

Research Report
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FLY ASH STABILIZED BASES IN KENTUCKY

by

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INTRODUCTION

NATURE OF POZZOLANIC MATERIALS

The use of pozzolans in cementing materials antedates recorded history. The name pozzolana was first applied to loosely consolidated rock of volcanic origin consisting of pumice, obsidian, feldspar, pyroxines, and quartz. The term also has been recently extended to include not only natural volcanic materials, but diatomaceous earths and other highly siliceous rocks and artificial products. Pozzolans, even though not cementitious in themselves, are now defined as siliceous materials that 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. Pozzolans may or may not require calcination to make them active. Most natural (and artificial) pozzolans require grinding to a high degree of fineness to make them suitable. Artificial pozzolans come from industrial byproducts or wastes and include fly ash (flue dust), silica fume, powdered brick, burnt clays and shales, and some slags.

USE OF POZZOLANIC MATERIALS

With the escalating costs of materials and construction for highways and streets, many agencies charged with the responsibility of designing and constructing highways are utilizing byproduct stabilized (pozzolanic) materials. Low-strength (pozzolanic) materials have been used fairly extensively in some areas of the United States as well as abroad. In general, pozzolanic materials have been used to stabilize an aggregate base or subbase by addition of fly ash and a source of lime to develop a cementitious reaction. Additionally, portland cement or cement kiln dust have been used to stabilize aggregate subbase and (or) base materials.

Until recently, the use of stabilized materials in highway and street construction in Kentucky was not often economically competitive with abundant supplies of high-quality aggregates. However, as costs of production and processing aggregate materials have increased, so has the feasibility of stabilized bases, and particularly pozzolanic base materials.

Mixtures that have been considered recently and evaluated to some degree include the following:

- Lime kiln dust, fly ash, and dense-graded aggregate;
- Byproduct lime and dense-graded aggregate;
- Lime kiln dust, fly ash, dense-graded aggregate, and sand;
- Lime kiln dust, fly ash, and limestone mine screenings (waste material from limestone quarrying operations); and
- "Scrubber sludge," quicklime, and dense-graded aggregate or pond ash.

Pozzolanic base or subbase materials have been utilized on an experimental basis for a number of Lexington, Kentucky, street projects. Two projects for the Kentucky Transportation Cabinet also are being evaluated. Thus, performance experience currently is limited, but at the same time evolutionary.

SPECIFICATIONS

The Kentucky Department of Highways specifications for fly ash stabilized base (1) are covered by Special Provision No. 70(83). These specifications were utilized for Demonstration Project 59, Man-O-War Boulevard in Lexington.

These specifications pertain to pozzolanic base construction, including the ingredient materials, mixture production, construction, and measurement/payment. The specifications provide for the traditional hydrated lime - fly ash - aggregate or for kiln dust (lime or cement) - fly ash - aggregate pozzolanic mixtures. Hydrated lime is required to conform to ASTM C 207, Type N, Sections 3, 6, 7.1.1, 10, and 11. Fly ash must meet ASTM C 593 requirements. Kiln dust (lime or cement) must be basically lime in the form of CaO and (or) CaOH. When used, the contractor is required to furnish past and current test data on chemical composition to show a uniform product. Aggregate shall be crushed stone, or a mixture of crushed stone and sand, conforming to Kentucky Department of Highways standard specifications for crushed stone (DGA) base. The bituminous curing seal is required to be one of the standard slow or rapid setting asphalt emulsions. The contractor proposes mixture proportions for approval by the Department on a project-by-project basis. Compressive strengths of the laboratory mixture after 7 days of curing at 100°F must be 600 psi or greater. Samples are prepared on the basis of specifications presented in ASTM C 593, except ASTM D 698 (KM 64-511)(2) is used instead of ASTM D 1557 as referenced in Section 8.3 of ASTM C 593.

Plant mixing is required in either a batch plant or a continuous volumetric type plant. Placing and compacting of base mixtures must be within 2 1/2 hours of mixing. A maximum lift thickness of 6 inches is permitted. Compaction to 102 percent of laboratory density (KM 64-511, comparative to AASHTO T 99) is required. The moisture content shall be maintained until a succeeding lift or a bituminous cure coat (within 24 hours) is applied. Normal requirements for protection and maintaining of the newly constructed base are specified. Seasonal limitations of April 30 to October 1 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, including water. The bituminous cure coat is a separate pay item.

MIXTURE DESIGN

Mixture design consists of various blends of fly ash, lime kiln dust and aggregate. By weight of the total mixture (excluding water), ranges of component materials are as follows:

Fly ash.....	5-8%
Lime Kiln Dust.....	4-8%
Crushed Limestone (DGA)....	34-90%
Natural Sand.....	0-50%

Specimens are prepared according to Kentucky Method 64-511, which is almost identical to AASHTO T 99. Specimens are cured and tested according to ASTM C 593 (7 days at 100°F). Typical optimum moisture content is about 6 1/2 to 7 percent. Unconfined compressive strengths typically range from about 900 to 1,500 psi. Mixtures containing 50 percent natural sand generally have the lower strengths.

QUALITY CONTROL

INGREDIENT MATERIALS

Fly Ash: Preliminary test data are received from the source prior to shipment to indicate uniformity and specification compliance. Complete test results are required quarterly with loss on ignition and fineness required weekly. Acceptance samples from the job site are taken by state personnel at the rate of one sample per 100 tons.

Lime or Kiln Dust: Hydrated lime is sampled for acceptance testing at the rate of one sample per 100 tons. Kiln dust is required to be uniform and capable of achieving the specified strength in the pozzolanic base mixture. The source (through the contractor) is required to furnish test data showing chemical test results, especially CaO or CaOH contents, and uniformity of those test results. Samples are required from the job site at the rate of one sample per 100 tons.

Aggregate: Aggregate from approved sources is sampled at the rate of one sample per 25,000 tons for sodium sulfate soundness and L. A. abrasion. Sand equivalent tests are performed every 10,000 tons and gradation every 2,000 tons.

PROPORTIONING AND PLACEMENT

Mixture: Mixture proportioning is checked in the laboratory prior to beginning production to determine optimum moisture content, target density for field compaction, and compressive strength. Mixture design checks with plant-mixed material are made as required by changes in materials, proportioning, etc. and by project size. Proportioning checks normally are made twice daily (or more often) at the mixing plant.

Field Compaction (Density) Control: Density tests with nuclear gages are made at the minimum frequency of one test per 1,000 linear feet per lift of stabilized base.

Cores: After a minimum 28-day cure, cores are taken from the stabilized base for verification of thickness and strength of the layer and mixture. Those cores are not for routine strength acceptance purposes but serve as a check between the laboratory-mixed and cured specimens and the plant-mixed material and specified field curing conditions.

DESIGN REQUIREMENTS

Kentucky specifications (1) currently require fly ash stabilized base mixtures used as base components of pavement structures to have unconfined compressive strengths greater than 600 psi at 7 days when specimens are prepared, cured, and tested in accordance with ASTM C 593 with previously noted exceptions. Mixtures used for bases normally have three components: fly ash, a source of lime (hydrated lime, quicklime, or lime kiln dust), and an aggregate. Cement or cement kiln dust have been substituted for the lime source.

Pozzolanic mixtures used as subbases are not generally required to have strengths as great as those for bases. There are no strength requirements in Kentucky for subbase applications. Recent experience on one project resulted in compressive strengths in the order of 300 psi at 7 days when cured according ASTM C 593. Two mixtures that have potential as a subbase material have been investigated in the laboratory: 1) scrubber sludge, aggregate, and some form of lime and 2) aggregate stabilized with baghouse lime. Compressive strengths of 300 to 600 psi at 7 days when cured according to ASTM C 593 have been obtained.

PAVEMENT THICKNESS DESIGN

DESIGN METHODOLOGIES

Structural Number: Other agencies have developed layer coefficients for pozzolanic base materials for use with the AASHTO Interim Guide for flexible pavement design (3). There has been considerable discussion regarding the use and reliability of layer coefficients compared with more rationally based design systems advocated by others.

A review of literature has indicated considerable variability among suggested layer coefficients for pozzolanic materials (4, 5, 6). The range of suggested coefficients varies from 0.20 to 0.44 with most recommendations in the order of 0.28 to 0.30 for pozzolanic base mixtures. Lesser values for structural coefficients are recommended for lower strength materials used as subbases.

Stress Ratio: Other thickness design procedures (7) use a failure criterion relating the ratio of flexural strength to modulus of rupture as a function of repetitions to failure. Flexural strength and modulus of rupture are determined from laboratory tests and analyses.

ELASTIC LAYER APPROACH

Current thickness design procedures for both rigid and flexible pavements in Kentucky have been developed using elastic layer theory matched with pavement performance histories. Flexible thickness design procedures (8, 9) are supported by over 40 years of pavement performance experience and also have been related to AASHTO Road Test data and pavement performance experience.

Thickness designs in Kentucky (both flexible and rigid) are based on limiting strain criteria. A strain-repetitions failure criterion was developed by matching theoretically computed strains with repetitions determined from historical pavement performance data and previous empirical thickness design procedures.

The flexible pavement criterion limits vertical compressive strains at the top of the subgrade and the tensile strain at the bottom of the asphaltic concrete (8, 9). For rigid pavements, a limiting strain criterion was developed and related to the merged fatigue criteria of the Portland Cement Association and AASHTO thickness design procedures (10, 11, 12). The rigid pavement design criterion is an expression of a stress-ratio fatigue criterion in terms of tensile strain versus repetitions for various combinations of modulus of elasticity and modulus of rupture. The same approach was used to develop a tensile strain-repetitions criterion for pozzolanic base materials (13) except shifted to reflect the much lower modulus of elasticity associated with pozzolanic materials.

For the pozzolanic material, the ratio of flexural stress to compressive stress at failure was estimated to be 0.25 at the ultimate compressive strength (7, 13, 14) . Based on current Kentucky specifications of a minimum compressive strength of 600 psi, the flexural stress for pozzolanic base materials is 150 psi. The minimum design modulus of elasticity is 250,000 psi (13). The assumed Poisson's ratio of 0.15 for pozzolanic materials (14) is near the value used to develop rigid pavement designs in Kentucky. Thus, the shape of the fatigue envelopes are similar but have been shifted on the basis of modulus of elasticity.

EVALUATIONS, RESEARCH, AND DEVELOPMENT

Research and development activities have generally involved characterization of material properties as relates to pavement and base thickness design determinations and long-term pavement performance of pozzolanic base sections. Laboratory material characterizations have involved "standard" methods to determine compressive strength, tensile strength, modulus of elasticity, and Poisson's ratio. Laboratory experience has been supplemented by routine data obtained while monitoring quality of construction. Additionally, the literature has been reviewed to determine the experience of others. Deflection measurements have been and continue to be used to monitor long-term structural performance of in-service pavements.

ELASTIC MODULUS OF POZZOLANIC BASE MIXTURES

A literature review indicated a wide range of elastic moduli for low-strength base and subbase materials dependent upon specific procedures used to determine the parameters. All studies reviewed indicated increasing elastic moduli for pozzolanic materials proportionate to increases in compressive strength and (or) tensile strength. However, magnitudes of elastic moduli did vary considerably for similar compressive strengths.

Estimates of elastic moduli were determined by the static-chord method (ASTM C 469(65)) and generally were relatively low (30,000 psi to 300,000 psi) for past projects (13). Elastic moduli for lime-fly ash mixtures reported in Reference 4 on the basis of plate loading tests were on the order of 100,000 psi at a 400-psi compressive strength to 500,000 psi at a 1,000-psi compressive strength. Even greater magnitudes of elastic moduli (1,600,000 psi to 3,300,000 psi) have been reported by others (4).

Ahlberg and Barenburg (14, 15) reported flexural moduli of elasticity from 1,500,000 to 2,500,000 psi. Resilient moduli reported by Collins and Emery (16) varied from 370,000 to 3,300,000 psi.

OTHER CONSIDERATIONS

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.

Autogenous healing apparently occurs in pozzolanic base specimens if left undisturbed and curing conditions remain favorable. However,

conditions in the laboratory may not duplicate field conditions. Autogenous healing of cracks in field installations may be slowed by the stressing under traffic loadings. Field curing conditions (temperature and moisture) also may vary considerably.

CASE STUDY: MAN-O-WAR BOULEVARD

Kentucky's Demonstration Project 59 was set up for a 1.7-mile section of the Man-O-War Boulevard, a new suburban outer loop in southern Lexington. This section is expected to carry about 14,000 vehicles per day, including about two percent trucks. The project, Fayette County SSP 034-7229-60-C, is a combined grade and drain and surfacing job. Alternate pavement designs were provided as follows on earth subgrade of CBR 5 or better:

Alternate 1

8" Non-Reinforced Portland
Cement Concrete
3" Crushed Stone Base

Alternate 2

6 1/2" Asphaltic Concrete
12 1/2" Crushed Stone Base

Alternate 3

9 1/4" Asphaltic Concrete
(Full Depth)

Alternate 4

4" Asphaltic Concrete
10 1/4" Fly Ash Stabilized
Base

Bids were received on only Alternates 3 and 4. Alternate 4 was the low bid on the 2.2 million dollar project by less than \$3,000. For paving materials only, Alternate 3 was actually lower.

The fly ash stabilized base mixture proposed for the project by the contractor was 8 percent flyash, 8 percent lime kiln dust, and 84 percent crushed limestone having the following average gradation:

Sieve Size:	1" -	3/4" -	3/8" -	#4 -	#10 -	#40 -	#200
Percent Passing:	100 -	97 -	72 -	52 -	32 -	15 -	9

The fly ash was high quality with a loss on ignition of two percent. The lime kiln dust contained 53 percent of a uniform CaO. Laboratory-mixed and compacted specimens indicated an optimum moisture content of 6.6 percent and a maximum unit weight of 145 pounds per cubic foot with a compressive strength on 4-inch diameter specimens (7 days oven cure at 100 F) of 2,350 psi. This compared to two plant-mixed specimens, compacted and cured in the laboratory, having the following results:

<u>Sample</u>	<u>Dry Density</u>	<u>%Moisture</u>	<u>Compressive Strength</u>
No. 1	137 pcf	6.7	1,650 psi
No. 2	140 pcf	5.8	2,090 psi

Cores taken after approximately 65 days of cure showed an average compressive strength of 1,730 psi with a range on eight cores from 1,000 psi to 2540 psi.

The fly ash stabilized base material was mixed in a continuous volumetric-proportioned pugmill. The lime kiln dust was fed dry from a silo onto the aggregate belt; however, the fly ash was stockpiled, without protection, and was loaded into its feeder bin at the prevailing moisture content. Frequently, the fly ash clumped, preventing uniform flow from the bin opening.

Since compaction of the base is very important toward achieving the expected strength, the standard was high -- 102 percent of standard proctor laboratory density of 145 pcf. During construction, only 25 percent of the tests indicated this minimum requirement was achieved. The average density was 100.3 percent of 145 pcf with a standard deviation of 1.6 percent from the mean. A more reasonable standard would be 100 percent of laboratory density.

The 2 1/2 hour maximum time limit between mixing and completion of compaction proved impractical. The base was frequently still plastic 2-3 days after placing. This also caused a problem with applying the bituminous cure coat within the specified 24 hours after completion of finishing. Normal application by distributor using the spray bar would rut the still plastic base.

REFLECTIVE CRACKING

After the project was under contract, reflective cracking was noted on a section of a previous project using fly ash stabilized base. This first project was a ramp and mainline segment constructed in October 1983. Cracking was noted in the Spring 1984 in the mainline pavement in the 4-inch bituminous concrete overlay on a 80- 90-foot interval. Therefore, prior to beginning construction on Demonstration 59 project, it was decided to try some measure to prevent or retard expected reflective cracking. A change order was executed to apply a rubber-asphalt SAMI (Stress-Absorbing Membrane Interlayer). However, due to scheduling problems, the rubber-asphalt SAMI could not be used. Instead a SAMI using a CRS-2S polymer emulsion with 1/2-inch to No.-4 chip aggregate was used. Although this application was only about half as heavy as the intended rubber-asphalt SAMI, some benefit is expected. Performance to date indicates a nontreated section has cracked (two cracks in 1,500 feet) over the winter while the polymer asphalt SAMI treated section has not cracked as yet.

RESEARCH EVALUATIONS

All specimens for research evaluations 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 5.5-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). Maximum dry density and optimum moisture content were determined using a polynomial curve-fitting procedure. A smoothing technique was used to eliminate localized changes in concavity.

Analyses have included detailed material characterizations by destructive testing as well as development of a "deflection" history for

the section. Unconfined compressive strength tests (ASTM C 39(72)) were routinely performed on samples obtained during routine quality control testing. Typical compressive strengths obtained during testing were 1,700 psi. Splitting tensile strength test results (ASTM C 496(71)) and results from tests for static-chord modulus (ASTM C 469(65)) are being analyzed for research purposes. A Road Rater Model 400B was used to obtain deflection measurements on the subgrade, on the completed pozzolanic base, and on the completed asphaltic concrete pavement.

Analysis of data for compressive strength-modulus relationships have not been completed at this time. Preliminary analyses have indicated compressive strength and associated static-chord moduli ranging from strengths of 1,500 psi and moduli of 450,000 psi at a nominal 7-day accelerated cure (ASTM C 593) to strengths of 2,500 psi and moduli of 750,000 psi at a 90-day ambient cure. Analyses of cores obtained from the Man-O-War project have not been completed. Preliminary results have indicated compressive strengths on the order of 700 psi and the associated static-chord modulus of 90,000 psi for 14-day old field-cured specimens. The relative difference between field cores and laboratory-prepared specimens may be attributed to a lack of uniform compaction throughout the base course. Visual inspection of laboratory specimens indicated more uniform density when compared with field cores. A Model 400 Road Rater was used to obtain deflection measurements from in-service pozzolanic pavements. Deflection data indicated considerable variability and are currently being evaluated in more detail. Preliminary analyses indicate back-calculated elastic moduli of 1,000,000 to 3,000,000 psi.

PERFORMANCE HISTORIES

A number of pozzolanic pavement sections consisting of 6 inches of pozzolanic base and 2 inches of asphaltic concrete surfacing have been in service since August 1982. Deflection measurements have been obtained periodically. Cores also have been obtained periodically and the results of destructive evaluation correlated with deflection measurements and "back-calculated" effective pavement behavior. All these sections are local road sections and involve low fatigue "residential" street traffic.

Kentucky Transportation Cabinet experience has involved two projects. The first section consisted of approximately 1,600 feet of ramp and interchange paving in Lexington and was constructed as 4 inches asphaltic concrete and 10 inches pozzolanic base material. A similar structural section was constructed on 1.7 miles of the Man-O-War Boulevard as previously described.

Visual assessment of performance have indicated reflective cracking in the asphalt concrete at regular intervals (80 to 90 feet) for one 800-foot section. Deflections have been very low. Preliminary analyses indicate back-calculated moduli on the order of 1,000,000 to 3,000,000 psi.

Durability analyses have been limited. Vacuum saturation analyses have generally indicated high durability. Additional analyses involving freeze-thaw testing is planned.

CLOSING COMMENTS

Experience in Kentucky relative to the performance and life-cycle costs of pozzolanic pavements has been almost nonexistent. Thickness design procedures for pozzolanic pavements have been developed by other agencies for other regions, but the extent to which Kentucky conditions are represented has not been determined at this time.

Additional research and experience relative to life-cycle costs, durability of materials, fatigue-shear strain relationships, and pavement performance will be necessary to refine procedures and methodologies for thickness design. Pavement sections are currently in place and are being monitored to provide such data for future analyses and refinements. Economic analyses apparently indicate competitiveness with other materials for initial construction.

A major criticism of pozzolanic pavements relates to reflective cracking of asphaltic concrete layers associated with shrinkage cracking in the pozzolanic base. Additional evaluations are currently ongoing. One technique with significant potential involves the use of stress-relief layers between the pozzolanic and asphaltic concrete layers. Benefits are yet to be determined.

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