Research Report
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LOW-STRENGTH (POZZOLANIC) MATERIALS FOR HIGHWAY CONSTRUCTION

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INTRODUCTION

NATURE OF POZZOLANIC MATERIALS

The use of pozzolans in cementing materials antedates recorded history. Ancient Egyptians used a cement composed of calcined impure gypsum. The Greeks and Romans used calcined limestone and later developed pozzolanic cements by grinding together lime and a volcanic ash.

The material added to hydraulic cements by the Romans to improve quality was a loosely consolidated rock of volcanic origin, consisting of various fragments of pumice, obsidian, feldspar, pyroxines, quartz, etc. The name **pozzolana** was first applied to that material; however, the term has been extended to include not only natural volcanic materials, but diatomaceous earths and other highly siliceous rocks and artificial products. Pozzolans are defined as siliceous materials, even though not cementitious in themselves, contain constituents that will combine with lime in the presence of water at ordinary temperatures to form compounds that possess cementing properties.

Naturally occurring pozzolans include clays and shales, opaline materials, and volcanic tuffs and pumicites. Some pozzolans require calcination to make them active. Others may or may not require calcination to be active as a pozzolan. Most natural (and artifical) pozzolans require grinding to a high degree of fineness to make them suitable. Artifical pozzolans come from industrial byproducts or wastes and include fly ash (flue dust), silica fume, powdered brick, burnt clays and shales, and some slags.

USES OF POZZOLANIC MATERIALS

Until recently, the use of pozzolanic materials in highway and street construction in Kentucky was not often economically competitive with abundant supplies of high-quality aggregates. Pozzolanic bases have been used primarily in low-volume traffic situations. Mixtures that have been considered recently and evaluated to some degree include the following:

- Lime kiln dust, fly ash, and dense-graded aggregate;
- By-product lime and dense-graded aggregate;
- Lime kiln dust, fly ash, dense-graded aggregate, and sand;
- Lime kiln dust, fly ash, and limestone mine screenings; and
- "Scrubber sludge," quicklime, and dense-graded aggregate or pond ash.

KENTUCKY STUDIES

Research relative to the use of "conventional" pozzolanic subbase and base mixtures have included the following:

- Comparison of unconfined compressive strengths for various mixture proportions,
- Evaluation of effects of laboratory curing conditions on unconfined compressive strengths,
- Evaluation of the effects of curing on field deflection measurements,
- Development of alternate pavement thickness designs, and

- Development of construction and quality-control specifications.

Ultimate compressive strengths for mixtures have been determined as a function of the proportions of the various components. Compressive strengths have been obtained for various ages. Laboratory curing was varied from 100 F to room temperature for 7 to 28 days. Some specimens were cured in sealed containers whereas others were cured in open air. Deflection measurements using a Road Rater were obtained for a number of field sites. Placement conditions varied from spring to summer to fall. Weather conditions during placement varied from cool and wet to hot and dry to cold and dry. Young's moduli of elasticity, Poisson's ratios, and ultimate unconfined compressive strengths are being used to develop thickness designs for low-volume city streets.

FIELD STUDY

Construction of an experimental project utilizing a mixture of scrubber sludge, quicklime, and pond ash from a power plant in Western Kentucky (Sebree Bypass) is anticipated for the spring of 1984. Trial mixtures to determine ultimate unconfined compressive strengths, moduli of elasticity, and Poisson's ratios have been evaluated in the laboratory. That information was used in combination with elastic theory (the Chevron N-layer computer program) and a limiting strain criterion to develop pavement thickness designs with a scrubber sludge - pond ash mixture as a subbase.

EXPERIENCES OF OTHERS

Pozzolanic bases have been used for some 20 years by a number of highway agencies. That utilization included hydrated lime or by-product lime, fly ash, and aggregate mixtures. Perhaps the most extensive use has been in Illinois where pozzolanic base is bid as an alternative to bituminous stabilized and cement stabilized bases. The typical Illinois mix contains 3.5 percent lime and 9 to 10 percent fly ash. Although limited to low-volume roads initially, pozzolanic bases are being used on higher-type facilities. Experience in Illinois, as well as Ohio and Pennsylvania, confirmed that warm temperatures are necessary for favorable curing and strength gain. Generally, a seasonal cutoff date of September 15 is specified. A compaction level of 100 percent of AASHTO T 99 (standard compaction) was considered minimum; Illinois specifies 97 percent of AASHTO T 180 (modified compaction). Pozzolanic bases are generally permitted only where reflective cracking can be tolerated or where a crack-sealing program is planned.

POZZOLANIC SUBBASE MIXTURES

MATERIALS

Principal pozzolanic base and subbase mixtures evaluated in the laboratory included fly ash, hydrated lime, quicklime, lime kiln dust, scrubber sludge, and aggregate.

Scrubber sludge is a waste material obtained with the use of scrubbers to remove fly ash and residue from the coal-burning processes of electric generating power plants such as the Big River Electric Corporation generating station in Sebree, Kentucky. The scrubber sludge consists of fly ash and a lime dust slurry filter cake material (flue gas desulfurization sludge) consisting of calcium sulfate and calcium sulfite. Quicklime is added to the sludge for stabilization. The fly ash is silt-size spherical particles 0.015 to 0.050 mm in diameter. Typical ash properties are shown in Table 1.

Stabilization reactions begin almost immediately after the combination of fly ash and lime to the dewatered sludge. The resulting stabilized compound is ettringite ($3 CaO.Al_2O_3.3 CaSO_4, 32 H_2O$).

SPECIMEN PREPARATION

Specimens were prepared in general accordance with ASTM C 593(79) in 4-inch diameter by 4.6-inch molds. Deviations from that method involved the use of a 5-pound hammer and a 12-inch free fall instead of the specified 10-pound hammer and 18-inch drop. Moisture-density relationships were determined in accordance with ASTM D 698(79) instead of ASTM D 1557(79).

Initial mixtures contained high percentages of fine particles, and compaction procedures were varied from those specified in ASTM C 593(79), which are more applicable to coarse mixes. Even though subsequent specimens involved coarser mixes, compaction techniques were kept constant so direct comparisons of engineering properties could be made.

TABLE 1. TYPICAL ASH PROPERTIES OF COALS USED AT ROBERT REID STATION, BIG RIVERS ELECTRIC CORPORATION

CONSTITUENT	PERCENTAGE BY WEIGHT
Silica	39 - 44
Aluminum and Iron Oxide	40 - 61
Calcium Oxide	4.5 - 6.0
Magnesium Oxide	2 - 3
Sulfate	0.5 - 5.0
Sulfite	0.2 - 1.0

UNCONFINED COMPRESSIVE STRENGTHS

Ultimate compressive strengths (Table 2) were determined using ASTM C 593(79) and ASTM C 39(79) procedures. A series of dial guages were located at third points along the mid-height circumference of each specimen. As the specimen was loaded, measurements of lateral strain were obtained from the three dial guages. The average lateral strain for each load increment was determined. Axial strain also was determined using a dial guage attached to the load plate.

Relationships between axial strain and stress and between average lateral strain and stress were determined using curve-fitting techniques. Poisson's ratio was calculated as the ratio of the maximum slopes of the lateral-strain curve to the axial-strain curve. Young's modulus of elasticity also was estimated from the maximum slope of the stress-axial strain curve (see Figure 1).

It is apparent from data in Table 2 that mixtures of by-product lime were somewhat weaker in terms of unconfined compressive strength and modulus of elasticity when compared to mixtures using fly ash. That may not necessarily be detrimental. Additional thicknesses of a weaker material normally would be required for designs equivalent to those consisting of materials having greater load-carrying capabilities. The feasibility of using specific mixtures (i.e., two-component versus three- or four-component mixtures) may be dependent upon individual project specifics such as availability of materials and transportation costs.

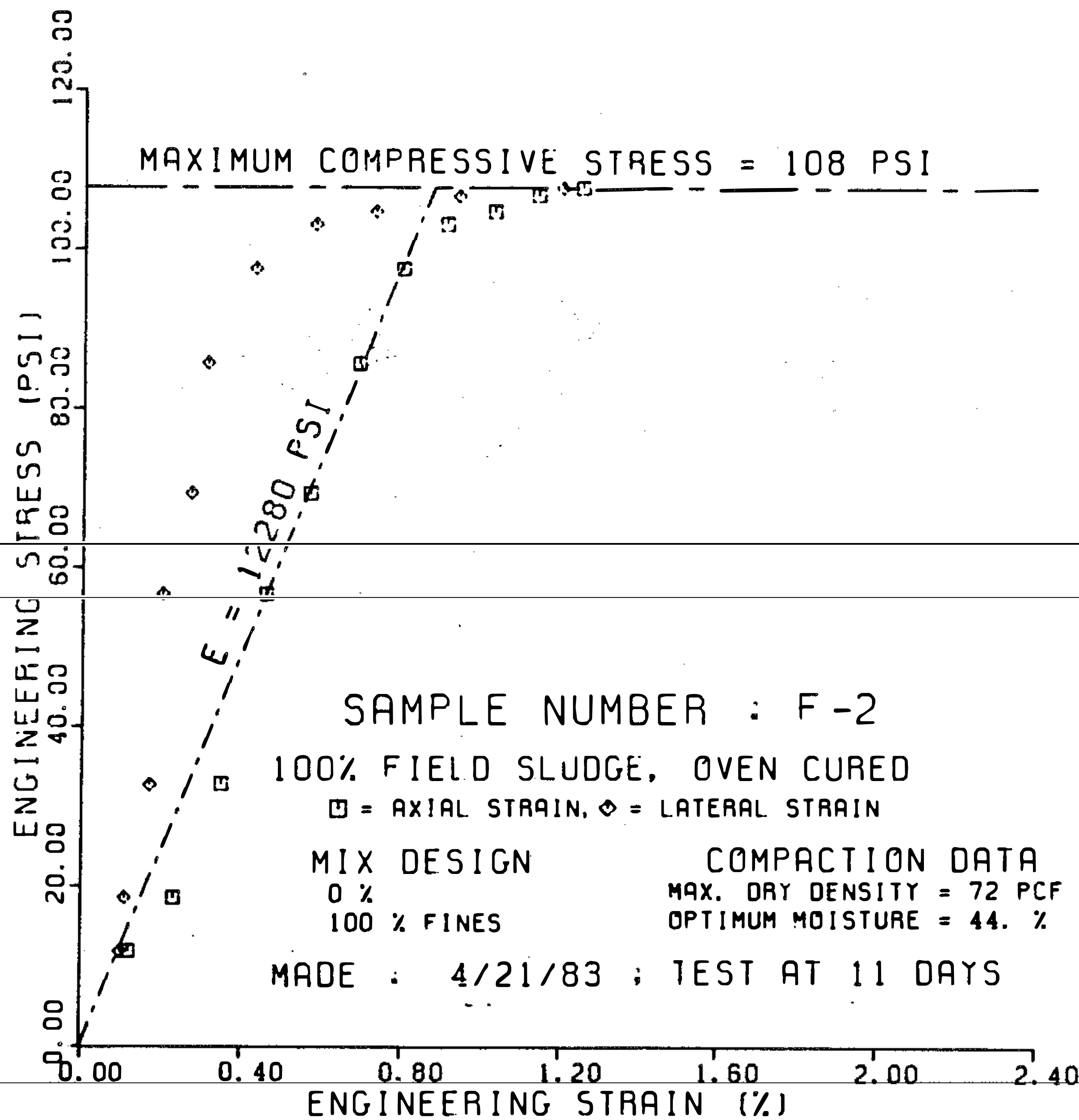

Figure 1. Example Determination of Modulus of Elasticity.

TABLE 2. UNCONFINED COMPRESSIVE STRENGTHS FOR VARIOUS POZZOLANIC MIXTURES AND FOR VARIOUS CURING CONDITIONS

MIXTURE COMPONENTS (percent)								UNCONFINED COMPRESSIVE STRENGTHS (psi)					
								CURING CONDITIONS (a)					
FLY ASH	LIME KILN DUST	BY-PRODUCT LIME	RIVER SAND	DENSE-GRADED AGGREGATE	DATA SOURCE	OPTIMUM MOISTURE CONTENT (percent)	MAXIMUM DRY DENSITY (lb/cu ft)	No. 1	No. 2	No. 3	No. 4	No. 5	No. 6
8	8			84	KyDOH*	7.6	141.7	1,161		70			384
					KTRP**	7.1	137.8	1,766					
					KTRP	7.5	139.4	1,577	2,829			898	231
					Average	7.4	139.6	1,501	2,829	70		898	308
5	5			90	KTRP	7.5	139.2	1,403	1,526		176		
					KTRP	6.5	141.9	1,390			279	280	
					Average	7.0	140.6	1,397	1,526		228	280	
8	4			88	KTRP	6.9	142.1	1,116					
8	6			86	KTRP	8.1	150.8	1,290					
		12		88	KTRP	6.5	142.1	646					
		16		84	KTRP	7.3	140.6	636					
		20		80	KTRP	6.8	135.8	315					
8	8		42	42(b)	KyDOH	7.1	133.5	1,102	69	439			
					KTRP	7.3	133.7	1,285					
					Average	7.2	133.6	1,194	69	439			
8	8		10	74	KTRP	7.5(c)	141.0(c)	1,255	280(f)	134	136(g)		
8	8		25	59	KTRP	7.6(d)	138.7(d)	1,272	356(f)	82	123(g)		
8	8		50	34	KTRP	7.1(e)	135.6(e)	923	157(f)	105	79(g)		

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a. Curing Conditions
  No. 1 -- 7 days at 100 to 110 F in a sealed container (ASTM C 593(79))
  No. 2 -- 7 days at 100 to 110 F in a sealed container and then 7 seven days at room temperature in air
  No. 3 -- 7 days at room temperature in a sealed container
  No. 4 -- 14 days at room temperature in air
  No. 5 -- 21 days at room temperature in a sealed container
  No. 6 -- 28 days at room temperature in a sealed container
b. Limestone run of mines aggregate
c. Specimen Conditions: 7.4 percent moisture and 139.2 lb/cu ft density
d. Specimen Conditions: 7.4 percent moisture and 139.3 lb/cu ft density
e. Specimen Conditions: 7.2 percent moisture and 133.6 lb/cu ft density
f. Same as Curing Condition No. 1 except in an unsealed container
g. Same as Curing Condition No. 3 except in an unsealed container

Another aspect associated with low-strength pozzolanic base materials involves reflective cracking of asphaltic concrete surfacing over the base. It is anticipated that greater amounts of cracking during curing will occur in higher-strength pozzolanics. Thus, the potential for reflective cracking may be significant.

AUTOGENOUS HEALING

A series of lime kiln dust - fly ash - dense-graded aggregate mixtures were prepared in 6-inch diameter by 12-inch cylinders and cured at room temperature for 28 days. There were significant variations in compressive strengths that were attributed initially to variations in laboratory curing conditions. The results stimulated additional investigations of curing effects and will be discussed below. The 6- by 12-inch cylinders were not destroyed after compressive testing, but were sealed in plastic bags and curing was allowed to continue. A comparison of compressive strengths before and after initial testing is presented in Table 3.

Autogenous healing apparently occurs in pozzolanic base specimens if left undisturbed and curing conditions remain favorable. However, conditions in the field may not be duplicated by laboratory conditions. Autogeneous healing of cracks in field installations may be slowed by the stressing of traffic loadings. Field curing conditions (temperature and moisture) also may vary considerably.

EFFECTS OF CURING CONDITIONS

Curing conditions were varied in the manner indicated in Table 2. A summary of unconfined compressive strengths are presented in Table 2.

Effects of curing also were detected in the field by deflection measurements. Table 4 presents a summary of deflection data obtained directly on a 6-inch layer of lime kiln dust - fly ash - dense-graded aggregate base for three city street projects. Design proportions for Site 1 and Site 2 were the same: 8 percent lime kiln dust, 8 percent fly ash, and 84 percent dense-graded aggregate. Design proportions for Site 3 were 6 percent lime kiln dust, 6 percent fly ash, and 88 percent dense-graded aggregate. Field deflection measurements were obtained at similar ages for all sites: 7 to 9 days after placement.

Other test data indicated subgrade conditions were similar for the three projects. The California Bearing Ratio (CBR) ranged from 2 to 5. Deflection testing of adjacent streets and back calculation to estimate subgrade strengths for two projects resulted in subgrade moduli similar to those obtained from laboratory CBR testing of subgrade on the third site.

Site 1 was placed in mid August and curing conditions were very favorable -- temperatures ranged from 60 F to 80 F and the bituminous curing membrane was in good condition. Site 2 was placed in early November when air temperatures were much cooler (40 F to 60 F). The bituminous curing membrane was not placed immediately after compaction. Site

TABLE 3. EVALUATION OF AUTOGENOUS HEALING

MIXTURE NUMBER*	TEST	AGE (days)	UNCONFINED COMPRESSIVE STRENGTH (psi)
1	Initial	28	231
	Final	240	870
2	Initial	28	208
	Final	240	1,367

* Mixture 1
- 8% fly ash
- 8% lime kiln dust
- 84% dense-graded aggregate
- 7.5% optimum moisture content
- 139.4 lb/cu ft maximum dry density

Mixture 2
- 8% fly ash
- 8% lime kiln dust
- 42% river sand
- 42% limestone run of mines
- 7.3 % optimum moisture content
- 133.7 lb/cu ft maximum dry density

Cured at room temperature (68 to 73 F) in a sealed plastic bag

TABLE 4. ROAD RATER DEFLECTIONS ON 6-INCH POZZOLANIC BASES

PROJECT NUMBER	DEFLECTIONS (inches x 10^{-5}) SENSOR NUMBER No. 1	No. 2	No. 3
1	53.8	29.8	15.1
2	118.5	46.0	24.8
3	147.2	56.9	24.3

3 was placed in early May. Air temperatures were unseasonably cool and rainfall was record setting. Site 3 was drenched immediately after placement of the bituminous curing membrane, and the membrane was "washed" away in some locations. In those areas, the surface of the base course was unbound or poorly bound. The site also was subjected to significant rainfall during the initial 7-day curing period.

It is apparent from the deflection data that lower strengths resulted as the curing conditions deteriorated. Deflection data also indicated the significance of the bituminous curing membrane.

Both laboratory and field data indicated that high temperatures and moisture retention are primary contributors to good curing and associated gains in strength. Thus, placement of pozzolanic base materials is recommended when air temperatures are expected to be greater than 60 F for at least 7 days. Placement of a bituminous curing membrane is apparently essential for the development of high early strengths.

CONSTRUCTION SPECIFICATIONS

The Kentucky Department of Highways has proposed specifications for the construction of pozzolanic bases; the specifications have been utilized as a guide on an experimental project (approximately 10 inches of pozzolanic base was placed).

The proposed specifications cover the production of pozzolanic base utilizing either hydrated lime or kiln dust (by-product lime or other approved kiln dust based on an acceptable service record). The fly ash must meet ASTM C 593 requirements, excluding the portion on plastic mixes in Section 7. Additionally, the loss on ignition shall be 10 percent or less as determined by ASTM C 311. Aggregate may be crushed stone, gravel and sand, or slag meeting the Kentucky Department of Highways standard specifications and addenda thereto for dense-graded aggregate base. The contractor proposes mixture proportions for approval by the Department on a project-by-project basis. Compressive strengths at 7 days of both laboratory- and plant-mixed materials must average 400 psi, with no single specimen having a strength less than 300 psi.

Placing and compacting of base mixtures must be within 4 hours of mixing. A maximum lift thickness of 10 inches is permitted. Compaction to 100 percent of laboratory density (AASHTO T 99) is required. The moisture content shall be maintained near the optimum (AASHTO T 99) until a succeeding lift or a bituminous cure coat is applied. Normal requirements for protection and maintaining of the newly constructed base are specified. Seasonal limitations of April 15 to September 15 apply, and placement of base is not permitted when the air temperature is below 40 F. The base must be covered by at least one lift of the succeeding pavement layer prior to the winter months or opening to traffic. The fully constructed base is measured and paid on the basis of the weight of the accepted base mixture; water and bituminous cure coat are considered incidental.

Based on recent information, it appears that the specifications will require adjustment in the proportioning and mix design acceptance requirements. Also, it appears that the minimum strength requirements may be low.

POWER-PLANT WASTES FOR HIGHWAY CONSTRUCTION

Two applications relative to the use of scrubber sludge in highway construction have been studied. One application involves the use of scrubber sludge as embankment material. The second involves a mixture of scrubber sludge, quicklime, and aggregate as a subbase material in pavement construction. Two aggregate materials were investigated: conventional limestone dense-graded aggregate and pond ash from the coal-burning process.

LABORATORY TESTING OF SCRUBBER SLUDGE AND POND ASH OR AGGREGATE MIXTURES

Quicklime was used to stabilize the aggregate or pond ash and sludge mixtures. Laboratory testing of scrubber sludge and aggregate mixtures were similar to procedures used with other pozzolanic mixtures.

Some specimems slaked when immersed in water prior to testing for compressive strengths. It was hypothesized that those problems were related to blending of dewatered components of scrubber sludge in the laboratory. Therefore, "raw" sludge was obtained and blended with pond ash; slaking was less pronounced. A summary of unconfined compressive strengths and Young's moduli are presented in Table 5.

THICKNESS DESIGNS

The Chevron N-layer computer program was used to first evaluate strain characteristics of current typical designs for low-volume situations. Laboratory test data were used as input for computer analyses of a matrix of pavement thicknesses using pozzolanic subbases and bases and asphaltic concrete surface courses. A limiting strain criterion similar to that used in the development of current Kentucky flexible pavement design curves was developed. Designs were developed to accommodate variations in unconfined compressive strength and modulus of elasticity (Table 5) of pozzolanic materials.

Two structurally equivalent thickness designs were determined for the Sebree Bypass using Kentucky's 480-ksi flexible design curves. The first design consisted of 8.5 inches of asphaltic concrete on 17 inches of dense-graded aggregate; the second design was 13.5 inches full-depth asphaltic concrete. Those two designs are represented by points on the line labeled 0 inches of scrubber sludge - pond ash in Figure 2 and correspond to a work strain of 0.000288 at the top of the subgrade. The other lines for various thicknesses of scrubber sludge in Figure 2 represent equivalent designs for various thicknesses of scrubber sludge substituted for either dense-graded aggregate or for a combination of dense-graded aggregate and asphaltic concrete. Figure 2 was developed for a specific construction project and represents design curves for one fatigue value (EAL = 7,600,000) and for one subgrade support

TABLE 5. AVERAGE UNCONFINED COMPRESSIVE STRENGTHS AND ELASTIC MODULI (Scrubber Sludge - Aggregate Mixtures)

MIXTURE COMPONENTS (percent)								
SCRUBBER SLUDGE	AGGREGATE AMOUNT	AGGREGATE TYPE	OPTIMUM MOISTURE CONTENT (percent)	MAXIMUM DRY DENSITY (lb/cu ft)	UNCONFINED COMPRESSIVE STRENGTH (psi) 7 DAYS	UNCONFINED COMPRESSIVE STRENGTH (psi) 28 DAYS	ELASTIC MODULUS (psi) 7 DAYS	ELASTIC MODULUS (psi) 28 DAYS
10	90	Pond Ash	10.3	150.5	94	826	8,112	77,471
15	85	Pond Ash	11.2	151.4	160	646	10,285	59,187
20	80	Pond Ash	11.0	132.9	196	617	15,512	55,834
10	90	DGA	9.9	133.7	153	286	7,159	26,124
15	85	DGA	10.9	130.6	189	275	7,787	17,700
20	80	DGA	11.8	124.9	168	254	10,080	17,576
10	90	Pond Ash	11.8*	126.2*	186	557	14,453	83,856
15	85	Pond Ash	9.5*	143.6*	309	670	24,185	37,526
20	80	Pond Ash	11.7*	133.5*	264	560	18,067	29,029
30	70	Pond Ash	12.4*	128.3*	211	393	14,302	58,306

*As compacted

All specimens cured at 100 F for 7 days and then in air (room temperature) for 21 days

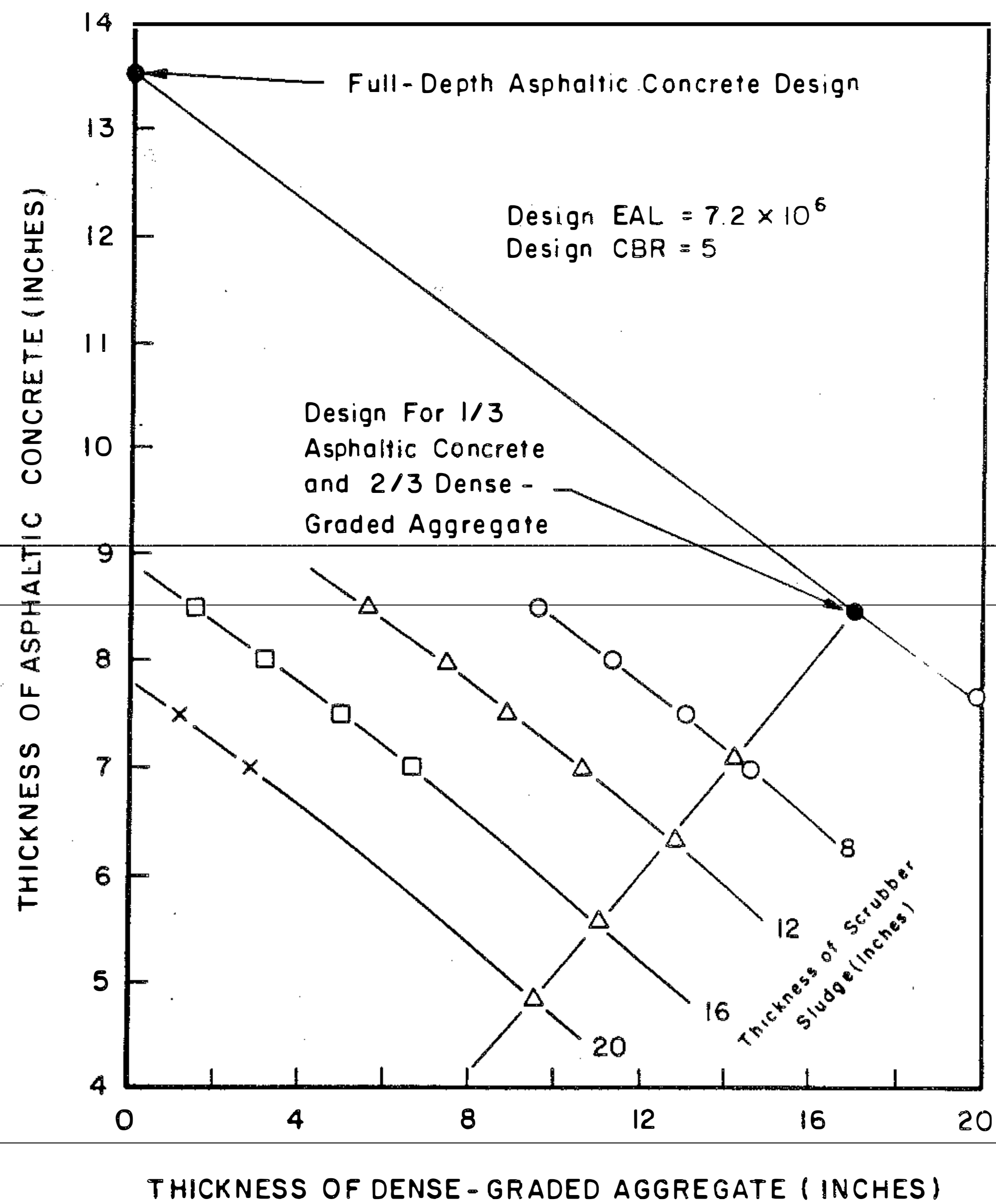

Figure 2. Thickness Design Curves Utilizing Scrubber Sludge and Pond Ash-Type Materials.

(CBR = 5). However, similar relationships can be determined for a range of fatigue loading levels and subgrade support values.

EMBANKMENT ANALYSES

Nine isotropically consolidated-undrained triaxial tests with pore-pressure measurements were run on the scrubber sludge (three tests on each of three mixtures). The specimens were 4 inches in height and 2 inches in diameter. The specimens were compacted at optimum moisture using the compactive effort of ASTM D 698(79) and immediately placed in the test chamber without curing. The specimens were then consolidated overnight under the chosen confining pressure. After consolidation, the specimens were loaded to failure (time of test approximately 20 hours) at an average strain rate of 0.014 percent per minute.

The specimens apparently continued to hydrate while in the test chamber. Very little volume change occurred during consolidation of the specimen, indicating a stiff specimen. Also, stress-strain curves (see Figure 3 for an example) were very "rough" after reaching a peak (failure). That indicated a number of localized brittle failures and slips, which is typical of stiff materials.

Figure 4 is an example of the effective stress paths. The internal friction angle can be calculated from $\phi' = \arcsin(\tan \alpha)$, in which α is defined as illustrated in Figure 4. Cohesion, c', is the y-intercept of the K_f-line, d, divided by the cosine of ϕ'. The ϕ' values were 41.8°, 40.5°, and 40.7° for Mixtures 1, 2, and 3, respectively.

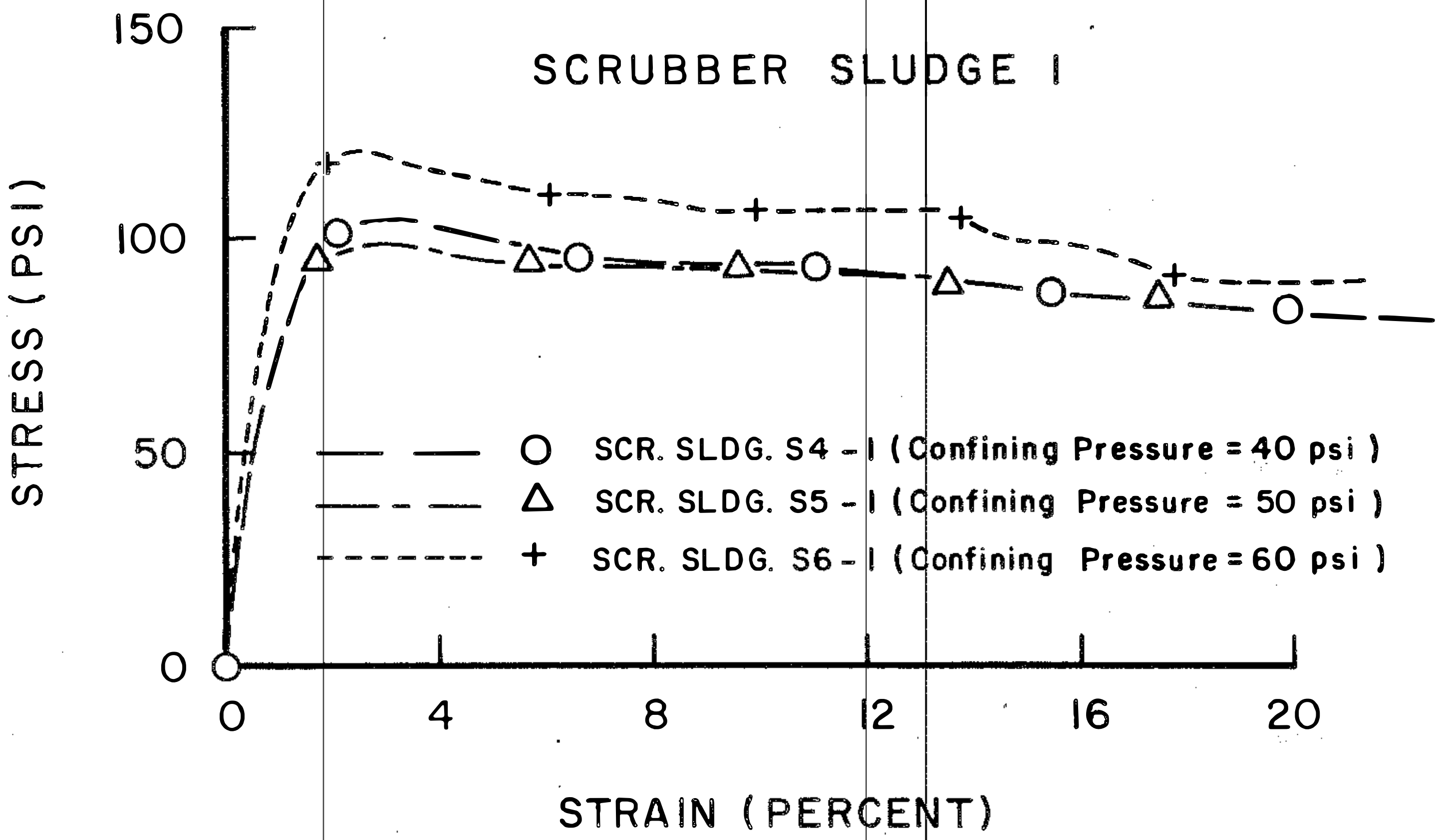

Figure 3. Stress-Strain Curves -- Mixture 1.

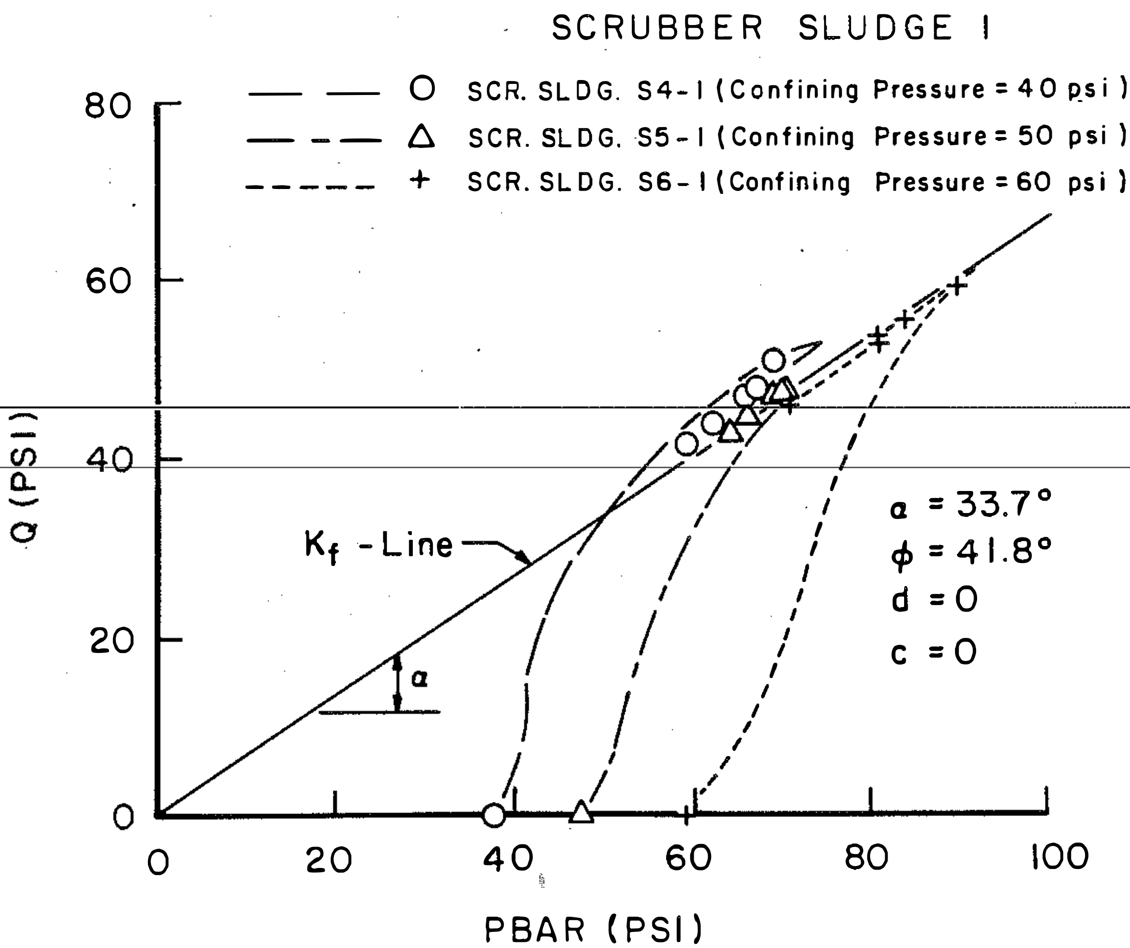

Figure 4. Effective Stress Paths -- Mixture 1.

Cohesion values were 0, 7.1 pounds per square inch, and 5.8 pounds per square inch, respectively.

To determine the effect of moisture content on shear strength properties, three triaxial tests were run on specimens compacted approximately three percent wet of optimum (Mixture 3). It appeared that the shear strength was not appreciably affected by moisture contents within two or three percent of optimum, when the material was compacted. Again, added strength from hydration may tend to negate any strength differences due to small changes in ~~moisture content. However, this may not be true for large moisture variations from optimum.~~

Because strength parameters for all tests were very similar, the results were combined into one plot (Figure 5) to determine a "collective" internal friction angle to be used in stability analyses. The resulting internal angle of friction was 40.8° and the cohesion was 6.1 pounds per square inch. Often, if a slide occurs in an overconsolidated clay or brittle material such as these mixtures, a tension crack will form on the active or "driving" side of the slide. When that occurs, the cohesion will be zero. Therefore, when making stability analyses for this study, cohesion was assumed to be zero.

Sludge material taken from the stockpile without laboratory processing ("raw" sludge) also was tested. Although the c' values for the three tests were different, the ϕ' values were approximately the same (average ϕ' =

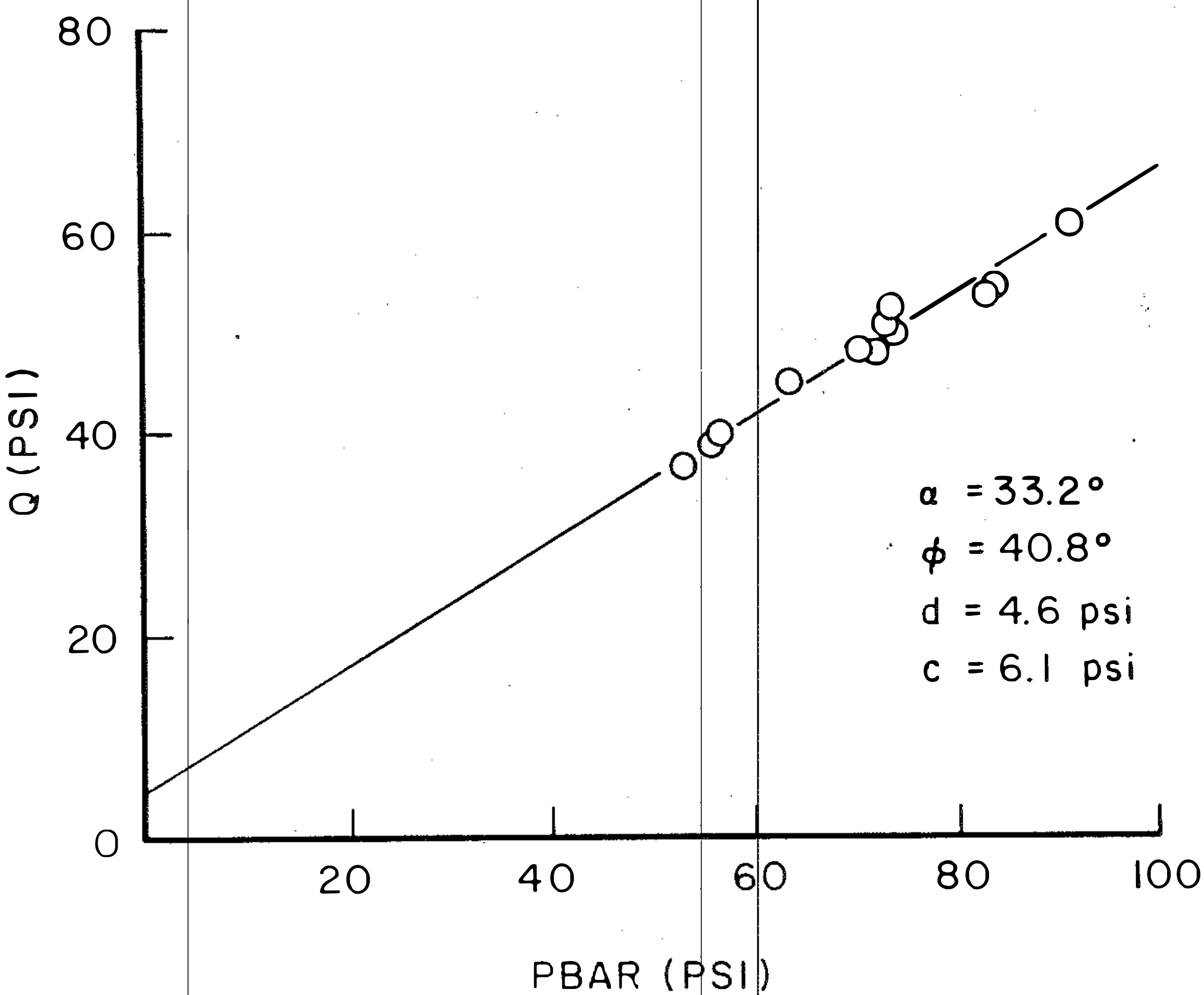

Figure 5. Combined Effective Stress Parameters for Three Mixtures Using the Peak Point from Each Stress Path.

39.9°). That compared well with the combined ϕ' value of 40.8° for the laboratory mixtures.

A number of "typical" embankment cross sections (see Figure 6 for an example and Table 6 for input parameters and resulting safety factors) were analyzed for stability using Bishop's simplified method of slices. Each embankment consisted of scrubber sludge with 2 feet of soil cover. The side slopes of the soil cover were flatter than the side slopes of the scrubber sludge core. A soil cover is required by the Kentucky Natural Resources and Environmental Protection Cabinet. In all analyses, the soil cover was assumed to have an internal friction angle of 28° and zero cohesion. The unit weight was assumed to be 115 pounds per cubic foot. An unit weight of 63 pounds per cubic foot was used for the scrubber sludge. Analyses were performed for both high and low water tables and for both rigid and compressible foundations.

Case 1 was a scrubber sludge core 18 feet high, 60 feet wide at the top with side slopes that were 2 feet horizontal to 1 foot vertical (2:1). The side slopes of the soil cover were 3:1. It appeared from these analyses that an embankment having a soil cover on a 3:1 side slope would be marginal (factor of safety less than 1.5) under high water conditions with failure occurring within the soil cover. However, if the water table could be maintained below the embankment level, it appeared the embankment with 3:1 side slopes may be a viable design. If the water table rises slightly above the level used in Analysis 1A, the factor of

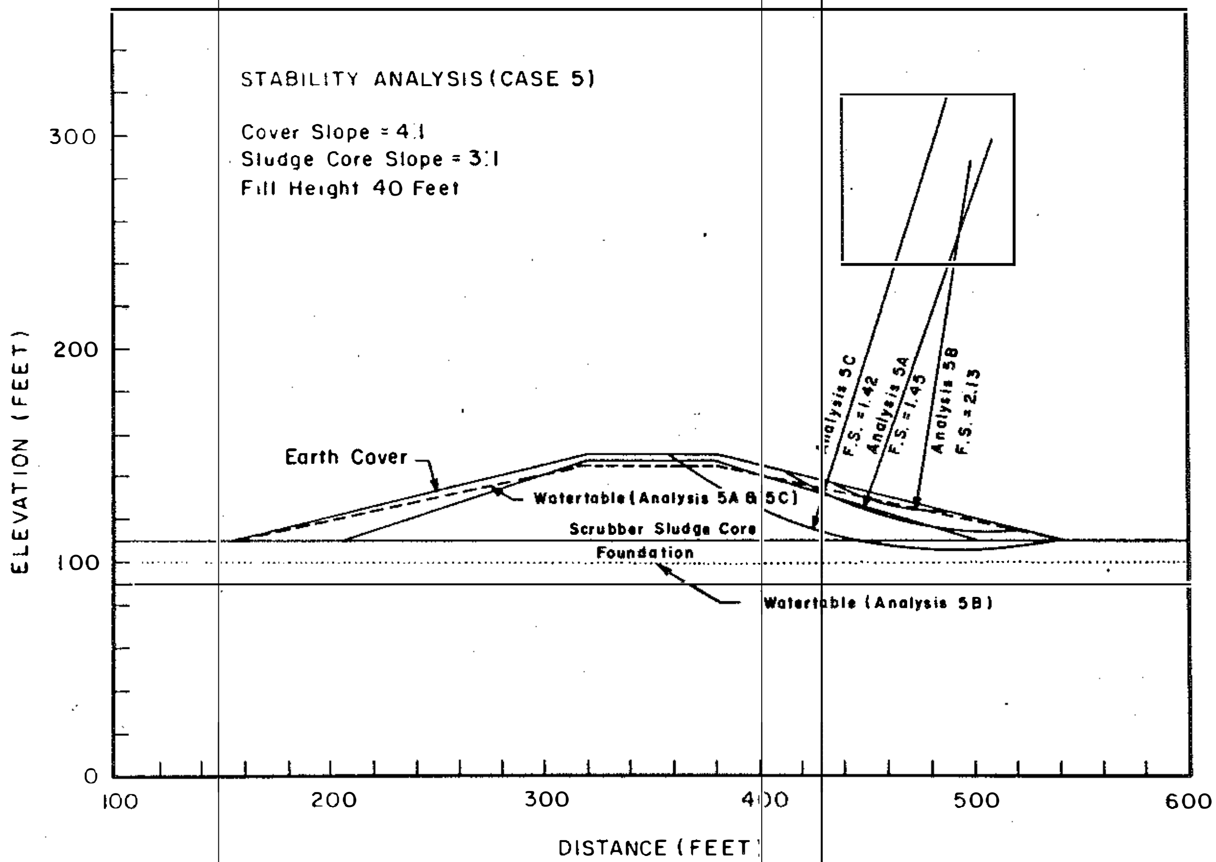

Figure 6. Case 5 Stability Analyses, Typical Embankment Cross Section.

TABLE 6. SUMMARY OF SLOPE STABILITY ANALYSES

CASE NO.	ANALYSIS NO.	FILL HEIGHT (feet)	SIDE SLOPES CORE	SIDE SLOPES COVER	FOUNDATION TYPE	WATER TABLE		SHEAR STRENGTH PARAMETERS CORE	SHEAR STRENGTH PARAMETERS COVER	SHEAR STRENGTH PARAMETERS FOUNDATION	MINIMUM SAFETY FACTOR
1	1A	20	2:1	3:1	Rigid	High	ϕ'	40.7	28.0	45.0*	1.21
							c'	0	0	4500*	
	1B	20	2:1	3:1	Rigid	Low	ϕ'	40.7	28.0	45.0	1.62
							c'	0	0	4500	
	1C	20	2:1	3:1	Compressible	High	ϕ'	40.7	28.0	28.0	1.17
							c'	0	0	0	
2	2A	20	2.5:1	3.5:1	Rigid	High	ϕ'	40.7	28.0	45.0	1.63
							c'	0	0	4500	
	2B	20	2.5:1	3.5:1	Rigid	Low	ϕ'	40.7	28.0	45.0	1.89
							c'	0	0	4500	
	2C	20	2.5:1	3.5:1	Compressible	High	ϕ'	40.7	28.0	28.0	1.55
							c'	0	0	0	
3	3A	20	3:1	4:1	Rigid	High	ϕ'	40.7	28.0	45.0*	1.67
							c'	0	0	4500*	
	3B	20	3:1	4:1	Rigid	Low	ϕ'	40.7	28.0	45.0	2.15
							c'	0	0	4500	
	3C	20	3:1	4:1	Compressible	High	ϕ'	40.7	28.0	28.0	2.08
							c'	0	0	0	
4	4A	40	2:1	3:1	Rigid	High	ϕ'	40.7	28.0	45.0	1.03
							c'	0	0	4500	
	4B	40	2:1	3:1	Rigid	Low	ϕ'	40.7	28.0	45.0	1.60
							c'	0	0	4500	
	4C	40	2:1	3:1	Compressible	High	ϕ'	40.7	28.0	28.0	1.03
							c'	0	0	0	
5	5A	40	3:1	4:1	Rigid	High	ϕ'	40.7	28.0	45.0	1.45
							c'	0	0	4500	
	5B	40	3:1	4:1	Rigid	Low	ϕ'	40.7	28.0	45.0	2.13
							c'	0	0	4500	
	5C	40	3:1	4:1	Compressible	High	ϕ'	40.7	28.0	28.0	1.42
							c'	0	0	0	

* ϕ' in degrees and c' in lb/sq in.

safety falls to less than 1.0. Again, failure would occur in the soil cover.

Case 2 was a scrubber sludge core 18 feet high, 60 feet wide at the top, with side slopes that were 2.5 feet horizontal to 1 foot vertical (2.5:1). Side slopes of the soil cover were 3.5:1.

Case 3 was a scrubber sludge core with side slopes of 3:1 and a soil cover with side slopes of 4:1. As in Case 1, the embankment was 60 feet across the top and the scrubber sludge core was 18 feet in height.

~~Cases 4 and 5 were similar to Cases 1 and 3, respectively, except the height of the scrubber sludge core~~ was 38 feet.

A summary of all analyses is given in Table 6. Most critical arcs were shallow slips through the earth cover. However, when using a compressible foundation and a high water table, the critical arcs passed through the sludge core and foundation. That occurred only when the side slopes of the earth cover was 3.5:1 or 4:1. When the earth cover side slopes were 3:1, the shallow slips in the cover still prevailed.

It is recommended that embankments 20 feet or less in height be constructed with side slopes on the earth cover no steeper than 3.5:1. For embankments over 20 feet, side slopes should be 4:1 or flatter. This recommendation is based upon information shown in Figure 6. Analysis 5C had a factor of safety of 1.42, which is considered marginal (less than 1.5). Therefore, any side slope steeper that 4:1 will, undoubtedly, yield a factor of safety even lower, making the design unacceptable. These recommendations are based on the assumptions of a high water table and that the material will be placed with moisture contents near optimum and with unit weights near the laboratory maximum dry density.