columns except for the organic layer, where the dosage was increased to 175 kg/m^3. These columns were tested as follows: one column by using a conventional CPT for the initial seven hours of curing with CPTs performed at t = 0.5 hr, t = 3 hr, and t = 6 hr after installation; three columns by using the RCPT at t = 24 hr, t = 48 hr, and t = 72 hr; two columns by using a pressuremeter at t = 5 days; and two using the RCPT at t = 5 days. Normalized CPT results from the first 6 hours after column installation are presented in Figure 14, showing the net column strengths, (column strength – in-situ soil strength) are shown.

Field axial compression tests were planned for selected columns to estimate the elastic modulus for the in-situ columns. This idea was abandoned as impractical as no provisions could be made for measuring strain with depth for the column. Also, considering that the considerable side friction forces along the column would gradually counteract the applied compressive force, the compressive stress felt by the column would not be constant and measurable, but would decrease with depth until fully dissipated, and at some depth, no compressive load would be felt.

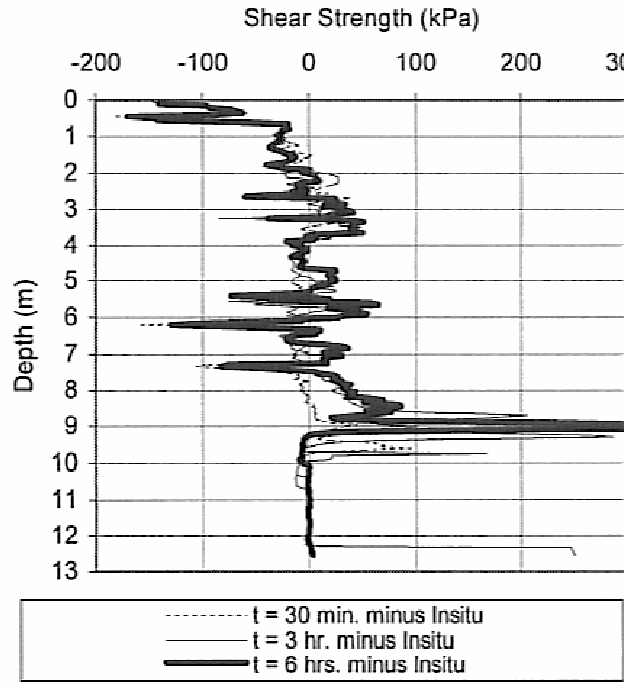


Figure 14 Normalized CPT Data

High shear strength would be desirable for production columns, but for the Phase II columns, the average strength had to be predictable, but weak enough to permit failure. To achieve a more uniform strength throughout, the contractor recommended injecting 60% of the binder (150 kg/m^3) at ½-inch penetration per revolution through the organic layer, followed by an increase in penetration rate to 5/8-inch per revolution below the organic layer. On the way back up, the binder flow was turned off until the organic layer was reached, when the remaining 40% of the binder was injected. The tool was then reintroduced into the column to remix the upper couple of meters. The contractor installed the trench support columns, which were not to be tested, using a binder loading of 150 kg/m^3 injecting 60% on the way down and 40% on the way up for the entire length of the column. Higher strengths are easily obtainable by increasing the binder-loading rate. However, to insure failure of the test section maximum shear strength of 40 psi was targeted.

The phase II columns to be loaded were arranged in panels with an overlap of 6 inches to inhibit a progressive failure of individual columns. (Broms, 1999) After Phase II columns were installed the area to be loaded was excavated down to the tops of the columns. The loading platform was not installed directly on top of the columns; rather alternating layers of soil and geotextile were used to create a load transfer pad. It was decided to load the columns with steel plates for a more uniform loading regime and therefore an easier back analysis of performance. Also the height of the load would have been at least three times as high using soil or concrete, creating an unsafe condition.

OBSERVATIONS

During Phase I of the test program, the Corps of Engineers staff was able to observe column installation, and so the pitfalls and unforeseen challenges associated with administering an actual construction contract to install dry mix columns. Observations included:

1. Columns of uniform strength were not practical to construct in the stratified foundations.
2. The centers of the columns were not well mixed, resulting in weak centers.
3. Subsurface roots presented no problem for the column installation.
4. The pitcher sampler produced better samples for laboratory testing than the triple tube sampler or the Shelby tube.
5. Of the QA measures taken, the pressuremeter gave the best results, although scatter of the data is significant. This test however lacks immediacy since the columns must be approximately three days old, before coring.
6. No increase in soil shear strength was observed 27 inches from the center of columns.
7. During penetration, binder flow was not turned on until the binder port was a meter below the ground surface; during withdrawal binder flow was shut off a meter below ground. This was in order to limit release of binder into the atmosphere. However, even with limiting binder flow to a meter below the ground surface, plumes of binder would exit the ground surface on completion of the column, creating a dust concern. During production this concern can be alleviated by placing three feet of levee embankment prior to installation of deep mixed columns.
8. There was no apparent correlation between the bench scale test results and the in-situ pressuremeter tests on the columns. Prediction of actual column strength could not be based on the bench scale data.
9. During excavation of Phase I columns it was noticed that the upper portion of column was of not fully formed. Faint outlines of the columns were observed, however the columns were no stronger than the original in-situ soil. It appears that the upper 4.5 to 6 feet of soil is too dry to provide enough moisture for cement hydration. The Liquidity Index of the in-situ soil as a measure of the available moisture was suggested. (Esrig, 1999) Liquidity Index is defined as:

$$LI = (w - PL)/PI \qquad (2)$$

Where LI= Liquidity Index
w = water content
PL= Plastic Limit
PI= Plasticity Index

Regardless of the water content, when the LI is less that 0.5, the available moisture is insufficient for hydration. It was suggested that although the water content of the soil was 40%, much of the pore water is not available to hydrate the binder when the liquidity index is below 0.5. (Esrig, 1999) Moisture content or piezometric level alone is not per se a guarantee that dry mixing can be conducted. A graphical representation of the Liquidity index vs. Depth for Boring TS-1U is presented in Figure 15. To create effective dry mix columns in soils with LI less than 0.5, some pre-wetting of the soil may be necessary to ensure that enough water is available for cement hydration.

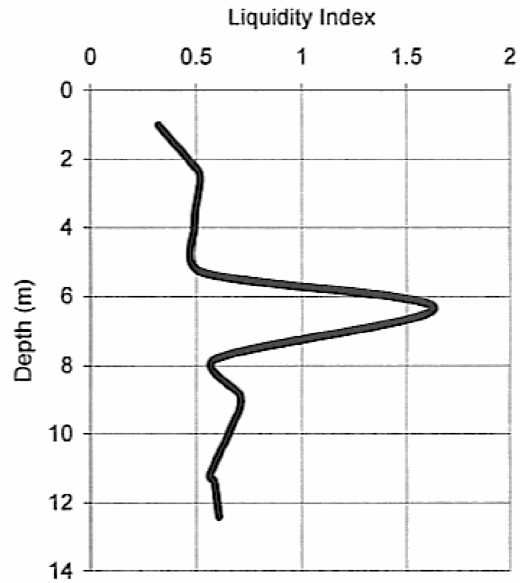


Figure 15 Liquidity Index vs. Depth

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