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EVALUATION OF POZZOLANIC MIXTURE CONTAINING CEMENT KILN DUST (CKD) (IL 83-11)

by

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INTRODUCTION

Illinois Department of Transportation policies permit two types of pozzolanic aggregate mixture (PAM) base courses for flexible pavement; they are cement-fly ash-aggregate (CFA) mixture or lime-fly ash-aggregate (LFA) mixture. Their usage has been supported by numerous field studies and by formal research studies conducted at the University of Illinois.

In recent years, N-Viro Energy Systems of Toledo, Ohio has launched a nationwide effort for the marketing of "N-Viro Crete", a pozzolanic aggregate mixture containing cement-kiln-dust (CKD) as a substitute for hydrated lime. It has been stated that CKD and hydrated lime react similarly with fly ash to develop a pozzolanic material. Illinois Department of Transportation's own laboratory studies concluded that CKD may provide sufficient pozzolanic strength to meet the present requirements. The cost of CKD is lower than the hydrated lime; therefore, PAM containing CKD will result in lowering the base course cost.

The objective of this study is to evaluate the performance of PAM containing CKD, fly ash, aggregate, and water. The work plan of the subject study calls for the placement of PAM containing CKD on five (5) highway projects. Under the Special Provision prepared for this study, the Contractor was given the option of constructing either a Bituminous Base Course or Pozzolanic Base Course. Until 1985 the Pozzolanic option had been used by the Contractor only on one project (I-255-7(157)7). This project was constructed in one day (August 10, 1983). The design thickness of the pavement consisted of 3 inches of Mix D Surface and 5 inches of PAM base. The usable length of this experimental project is only about 250 feet and the findings are insufficient to draw conclusions.

Due to an unforeseen industry trend, the actual construction of a reasonable length of pavement where the experimental feature could be investigated was delayed until the fall of 1986.

It is the purpose of this report to describe the construction of pozzolanic base course and to discuss its behavior after two winters.

DESCRIPTION OF PROJECT

The experimental base course is located on the Northwest Drive in the city of Collinsville. This project was constructed by the city of Collinsville but was totally funded by the Illinois Department of Transportation. The net project length is 1.217 miles of which the usable length for this research is about 3700 feet. The average annual precipitation in this area is about 36 inches. The temperature ranges from about 69-88° F. in the summer to 23-44° F. in the winter. Normally, the frost penetration average is about 13 inches. The main soils on the site of the experimental project are A-4(6) down to about 3 feet and A-7-6(68) for 3-6 foot depth.

The design thickness of the pavement consists of 4 inches of Mix D surface and 11 inches of PAM base. Various details of the test pavement are given in Figure 1.

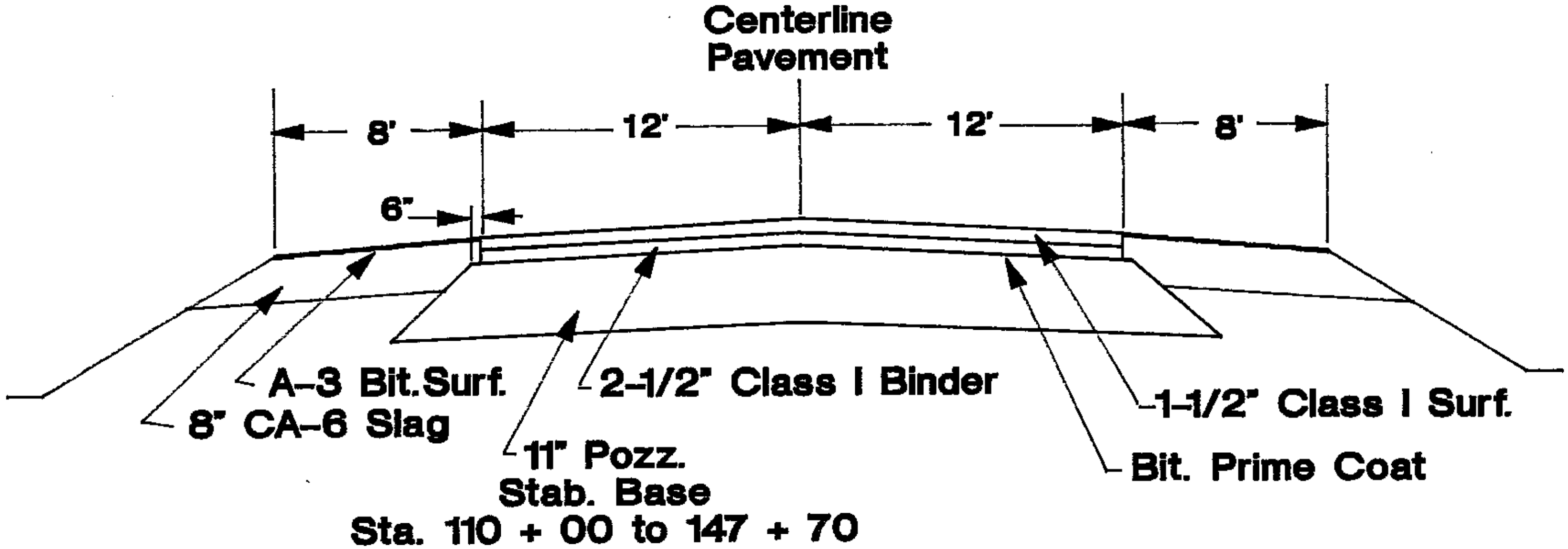


Figure 1. Typical cross-section of the research pavement

EMBANKMENT/SUBGRADE

The embankment is between about 2 to 4 feet high and it was constructed mainly from loessial soil. Natural ground is low lying and poorly drained. The maximum density determined in accordance with AASHTO T 99 for this embankment soil was 108 lb./cu. ft. and the optimum moisture was 16.5 percent. The IDOT specifications calls for 95 percent of standard laboratory density for the top layer. The compaction test results for the top lift were as follows:

	Density (lb./cu. ft.)	Moisture (%)	Percent of Standard Test Density	Percent of Standard Test Moisture
Mean	110.0	17.7	101.8	107.2
Sta. Dev.	2.4	2.1	1.9	13.0
No. of Tests	5	5	5	5

As can be seen from these field test results, the embankment was well compacted during its construction. But during the construction of the PAM base, some problems which appeared to be associated with the embankment developed. These are discussed under PAM base construction.

EVALUATION OF MIXTURES

The mix design was furnished by the Central Bureau of Materials and Physical Research in accordance with the State's laboratory mix design procedure for PAM (LFA). All samples were cured for 14 days at 72° F. and soaked 4 hours before testing. The mix design data was as follows:

Mix Proportions (%)					Average Specimen
Kiln Dust	Fly Ash	Agg.	Density PCF	Moisture %	Comp. Strength (PSI)
8	8	84	141.7	5.7	1089

The CKD came from Dundee Cement, Clarksville, Missouri, fly ash from National Mineral, Baldwin, Illinois, and limestone aggregate from Columbia Quarry #9. It was intended to collect daily a production sample of CKD and fly ash for specification validation but unfortunately they were collected only on the last day of production. Typical physical and chemical properties for these samples were as follows:

1. Physical properties of CKD Residue

Sieve	Specification Max. % Retained	Sample Result % Retained
No. 4	0	0
No. 30	2.5	0
No. 100	15	5.8

2. Chemical Properties of CKD

	Specification	Sample Result
a. Total reactive alkalines		
% CaO + % MgO - (%L.O.I. x 1.2)	Min. 10%	11.06%
b. Total reactive alkalis		
% K_2O + %Na_2O	Max. 10%	4.54%
c. SO_3	Max. 10%	5.58%

3. Physical Properties Fly Ash

Sieve	Specification Min. % Passing	Sample Result % Passing
1/2 inch	100	100
3/8 inch	95	100
No. 10	75	100

4. Chemical Properties of Fly Ash (Type F)

	Specification	Sample Result
Loss-on-Ignition	Max. 10%	0.82%
Moisture Content	Max. 35%	0.12%

The fly ash was stored in a large storage tank and from there it was delivered to the collecting belt by means of a 12-inch auger. The CKD was pumped from delivery trucks and delivered to collecting belt by means of a vane feeder. The aggregate collector belt and the kiln dust feeder belts were equipped with belt scales with digital readout in the control room. The quantity of the fly ash was dependent upon the vane feeder settings. City water was added by a sprayer at the head of the mixing lane. All ingredients, in theoretically correct proportions, were mixed in a Barber Greene Model KA-85 continuous pugmill. Based on the earlier calibrations, the paddles on sets No. 1, 4, and 8 were reversed to increase the mixing time. The mix was delivered to the jobsite by trailer trucks.

To evaluate the mixing efficiency of this plant, 3-4 compression samples per paving day were molded prior to the placement of the mix. The locations were recorded for long-term field evaluation. The test results of these samples are presented in Table 1. These specimens were the same dimensions as used in the initial design procedure. They also were cured (14 days @ 72° F.) and tested in the same manner as the initial mix design. There were 14 specimens and the average compression strength was 699 psi (range 481 to 994 psi). This resulted in an average mixing efficiency (field strength/lab strength) of 0.64 (range 0.44 to 0.91). The mix was placed in two lifts providing the average strength for bottom lift of 707 psi (range 481-994) and for the top lift of 667 psi (range 562 to 791). The plant mixing efficiency of 0.66 is generally considered acceptable.

Table 1

SUMMARY: FIELD MOLDED SPECIMENS FROM N-VIRO CRETE

Date	Sample #	Location	Lift	Wet Dens. (PCF)	Dry Dens. (PCF)	Moisture %	14 Day Comp. Strength PSI
10/9/86	1	111+50	Bottom	147.4			520
10/9/86	2	117+50	Bottom	149.3			820
10/9/86	3	120+60	Bottom	145.4			547
10/9/86	4	112+75	Top	150.7			564
10/10/86	5	124+50	Bottom	146.9	138.5	6.1	797
10/10/86	6	130+25	Bottom	151.8	141.2	7.5	481
10/10/86	7	134+35	Bottom	153.3			625
10/10/86	8	128+50	Top	150.3			654
10/11/86	9	138+40	Bottom	149.6	141.0	6.1	994
10/11/86	10	136+00	Top	149.4	139.0	7.5	791
10/11/86	11	143+50	Top	149.8	140.7	6.5	767
10/16/86		101+75	Bottom	149.0	139.5	6.8	738
10/16/86		100+75	Bottom	149.1	139.4	7.0	841
10/16/86		102+80	Top	148.1	137.5	7.7	562

	Ave. Compression Strength (PCF)	Plant Efficiency %
Bottom Lift	707	65
Top Lift	667	61
Combine	699	64

CONSTRUCTION OF POZZOLANIC BASE

The pozzolanic base construction began on October 8, 1986. On the first day the plant broke down after producing only two truckloads. This mix proved difficult to compact and was removed. Placement was resumed on October 9 at Station 110+50 and proceeded north to Station 147+70. This portion was completed on October 11, 1986. Rain delayed the completion of the section from Station until October 16, 1986.

The pozzolanic mixture was placed in two lifts by a CMI slip form paver to a compacted thickness of 7 1/2 inches for the bottom lift and 6 inches for the top lift. Compaction was performed using 10 to 12 passes per lift with a vibrating pad foot roller on the first day; this was subseauently reduced to 8 passes. Final compaction was achieved with three passes of the top lift with a smooth drum vibratory roller.

It was observed that the compaction operation was not providing a proper interface between the lower and top lift. The wide-toothed roller used for compaction was leaving the top two inches of the first lift in a disturbed state. By the time the top lift was placed, this disturbed surface had dried up. This condition was confirmed during coring operation. A layer of loose or uncemented material between the top and bottom lifts was observed in the majority of the cores that were cut after construction.

At the plant, there were a number of moisture related problems even though the plant was calibrated immediately prior to mix operations for the project. Field testing revealed that minor moisture changes, which were usually undetectable at the plant, produced major changes in the compactability of the material. The problem was solved by maintaining a more uniform control of the aggregate moisture content.

Another problem was caused by an increase in moisture in the embankment/subgrade during the interim period between its construction and the placement of the base course. As has been stated, the natural ground is low lying and poorly drained. Rainfall during this period created ponding on the original surface, and more importantly, against the embankment causing it to "pump" water and to become unstable during the hauling and compacting of the pozzolan.

The binder course on the PAM base was placed on November 3 and 6, 1986. The "D" mix surface was placed on December 4 and 5, 1986.

To control the transverse cracks of the pozzolanic base, contraction joints at 30' c.c. were cut 3-4 days after paving. The joints in the asphaltic surface were cut within 1/2-inch of the PAM joints, which were filled with a hot poured joint sealer.

FIELD PERFORMANCE

For the purposes of this study, the field performance of the PAM base containing CKD was based on the data from deflection measurements, field coring, and pavement condition surveys.

DEFLECTION

Deflections were measured under 9000 lb. force of falling weight deflectometer (FWD). The results of deflection measurements under load and associated deflection basin "area" are presented in Table 2.

The "area" of the deflection basin is a parameter combining the four sensor readings. The measurement of the deflection basin provides a means of determining the relative stiffness of a pavement structure. The "area" will range between 11.1 and 36 inches. The stiffer the pavement, the larger the area. The "area" for sound and stiff base or pavement is 30 ± 5 inches while for badly D-cracked pavement or unsound base it is 18 ± 3 inches. The mean "area" of 25.6, 28.1, and 26.5 inches suggest that this pavement has developed and is maintaining a relatively stiff pavement structure.

Deflection for a new rigid pavement (10" PCC + 4" stabilized subbase) is in the range of 4 to 8 mils when tested in the outer wheel path under a 9000 lb. FWD load. As can be seen from Table 2, the mean deflection for this pavement is 6.3, 4.8, and 6.5 mils which is typical of the stiff pavement range.

Table 2

SUMMARY OF MEAN DEFLECTIONS

Date	Pav't. Temp. (°F.)	DO (Mils)	D1 (Mils)	D2 (Mils)	D3 (Mils)	"Area" (In.)	ERI (KSI)
4/14/87	85	6.3	4.7	3.9	3.2	25.6	8.0
	Sta. Dev.	1.6	1.2	0.8	0.6	1.5	2.2
	Cov. (%)	25.5	25.0	20.8	17.5	5.7	27.0
10/19/87	63	4.8	4.0	3.4	2.8	28.1	9.5
	Sta. Dev.	1.1	0.8	0.6	0.4	1.9	1.9
	Cov. (%)	23.6	21.2	17.9	15.6	6.7	19.5
4/5/88	92	6.5	5.0	4.2	3.4	26.5	7.2
	Sta. Dev.	1.9	1.4	0.9	0.6	1.6	2.1
	Cov. (%)	29.9	27.9	22.2	17.3	6.2	29.6

DO = Deflection underload
D1 = Deflection 12 in. from load
D2 = Deflection 24 in. from load
D3 = Deflection 36 in. from load
"Area" = 6 (DO+2D1+2D2+D3)/DO (inches)
ERI = 25.7-7.28 D3+0.53 D3xD3 (KSI)

There is not much difference between the fall and spring deflection and "area" for this pavement. During normal winters in the St. Louis area, two to three freeze-thaw cycles take place. The last two winters (1986-1987 and 1987-1988) have been very mild and did not produce a single good freeze-thaw cycle. Therefore, the effect of freeze-thaw could not be evaluated.

All of the deflections are considered to be quite low and "area" considered to be high, indicating good base support at all locations.

FIELD CORING

To evaluate the field performance of the PAM base, cores were cut 45-55 days after placement rather than during the first and second spring. A series of 7 cores were cut each time and they were cut within one-foot of the original location. The original locations were selected based on the initial deflection measured by the falling weight deflectometer (FWD) after the binder course was placed. Three locations represented low deflection (3.2-4.7 mils), three had medium deflection (6.1-8.5 mils) and one had high deflection (19.4 mils). Data from these cores is presented in Tables 3, 4, and 5. The conditions of the cores are shown in Figure 2.

As can be observed from the photographs, favorable climatic conditions (above average temperatures and below normal precipitation) over the two winters following the PAM base construction contributed to an improvement in its durability/cementation characteristics and a reduction in the damaging effects of freeze-thaw cycling. The condition of the cores ranged from a good solid

Table 3
SUMMARY OF FIELD-CURED STRENGTH FROM CORES
(December 3-4, 1986)

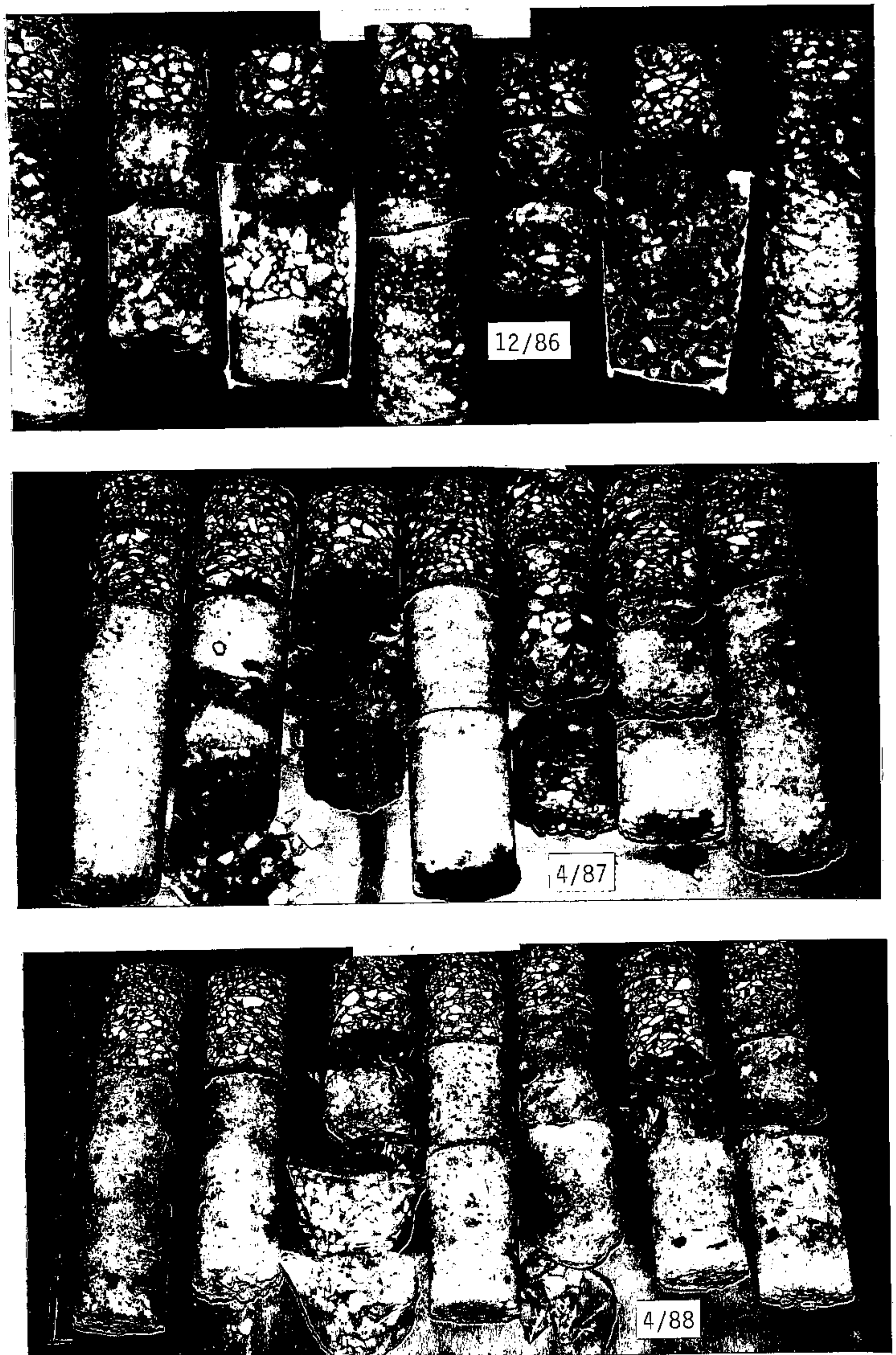
Core No.	Pavement Deflection (Mils.)	AC	Recovered Pav't. Thickness (In.) Upper Lift	Loose Mat'l.	Lower Lift	Comp. Strength (PSI)	Remarks
1	3.2	3.6		---	---	---	
Sta. 145+00 WB			5.7	---	---	1121	Good solid core
			---	0	---	---	
					5.2	1153	
2	3.9	3	---	---	---	---	
Sta. 137+00 EB			3.6	---	---	1036	Two pieces with loose material
			---	1.2	---	---	
			---	---	6.0	1173	
3	19.4	3	---	---	---	---	
Sta. 125+00 EB			4.2	---	---	---	Two pieces with loose material
			---	2.4	---	---	
			---	---	4.2	997	
4	4.7	2.7	---	---	---	---	
Sta. 118+00 WB			4.2			1056	Two pieces without loose material
			---	0	---	---	
			---	---	7.2	744	
5	6.1	2.4	---	---	---	---	
Sta. 114+25 EB			3.1	---	---	---	Two pieces with loose material
			---	3.6	---	---	
			---	---	4.1	897	
6	8.5	3.6	---	---	---	---	
Sta. 110+00 WB			0	10.8	0	---	Loose material; no solid piece
7	7.7	2.4	---	---	---	---	
Sta. 106+00 WB			6	---	---	830	Solid core, bottom-voids loose
			---	0	---	---	
					5.1	209	
					Average	922	

Table 4
SUMMARY OF FIELD-CURED STRENGTH FROM CORES
(April 30, 1987)

Core No.	Pavement Deflection (Mils.)	AC	Recovered Pav't. Thickness (In.) Upper Lift	Loose Mat'l.	Lower Lift	Comp. Strength (PSI)	Remarks
1	---	5.4	5.2	---	---		
			---	0	---		
Sta. 145+00 WB			---	---	5.9	1225	Good - 1 piece
2	---	4.75	3.1	---	---		
			---	0	---		
Sta. 137+00 EB			---	---	5.4	1269	Good - loose mat'l. bottom
3	---	4.0	0	---	---		
			---	7.25	---		
Sta. 124+50 EB			---	---	3.0		No core - loose mat'l. throughout
4	---	4.5	5.0	---	---	1307	Good
			---	0	---		
Sta. 118+00 WB			---	---	6.3	1527	Good
5	---	2.9	5.2	---	---	397	Fair
			---	1.3	---		
Sta. 114+60 EB			---	---	4.0	1347	Average
6	---	5.2	2.85	---	---	518	Fair
			---	1.65	3.0		
Sta. 110+00 WB			---	---	---	851	Fair
7	---	4.5	4.1	---	---		
			---	0	---		
Sta. 106+00 WB			---	---	6.4	477	Good
					Average	990	

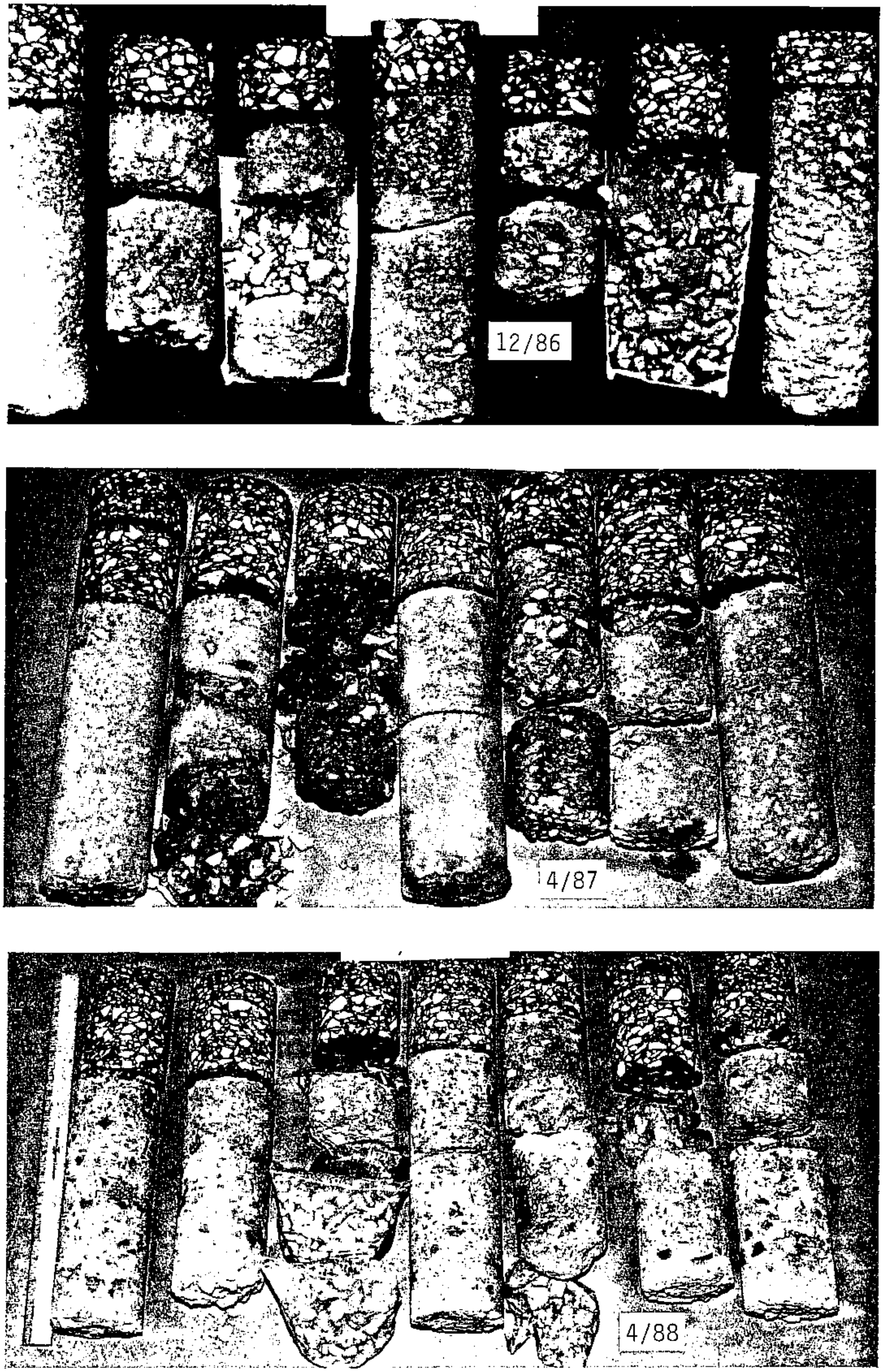
Table 5
SUMMARY OF FIELD-CURED STRENGTH FROM CORES
(April 13, 1988)

Core No.	Pavement Deflection (Mils.)	AC	Recovered Pav't. Thickness (In.) Upper Lift	Loose Mat'l.	Lower Lift	Comp. Strength (PSI)	Remarks
1	---	5.0	5.0	---	---	---	
			---	---	6.1	1634	Good solid
Sta. 145+00 WB			---	0	---	---	
2	---	4.5	3.0	---	---	---	
			---	---	6.1	1626	Good solid
Sta. 137+00 EB			---	0	---	---	
3	---	4.0	---	1.0	---	---	
			2.4	---	---	---	
Sta. 124+50 EB			---	7.25	---	495	Small - loose mat'l throughout
4	---	4.4	4.8	---	---	1213	Good
			---	---	---	---	
Sta. 118+00 WB			---	---	6.85	1523	Good - rough separation
5	---	3.1	4.8	---	---	781	Average - separation rough
			---	1.0	---	---	
Sta. 114+60 EB			---	---	5.2	1217	Average - separation rough
			---	1.6	---	---	
6	---	5.5	---	---	---	---	
			---	3.75	---	---	
Sta. 110+00 WB			---	---	5.6-	825	Average
7	---	4.0	---	1.75	---	---	
			3.3	---	---	1791	Good - solid
Sta. 106+00 WB			---	---	6.50	712	Good - solid
					Average	1182	



	WB	EB	EB	WB	EB	WB	WB
Station	145+00	137+00	124+50	118+00	114+60	110+00	106+00
Core	1	2	3	4	5	6	7

Figure 2. Photographs showing the conditions of the cores.



	WB	EB	EB	WB	EB	WB	WB
Station	145+00	137+00	124+50	118+00	114+60	110+00	106+00
Core	1	2	3	4	5	6	7

Figure 2. Photographs showing the conditions of the cores.

homogenous (core #1 deflection 3.2 mils) to loose gravel (core #3, deflection 19.4 mils.) A majority of the core samples did, however, reveal construction related problems specifically: lack of cementation and compaction, inadequate mixing, and segregation. At most core locations loose material was encountered between the top and bottom lift, a condition previously described as being created by the toothed roller used for compaction.

The compression strengths for the samples retrieved from coring are as follows:

	Average	Compression Strength (PSI)	
Date of coring	12/4/86	4/30/87	4/14/88
Average	922*	990	1182

*Inadvertently core specimens were stored at room temperatures for 40 days before testing. The curing environment in the heated building is higher and not representative of the field condition. In general, higher temperatures increases the compressive strength at a faster rate.

As expected, the compressive strength development has continued with age. It should be kept in mind that the compressive strengths presented above and in Tables 3, 4, and 5 represent only the portion from which test samples could be obtained. Therefore, due to the presence of the uncemented material between the lifts or segregation, the strength results are not representative of the total PAM base layer. This loose material will collect water and cause further deterioration. These conditions are now adversely effecting the PAM performance.

PAVEMENT CONDITION SURVEY

The experimental section was surveyed on April 13, 1988, and all of the contraction joints looked good and remained well sealed. In appearance, they resemble PCC contraction joints. Throughout the section three transverse cracks and some wandering longitudinal cracking was observed. At two locations, transverse cracking within one foot of the contraction joint had developed. These cracks had developed 1/4" to 1/2" "hump". The third transverse crack was 9 feet from a contraction joint and had no "hump". These "humped" transverse cracks could lead to premature localized deterioration, therefore, early corrective treatment would be beneficial. At two locations, intermittent longitudinal cracking had developed and at the time, were tight at the surface and appears to have minor effect on the pavement performance.

SUMMARY OF PRINCIPAL FINDINGS AND RECOMMENDATIONS

Observation of the behavior of the experimental PAM (containing CKD) base course during construction and over a 2-year service period indicates the following:

1. The cement kiln dust has minimum production quality control, therefore, daily samples at the PAM plant should be obtained to conform that they meet the physical and chemical requirements set in the mix design and specification.

2. A small moisture change in the mix, which could not be visually detected, had a large effect on the PAM compaction. This was corrected by maintaining a more uniform control of the aggregate moisture content. It is recommended that the moisture must be controlled effectively at the plant to compensate for variations in the stockpiled aggregate, moisture contents, and the climatic conditions during PAM placement.

3. The location of this project in the flood plain and a subgrade of loessial soils caused an increase in its moisture content after heavy rain. This increase in the moisture content caused the subgrade to "pump" and significant rutting developed at many locations during the pozzolan hauling and compaction. When there is a danger of excessive rutting or increase in the moisture content, then lime modification or granular fill/backfill is recommended to provide a working platform for the construction of PAM base course

4. Loose and uncemented material between the bottom and top lift was observed in a majority of cores when they were cut after construction. Water will collect in this loose material and will cause premature deterioration. To avoid these construction-related problems, it is recommended that the PAM should be constructed in one lift.

5. It appears that a properly designed and constructed PAM containing CKD can provide a moderate strength stabilized base course. A definite conclusion based on presently available information is considered premature at this time. Thus, it is recommended that reevaluation of the experimental base course after another 5-7 years of service be conducted to confirm the present findings.