Rehabilitation of the Lower Chariton River Levee by Lime/Fly Ash Slurry Injection

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A field investigation was conducted to document the distribution of lime and fly ash (L/FA) seams and the resultant shear strength of an earth structure stabilized by L/FA slurry injection. The rehabilitation of the Lower Chariton River Levee Unit L-246 in Dalton, Missouri, was selected as the test site for this study. The study involved 56 hand vane shear tests and the mapping of L/FA seam coverage on 20 sample areas in seven test pits. Nine laboratory direct shear tests were conducted on undisturbed block samples from both treated and untreated areas. The results of unconfined compression tests performed by the U.S. Army Corps of Engineers, Kansas City District, are also described. The L/FA distribution pattern and surface coverage evaluation revealed that, on the average 1.6 percent of the natural soil surface area is composed of L/FA seams. Based on an analysis of the strength tests reported, it appears that the double injection of the L/FA slurry in the levee increased the overall average strength of the soil by 15 to 30 percent, depending on the measure of comparison. In evaluating the long-term effectiveness of this stabilization technique, it should not be anticipated that the volume change potential of natural soil untouched by the injected slurry will have been changed. However, the crack filling and mending and the improvement in overall average shear strength suggest that L/FA injections are an appropriate and economical technique for maintaining slope integrity.

Lime slurry pressure injections have been successfully used to stabilize and control swelling of expansive soils and low-strength clay soils to depths up to 40 ft. When the activity of the soils is somewhat lower, or when the soils have higher void ratios, lime/fly ash (L/FA) slurry mixtures have been injected to fill voids in an attempt to create stronger seams that help mend possible failure surfaces and increase the overall stability of a slope by waterproofing the soil mass. It has also been suggested that by filling the fractured structure of a soil mass with a slurry of L/FA, physicochemical reactions such as cation exchange, agglomeration, and flocculation with the adjacent soil are responsible for modification of soil properties (*1*).

The primary objective of this field investigation was to document the distribution of L/FA seams and the resultant shear strength of an earth structure stabilized by the L/FA injection. The rehabilitation of the Lower Chariton River Levee Unit L-246, in Dalton, Missouri, was selected as the test site for this study.

A total of 56 hand vane shear tests were performed in seven test pits. L/FA patterns and areas covered were quantified and documented for the three vertical sides and floor of the test pits. Undisturbed block samples from areas treated and not treated were recovered and taken to the laboratory at North Carolina State University (NCSU) where nine conventional direct shear tests were conducted. Table 1 presents the detailed program used to evaluate the shear strength of the Lower Chariton River right bank levee.

The U.S. Army Corps of Engineers, representing the owner, assisted this field investigation by providing relevant data on unconfined compression tests performed on undisturbed samples obtained from the MRLS L-246 levee unit.

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BACKGROUND

History and Condition of the Levee

The following information was provided by the U.S. Army Corps of Engineers, Kansas City District, as part of an internal

TABLE 1 SUMMARY OF STRENGTH TESTS CONDUCTED AT LOWER CHARITON RIVER RIGHT BANK LEVEE

HAND VANE SHEAR TESTS:

STATION	# of Penetrations at Each Station -1-	-2-	-3-	-4-	TOTAL
99+85 LS	2	2	2	-	6
100+15 LS	2	3	1	-	6
127+00 RS	-	-	-	6	6
128+50 LS	-	4	-	1	5
187+50 RS	3	1	4	2	10
229+50 RS	3	2	3	3	11
265+50 RS	2	2	2	6	12

DIRECT SHEAR TESTS ON UNDISTURBED SAMPLES:

STATION	TEST TYPE	SAMPLE DEPTH	SOIL TYPE	NUMBER TESTS
99+85	LUS	2.0	CH	3
100+15	LUS	2.0	CL	3
127+00	LUS	2.0	CL	3

NOTE:
LS: Land Side
RS: River Side
-1-: Left Wall of Test Pit
-2-: Front Wall
-3-: Right Wall
-4-: Bottom Floor
LUS: Lab Undisturbed "slow"

report on the conditions of the MRLS L–246 levee unit (*2*). The levee unit is located on the right bank of the Lower Chariton River, and forms part of the Missouri River tieback levee system. Figure 1 shows the location of the site.

The levee was completed in 1971. During construction of the levee, the contractor used uncompacted fill between Stations 100 + 00 and 192 + 00 and between Stations 235 + 00 and 280 + 00. Uncompacted fill was placed in approximately horizontal layers not exceeding 3 ft in thickness. The fill was spread, distributed, and manipulated during placement to the extent that individual loads of material deposited on the fill did not remain intact, thereby eliminating large open voids in the fill.

The levee conditions prior to the beginning of the L/FA stabilization and rehabilitation program in 1987 indicated that before 1983 the levee unit appeared to be in stable condition. However, since that time numerous slides occurred, mostly on the riverside. There were no obvious reasons for the slides starting to occur more than 12 years after the levee was constructed.

Investigations indicated that as much as 13,600 ft of levee was in unstable conditions. When this Corps of Engineers internal report was written in 1985, approximately 1,300 linear feet of levee had been weakened by slides. Figure 2 shows the typical slide geometry and levee profile.

As part of the Corps of Engineers investigation, 10 test pits were excavated to obtain samples, document exposed failure planes, and observe types and condition of the embankment materials. The observations determined that the major contributor to the failure of the levee, after more than 12 years of withstanding nearly every kind of weather and river stage, was the fact that the embankment was constructed of nearly uncompacted, saturated clays with over 75 percent montmorillonite content. The fat clays had undergone volume changes associated with desiccation, which created a full network of finite joints, seams, and cracks. The Corps of Engineers determined that any Chariton River flows above a 20-year frequency with a low Missouri River, or even a 10-year Chariton River flow with a 7-year Missouri event, could cause a complete failure of the flood protection.

Levee Rehabilitation Program

In November 1984, the Woodbine Corporation demonstrated the L/FA slurry injection process on a short test section in the Lower Chariton River right bank tieback, which was built between 1967 and 1971 and was experiencing slope stability problems similar to the typical stability problems of the levee unit in question. The embankment materials in the test section was considered to be representative of the embankment material in the areas to be rehabilitated.

The test section was performed in November 1984 on 200 ft of levee just north of Station 101 + 00, with 100 ft injected and the other 100 ft remaining as a control section. Within 6 months, the untreated 100-ft section slid again; however, the soil mass sheared off at the contact with the treated section, leaving the 100-ft injected section intact.

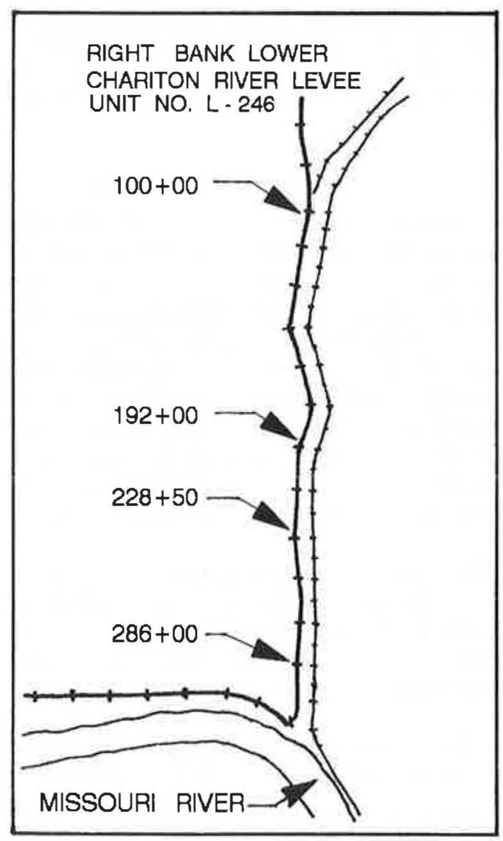


FIGURE 1 Site location.

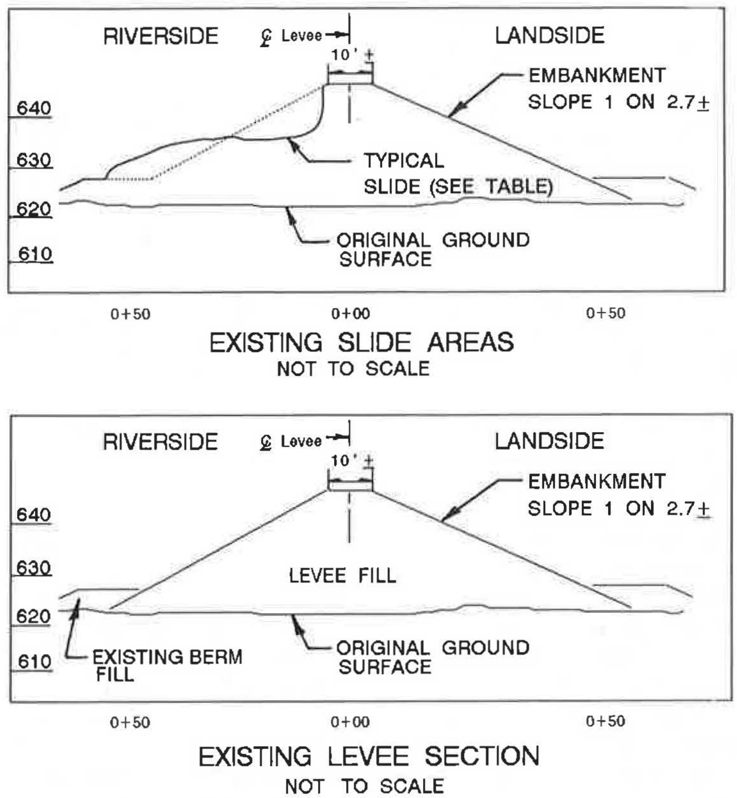


TABLE OF EXISTING SLIDE LOCATIONS

APPROXIMATE LOCATION STATION TO STATION		REMARKS
100+80	102+00	RIVERSIDE
100+75	104+20	LANDSIDE
229+05	230+10	RIVERSIDE
232+90	233+65	RIVERSIDE
234+45	235+30	RIVERSIDE
236+55	239+25	RIVERSIDE
264+10	268+85	RIVERSIDE
277+05	281+15	RIVERSIDE
284+60	285+40	RIVERSIDE

FIGURE 2 Typical levee slide and levee geometry.

According to the Corps of Engineers instructions, the L/FA slurry injection would stabilize the embankment in two ways (*3*):

> First, the injected slurry will penetrate any fractures, cracks or voids within the embankment, and form slurry seams. Pozzolanic reactions occur forming cementing compounds at the interface between the seams and the soil, thereby strengthening potential failure surfaces, sealing up fractures and cracks, and essentially waterproofing the embankment. Secondly, physicochemical reactions such as cation exchange, agglomeration, and flocculation, are responsible for a reduction in plasticity and swell characteristics. The soil is thereby stabilized by a modification of the soil properties, making it less likely to have the problems associated with large volume changes.

Inspection pits dug in the test area after the L/FA injection process revealed that the slurry had filled cracks that resembled circular arcs similar to shallow circular failure surfaces. It is not known whether the cracks were preexisting or caused by the high-pressure injections. There were also small, randomly located voids and cracks, in which there was little or no slurry penetration. Ideally, all these random cracks and voids should have also been filled with the slurry. It was believed that the use of injections at 50 to 75 psi, combined with the refusal criteria (L/FA slurry breaking the ground surface at the point of injection), did not allow full penetration of the slurry into small cracks and voids.

Woodbine Corporation began the stabilization of the Lower Chariton River right bank in October 1987. The L/FA slurry was made up of 4 lb of hydrated lime and fly ash solids to 1 gallon of water. Primary injections were set on a grid pattern of 5 ft on center. Because of the winter season, the job was stopped and resumed in April 1988. A set of secondary injections was performed 2.5 ft away from the primary injections also using the 5 ft on center grid pattern. The secondary injections used a L/FA slurry made of 6 lb of solids to 1 gallon of water. Both primary and secondary injection slurries were proportioned at a lime to fly ash ratio of 1 to 3 (*4*).

Injection Process

Because injection technology has been reported in the literature (*5,6*) only a brief description of the materials, handling, and slurry injection process is presented herein:

1. Calcium oxide (CaO) was converted into calcium hydroxide ($Ca(OH)_2$) at the job site in portable lime slaking tanks. The resulting slurry reached temperatures up to 195°F.
2. A portion of the slurry was then transferred to a second slurry tank where it was adjusted to the correct specific gravity by adding water. Addition of a retarder was required to prevent premature setting after mixing and prior to injection.
3. The slurry was then pumped into a jet slurry mixing valve where the dry fly ash was introduced. The resulting L/FA slurry was transferred directly into a deaeration-averaging tank before being pumped to the injection units. The specific gravity of the L/FA was continuously monitored, and immediate adjustments could have been made to stay within the targeted range, as shown in Table 2.
4. The L/FA slurry was pumped through hoses to the pressure pump trailer where a second deaeration tank mixed the slurry and maintenance of the design specific gravity was ensured prior to injection.
5. Survey crews set a grid pattern that showed the dozer operator where to position the probes. Penetration increments ranged between 12 and 18 in. depending on soil conditions. Depth of total penetration was specified to be 5 ft at the toe of the slope and 10 ft elsewhere.
6. Injection pressure was set to approximately 50 psi. The slope was injected from toe to crown. An assistant on the ground aided the dozer operator in determining increment depth, spacings, and alignment.

FIELD TEST PROGRAM

The field portion of the investigation involved the exploration of seven test pits using hand vane shear tests and mapping and quantifying L/FA patterns and distributions. The following section presents the details of the experimental work performed and the resultant findings.

Hand Vane Shear Test

Seven test pits were explored to document the spatial variation of shear strength. Table 3 presents a summary of uncorrected hand vane shear strength results obtained from individual test pits. The data are shown as being in one of the following three categories according to the location from which the data were obtained: (a) an area in the soil mass that did not show any presence of L/FA; (b) an area that showed a seam or column

TABLE 2 LIME/FLY ASH SLURRY INFORMATION

POUNDS LIME/FLY ASH PER GALLON H_2O	SPECIFIC GRAVITY	POUNDS PER GALLON SLURRY	POUNDS LIME/FLY ASH PER GALLON SLURRY	% SOLIDS BY WEIGHT
4	1.247	10.405	3.389	32.6
5	1.299	10.839	4.081	37.6
6	1.347	11.240	4.723	42.0
7	1.392	11.612	5.321	45.8
8	1.434	11.966	5.880	49.1

NOTE: Calculations based on:
1. One part hydrated lime to three parts fly ash
2. Slurry temperature 40°C (104°F)
3. Specific Gravity of Lime = 2.35
Specific Gravity of Fly Ash = 2.78

Source: Woodbine Corporation

of cured slurry from a secondary injection (recent injection at the time of testing, no more than a week old); and (c) within 8 in. of a primary injection or seam (6 months old).

Specifically, Columns 1 and 2 compare shear strength results in the intact soil and soil showing the presence of secondary L/FA injections. As noted, there is not a noticeable difference between the two. On the other hand, a comparison of these data with data in Column 3 from tests on soil adjacent to a 6-month-old primary injection point or a slurry seam, presented in Figure 3, indicate definite increases in shear strength within 4 in. of a seam or primary injection.

A further analysis of data where this noticeable change is more apparent (Test Pits 3, 6, and 7) reveal a trend toward decreasing shear strength with distance away from the primary seam or injection pipe, as shown in Figure 4. Shear strength values ranged from 2.8 ksf on the seam or primary injection, to 1.6 ksf at 2 in. away, and 0.6 ksf at more than 4 in. and the in remainder of the intact soil where no L/FA is apparent. These shear strength values correspond to net increases of 366 and 166 percent, respectively, over the shear strength of the intact soil (0.6 ksf).

L/FA Patterns and Distribution

In an attempt to quantify the frequency of L/FA-filled seams within the injected slopes, the side walls and floor of each test pit were carefully observed and photographed, and the evidence of slurry was documented. Figure 5 shows a group of pictures obtained from six test pits where the L/FA patterns were studied. The L/FA distribution was very irregular and not easily quantifiable. However, representative 2.5-ft by 2.5-ft areas on each of the side walls and floors were selected for analysis. Typically, the representative area was composed of slender seams and virgin soil. Seam thickness varied, but was generally less than 0.25 in. However, on one of the test pit walls (Station 100 + 15), L/FA coverage appeared over a large surface area, as shown in Figures 5a and 6. An analysis of the control area covered by the lime and fly ash indicated that approximately 0.3 ft^2 of the 6.25-ft^2 control area showed evidence of the L/FA. This represents an observed maximum coverage of 3.4 percent. However, the average coverage of the six treated test pits was approximately 1.6 percent.

TABLE 3 SUMMARY OF RESULTS FROM HAND VANE SHEAR TESTS AT LOWER CHARITON RIVER LEVEE

TEST PIT	LAND SIDE	RIVER SIDE	STATION (Water Content)	Vane Shear Strength (ksf) 1*	2*	3*
1	x		99+85	1.5		
			(w=33%)	1.4		
				1.4		
				1.2		
2	x		100+15			1.8(2")
			(w=22%)			1.8(2")
						1.7(2")
						2.0(2")
						1.9(2")
						1.8(2")
3		x	127+00	0.7	1.0	1.5(3")
			(w=27%)	0.9	1.0	1.4(3")
4	x		128+50	1.3	1.2	
			(w=20%)		0.8	
					1.3	
					1.4	
5		x	187+50	2.8	2.3	2.8(0")
			(w=28%)		2.3	2.8(4")
						2.8(2")
						2.8(2")
						2.8(2")
						2.8(2")
6		x	229+50	0.4	0.6	2.8(0")
			(w=40%)	0.6	0.4	2.8(4")
				0.5	0.4	
					0.5	
					0.7	
7		x	265+00	0.6	0.6	2.8(0")
			(w=27%)	0.5	0.7	1.2(2")
					0.7	0.6(4")
					0.8	0.6(4")
						0.9(3")
						0.8(3")

* Key:
1: Intact soil; No presence of L/FA; Distance greater than 5"
2: On secondary seam or pipe injection
3: Within 8" of primary injection seam or pipe; ( ) Distance

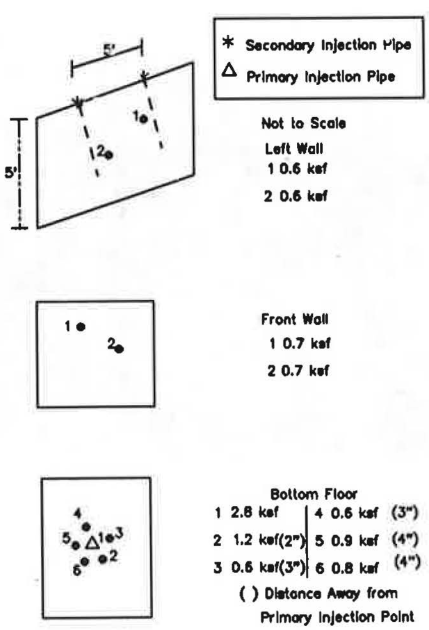


FIGURE 3 Hand vane shear results at Station 265+00.

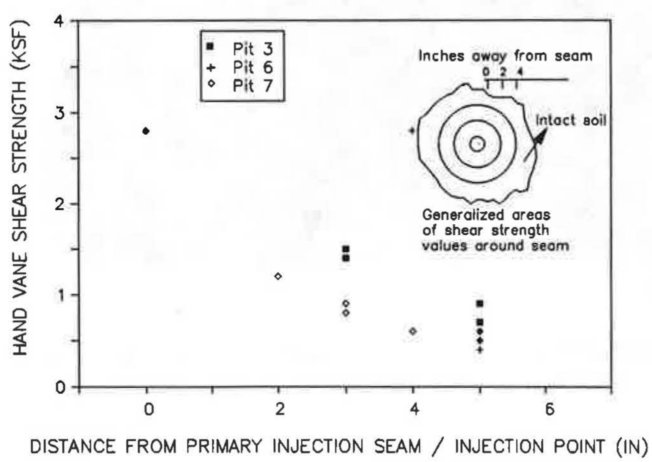


FIGURE 4 Vane shear strength as a function of location.

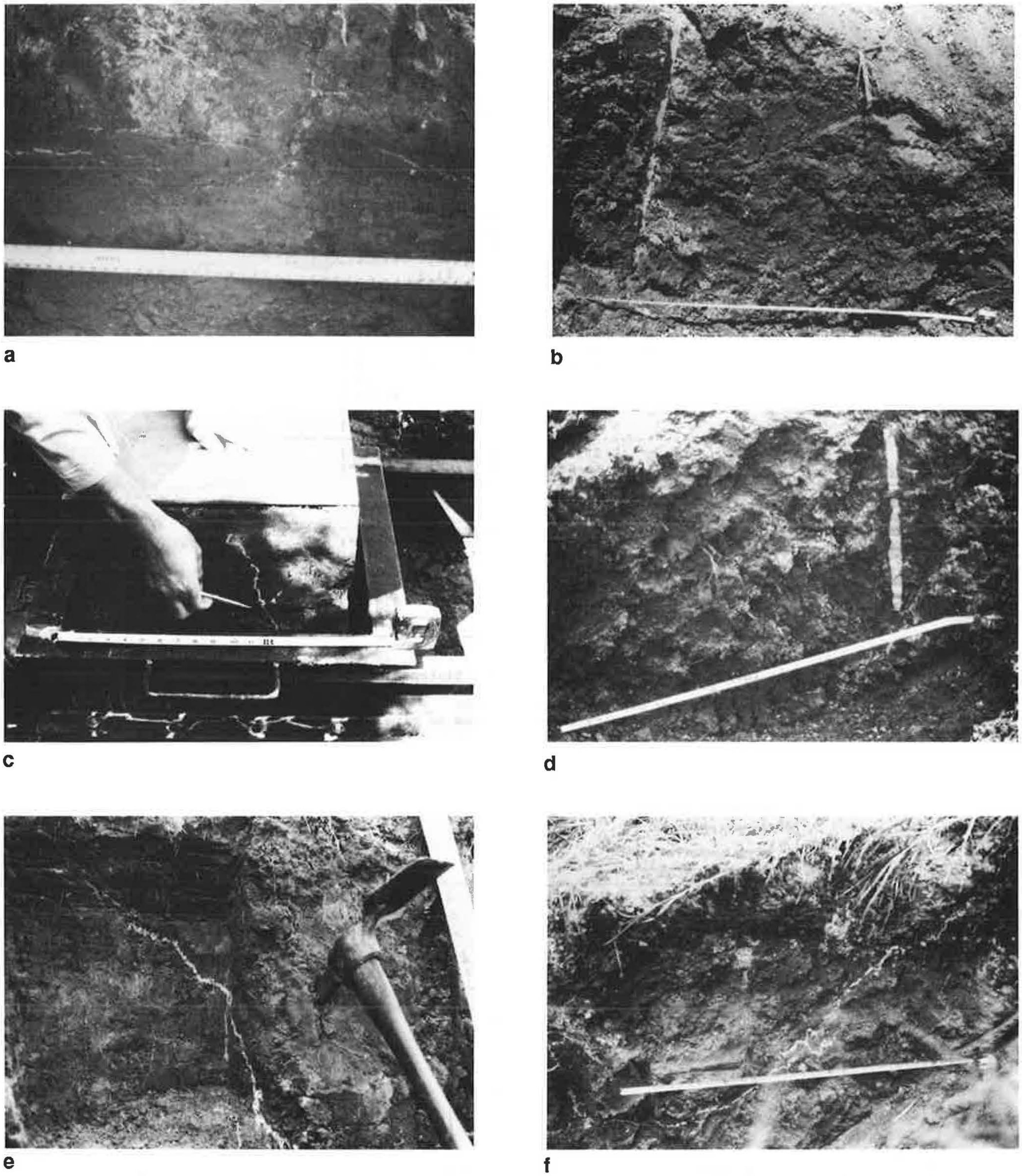


FIGURE 5 L/FA patterns at selected test pits. (a) Station 100 + 15, (b) Station 127 + 00, (c) Station 128 + 50, (d) Station 187 + 50, (e) Station 229 + 50, (f) Station 265 + 50.

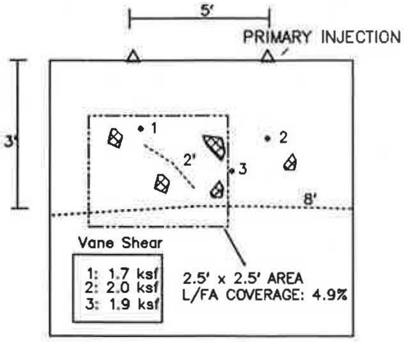


FIGURE 6 L/FA distribution on front wall of test pit at Station 100 + 15.

Table 4 shows the estimated L/FA coverage for each phase of the six test pits and several significant average values. The test pit at Station 187 + 50 was the only section where a tertiary injection was performed; it was also the test pit showing a significantly higher L/FA coverage on the side walls.

Field observations of L/FA slurry patterns showed that the slurry fills opening and fissures that are intersected by the penetrating injection pipe. Sometimes a fissure may be intersected by more than one injection pipe, and during injection the slurry may travel to a previously penetrated injection location and exit to the ground surface. The test pits also showed that some existing fissures do not come in contact with the L/FA slurry.

Effect of L/FA in the Overall Levee Shear Strength

If the 2.5-ft by 2.5-ft area of test pit is considered a sample representative of L/FA area coverage at the levee, and the pattern for shear strength values versus distance from a point of injection holds true, one could roughly predict the effect of L/FA injections in the overall shear strength of the mass in question. With these assumptions in mind, 1.6 percent L/FA area coverage, a weighted average of shear strength increase for 5 in. around the L/FA area coverage of 138 percent, and no shear strength increase beyond 5 in., the overall shear strength increase due to a double injection of L/FA is about 15 percent.

In summary, the results of hand vane shear tests from several test pits indicate that zones showing seams of L/FA do show corresponding increases in strength. However, the improvement is limited to the soil adjacent to the primary seam or injection location as the secondary injection occurred within 2 weeks before this investigation. Assuming that these factors hold true and that 1.6 percent of the area is covered by L/FA seams, the overall shear strength of the soil mass would increase by about 15 percent.

LABORATORY TESTING PROGRAM

Materials Tested

Soil samples obtained from the Lower Chariton River right bank levee were taken to the NCSU laboratory. Preliminary tests included grain-size analysis and plasticity indices. The majority of the tests indicated the presence of medium to highly plastic clays, as shown in the plasticity chart in Figure 7. Notice that many of the data plot near the U-line, indicating the presence of high-activity clay minerals (possibly montmorillonite).

Conventional Direct Shear Tests

Undisturbed block samples were recovered from Stations 100 + 15 and 99 + 85. The clayey block samples were obtained from a depth of 2.5 ft below the ground surface for both stations. From each station, three direct shear samples were tested in the laboratory at a displacement rate of 0.0005 in. per minute. Data obtained from the consolidation test on this sample denoted that a displacement rate of 0.0005 in. per minute was indicative of a "drained" test.

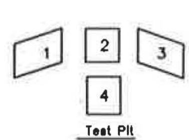
Figure 8 shows the average direct shear test results from three samples obtained at Stations 100 + 15 and 99 + 85. These results indicate that the soil from Station 99 + 85 is stiffer in

TABLE 4 L/FA COVERAGE IN 2.5 FT BY 2.5 FT AREA OF TEST PIT

STATION		1	2	3	4	Average
100+15	LS	1.0%	4.9%	1.9%	1.2%	2.3%
127+00	RS	---	1.2%	---	1.2%	0.6%
128+50	LS	---	1.4%	---	0.2%	0.4%
*187+50	RS	7.3%	1.8%	3.4%	1.2%	3.4%
229+50	RS	1.9%	1.8%	1.0%	1.4%	1.5%
265+50	RS	2.0%	1.2%	1.2%	0.2%	1.2%
	Average	2.0%	2.1%	1.3%	0.9%	1.6%

Legend:

LS: Land Side
RS: River Side
*: Section Reinjected
1: Pit Left Wall 3: Right Wall
2: Front Wall 4: Bottom Floor



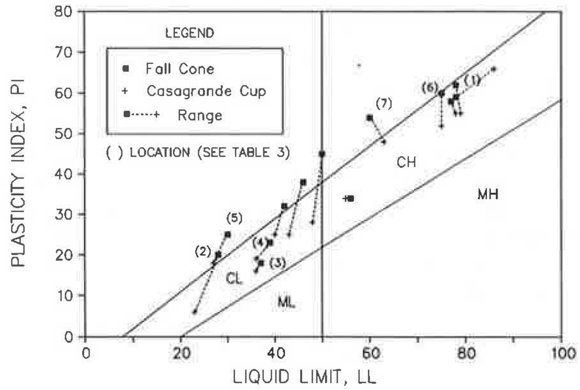


FIGURE 7 Plasticity chart showing range of soils obtained from Lower Chariton River right bank levee.

the first 0.1 in. of horizontal displacement, but that as the sample continues to be displaced, the maximum shear stress levels off. On the other hand, the soil from Station 100 + 15 (treated with L/FA) shows a continuous increase in shear strength until a peak is reached near a horizontal displacement of 0.25 in. The average maximum shear strength for Station 100 + 15 is about 1.05 ksf, whereas the average peak shear strength for soil from Station 99 + 85 is about 0.75 ksf. This finding represents a difference of 40 percent in favor of the treated soil. However, care must be exercised in this interpretation because the soil at Station 100 + 15 is both drier and denser than the soil from Station 99 + 85. Table 5 summarizes the pertinent data. The above direct shear results are valid for a vertical stress equal to 364 psf (the existing overburden stress at the average depth of the previously observed failure surfaces).

The results from the drained direct shear tests seem to indicate that the L/FA slurry is helping in some way. It is difficult to quantify the improvement provided by the addition

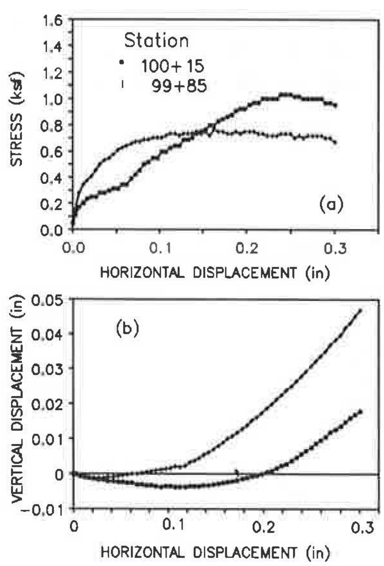


FIGURE 8 Average response of three direct shear tests from Stations 100 + 15 and 99 + 85.

TABLE 5 COMPARISON OF RESULTS ON UNDISTURBED SAMPLES

Location	Peak Shear Strength (ksf)	Dry Unit Weight (pcf)	Water Content (%)
STA 99+85	0.75	71	32
STA 100+15	1.05	83	21

of the L/FA slurry because of the lack of strength data at the exact location before the injection of the L/FA slurry.

Unconfined Compression Tests on Samples Obtained from the Levee Unit

The U.S. Army Corps of Engineers, Kansas City District, provided this research project with data obtained from test pits (control samples) as well as undisturbed samples obtained using a drill rig and 5-in. Shelby tubes. The record control (RC) samples contained lime and fly ash seams. There were no lime and fly ash seams in the undisturbed (U) samples. According to the Corps of Engineers, it was decided to use the 1:1 ratio because of the difficulty in trimming the samples to the conventional 2:1 length to diameter ratio. Unconfined compressive strengths obtained from these samples were used only as indicators and to aid in comparing the effect of L/FA presence in the samples.

The general classification of the soils indicated that they were brown lean to fat clays with liquid limits ranging from 45 to 90, and plasticity indices between 30 and 70. Dry unit weights ranged from 81 to 106 pcf for RC samples, and 75 to 90 pcf for the U samples. Percent saturation varied from 60 to 97 percent. Table 6 summarizes the results from the un-

TABLE 6 SUMMARY OF DATA FROM UNCONFINED COMPRESSION TESTS

Boring No.	Compressive Strength (tsf)	Dry Unit Weight (pcf)	Water Content (%)
RC-1	1.58	83	30.9
RC-2	2.52	103	15.3
RC-4	3.42	107	17.1
RC-5	1.20	82	23.7
RC-6	1.42	87	32.5
U-278	0.80	87	28.8
U-281	0.60	75	44.3
U-285	1.18	79	32.9
U-286	1.46	80	32.6
U-284	0.84	87	31.2
U-283	0.72	79	39.4
U-282	1.05	90	30.3
U-182	1.10	89	29.8
AVERAGE:			
RC	2.02	92.4	23.9
U	0.97	83.3	33.7
IMPROVEMENT RC/U	108 %		

NOTE:

RC = Record Control Sample (L/FA Present)
U = Undisturbed (No L/FA Present)

TABLE 7 SUMMARY OF DATA FROM UNCONFINED COMPRESSION TESTS: DRY UNIT WEIGHT 78–97 pcf, WATER CONTENT 22–37 PERCENT

Boring No.	Compressive Strength (tsf)	Dry Unit Weight (pcf)	Water Content (%)
RC-1	1.58	83	30.9
RC-5	1.20	82	23.7
RC-6	1.42	87	32.5
U-278	0.80	87	28.8
U-285	1.18	79	32.9
U-286	1.46	80	32.6
U-284	0.84	87	31.2
U-282	1.05	90	30.3
U-182	1.10	89	29.8
AVERAGE:			
RC	1.40	84	29.0
U	1.07	85.3	30.9
IMPROVEMENT RC/U	30 %		

NOTE:
RC = Record Control Sample
U = Undisturbed (No L/FA Present)

confined compression tests along with sample water contents and dry unit weights.

An evaluation of the tests performed by the Corps of Engineers indicates that RC samples showed that phenolphthalein reacted on the seams only (phenolphthalein is a color-sensitive pH indicator solution that can be sprayed on the soil to determine the presence of lime). This response suggests that there is little migration of L/FA components away from the seams. Initially one would conclude that unconfined compressive strength averages of RC samples appear to be 100 percent stronger than the average unconfined compressive strength of samples not showing any L/FA seams (U samples). However, there is a significant difference in both average dry unit weights and water contents. Together, these factors may have contributed significantly to the observed difference in unconfined compressive strength.

A more realistic evaluation of strength improvement would be obtained if strength data from U and RC samples with comparable water contents and dry unit weights are chosen. Therefore, samples with dry unit weight values between 78 lb/ft^3 (pcf) and 97 pcf, as well as water content values between 22 and 37 percent, were chosen because they represent the average of all tests plus or minus one standard deviation. Table 7 shows the data that fall within this range and the computed averages and strength improvement between soil samples with L/FA (RC) and without (U). The resulting strength improvement is now about 30 percent.

SUMMARY AND CONCLUSIONS

The hand vane shear results provided a quick and reliable way of assessing differences in shear strength in all of the test pits. The analysis of lime and fly ash area coverage provided a feeling for the potential distribution of the injections into a fractured lean to medium plasticity clay. An evaluation of the excavated test pits indicated that the L/FA seams vary in length as well as thickness. This study showed that the distribution on a horizontal plane is similar to that on a vertical plane. As a result, the study of the L/FA coverage indicated that on the average, 1.6 percent of a natural soil area was covered by seams of lime and fly ash due to a double-injection system.

Furthermore, the analysis of hand vane shear strength results showed the existence of a relationship between shear strength and distance away from a 6-month-old L/FA seam or injection point. However the shear strength increase was apparent only in the near vicinity (less than 5 in.) of the L/FA. Nevertheless, it was estimated that the presence of this higher-strength area improved the overall soil shear strength of the levee in question by about 15 percent.

Laboratory direct shear strength data for the soils in the stabilized zone and unstabilized zone indicated an average strength increase of 40 percent in favor of the treated soil. However, care must be exercised in this interpretation because the treated soil was both drier and denser than the unstabilized soil. In addition, the results from unconfined compressive strength tests on samples with the presence of L/FA seams appear stronger in compression by about 30 percent.

Laboratory and field shear strength data seem to suggest a definite improvement from the addition of L/FA injections; both data correlate well with certain exceptions and qualifications. The tests suggest that the double injection of L/FA at the levee improved the overall average shear strength characteristics of the soil between 15 and 30 percent.

In evaluating the long-term effectiveness of this stabilization technique, it should not be anticipated that the volume change potential of natural soil untouched by the injected slurry will have been changed. However, crack filling and mending and the improvement in overall average shear strength suggest that L/FA injections are an appropriate and economical technique for maintaining slope integrity. Both the test section, completed in November 1984, and the levee rehabilitation, completed in April 1988, have performed well to date. In contrast, the untreated portion of the test section failed within 6 months and several portions of the levee not stabilized in the major program have experienced sloughing failures during the past 2 years.

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