Pavement Thickness Designs Using Low-Strength (Pozzolanic) Base and Subbase Materials

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ABSTRACT

Information is presented on combining laboratory test data for pozzolanic base and subbase materials with elastic layer theory and a limiting strain criterion to determine thickness designs equivalent to conventional asphaltic concrete and crushed stone pavement structures. A summary of laboratory testing in Kentucky is also presented. An example thickness design determination is given that includes an economic comparison of alternative designs with the conventional asphaltic concrete and crushed stone thickness design.

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 term pozzolana 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 they are not cementitious in themselves, because they contain constituents that will combine with lime in the presence of water at ordinary temperatures to form compounds that possess cementing properties.

With the escalating costs of materials and construction for highways and streets, many agencies charged with the responsibility of designing and constructing highways are using byproduct pozzolanic materials. Low-strength (pozzolanic) materials have been used fairly extensively in some areas of the United States as well as abroad. 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. However, as costs of producing and processing aggregate materials have increased, so has the feasibility of using stabilized bases, particularly pozzolanic base materials. To date, pozzolanic bases in Kentucky have been used primarily in low-volume traffic situations. Mixtures that have been considered recently and evaluated to some degree include (a) lime kiln dust, fly ash, and dense-graded aggregate; (b) byproduct lime and dense-graded aggregate; (c) lime kiln dust, fly ash, dense-graded aggregate, and sand; (d) lime kiln dust, fly ash, and limestone mine screenings (waste material from limestone quarrying operations); and (e) "scrubber sludge," quicklime, and dense-graded aggregate or pond ash.

Pozzolanic base or subbase materials have been used on an experimental basis for a number of Lexington, Kentucky, street projects. Two projects for the Kentucky Transportation Cabinet also are being evaluated. Performance experience currently is limited but evolutionary. Modifications in the designs presented in this paper may be required to reflect additional field experience.

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 (1,2) are supported by more than 40 years of pavement performance experience and also have been related to AASHO Road Test data. Rigid pavement design procedures (3-5) have been related to performance experience embodied in design procedures of the Portland Cement Association (6) and the AASHO Road Test (7,8).

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

LOW-STRENGTH BASE AND SUBBASE MIXTURES

Materials

Kentucky specifications (9) currently require pozzolanic mixtures used as base components of pavement structures to have unconfined compressive strengths greater than 600 psi at 7 days when specimens are prepared and cured in accordance with ASTM C 593. 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 on the order of 300 psi at 7 days when cured according to ASTM C 593. Two mixtures that have potential as a subbase material have been investigated in the laboratory: (a) scrubber sludge, aggregate, and some form of lime and (b) 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.

Fly Ash

The properties of fly ash will vary depending on sources and properties of coal burned at the specific facility under consideration. The range of typical properties of fly ash are illustrated elsewhere (10). The fly ash is silt-sized spherical particles 0.015 to 0.050 mm in diameter.

Sources of Lime

Commercial sources of lime for use as a stabilizing material include quicklime and hydrated lime. Most highway agencies specify that lime materials shall meet requirements of ASTM C 207, Type N. Typical properties of limes used for stabilization are summarized elsewhere (10,11).

The characteristics of lime and cement kiln dusts may vary significantly, depending on specifics for each producing location. Typical ranges of composition and physical properties of cement and lime kiln dusts are reported elsewhere (12). Lime kiln dusts used in Kentucky for laboratory and field analyses were within those typical ranges.

Scrubber sludge is a waste material obtained with the use of scrubbers to remove fly ash and residue from coal-burning processes of electric generating power plants. Scrubber sludge (flue gas desulfurization sludge) consists of fly ash and a lime dust slurry filter cake material. The filter cake is a compound of calcium sulfate and calcium sulfite. Quicklime or hydrated lime normally is added to the sludge for stabilization. Stabilization reactions begin almost immediately after the combination of fly ash and lime with the dewatered sludge.

Aggregates

Aggregates for both base and subbase pozzolanic mixtures that have been investigated included dense-graded limestone aggregates, limestone mine screenings (byproduct of limestone quarrying operations), river sand, slag, and gravels. In addition, pond ash waste material has been evaluated in the laboratory and may be an appropriate aggregate for a subbase. The predominant aggregate in Kentucky has been dense-graded limestone.

Two types of aggregate--dense-graded limestone aggregate, which meets Kentucky specifications (13), and pond ash (also called bottom ash)--have been used to prepare sludge-aggregate mixtures. Gradation tests as well as a slake durability test (Kentucky method) (14) were performed on the pond ash. The slake durability test resulted in 5 percent loss. Gradation specifications for dense-graded aggregate as well as gradations and characteristics of a pond ash material from one facility in Kentucky are shown in Figure 1. There was a disproportionate amount (outside specifications for compacted base) of plus-1-in. material in the pond ash. The large size of the coarse particles is an indication that the pond ash might be more suitable as a subbase material than as a base material.

Specimen Preparation

All specimens for this study were prepared in general accordance with ASTM C 593(79) in 4-in.-diameter by 4.6-in. molds. Deviations from that method involved the use of a 5.5-lb hammer and a 12-in. free-fall instead of the specified 10-lb hammer and 18-in. 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.

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.

All specimens prepared for or obtained from base course mixtures were submerged in water for 4 hr be-

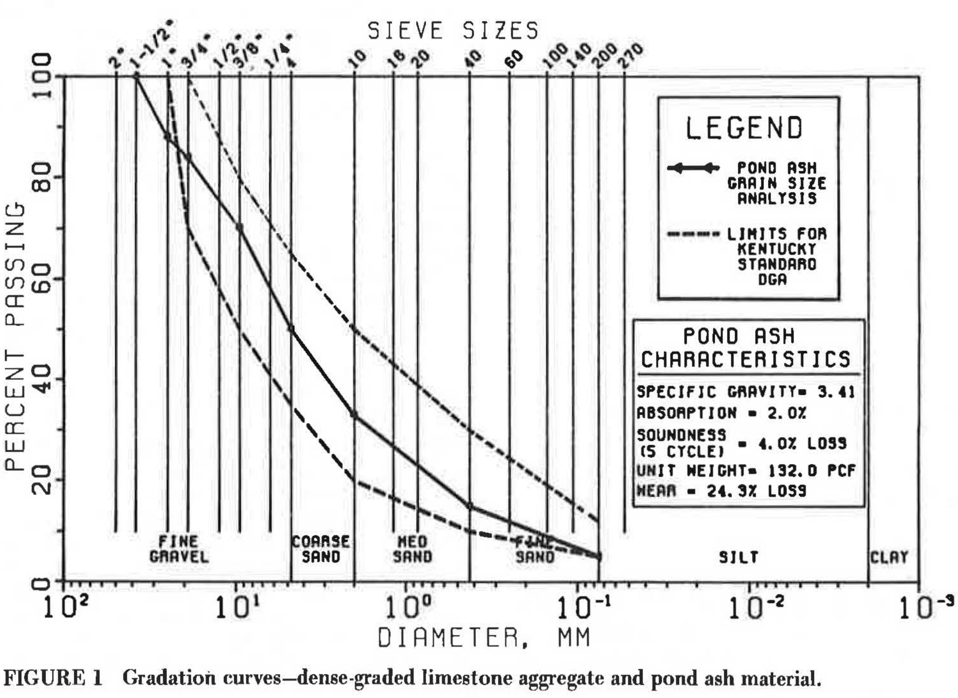

FIGURE 1 Gradation curves—dense-graded limestone aggregate and pond ash material.

fore testing for compressive strengths, as required by ASTM C 593(79). If slaking occurred, the materials or mixture proportions, or both, were eliminated from consideration as pavement components.

The only deviations from ASTM C 593(79) occurred when aggregate-scrubber sludge mixtures were tested. It was not possible to submerge sludge specimens, because some began to slake immediately on submergence. Slaking also prevented vacuum saturation or freeze-thaw testing. Strength testing of scrubber sludge was performed without submergence. This deficiency, although considered acceptable for material proposed as a subbase where confinement is provided by base and pavement layers, is not appropriate for base course construction. ASTM C 593(79) also specifies accelerated curing at 100°F in a sealed container. Other curing conditions included ambient curing and combinations of accelerated and ambient curing. Certainly, additional research is necessary to develop specifications and variations thereof to adequately reflect needed characterizations of materials for specific applications.

Testing

Unconfined compressive strength tests [ASTM C 39(72)], splitting tensile strength tests [ASTM C 496(71)], and tests for static-chord modulus [ASTM C 469(65)] were performed. During compressive strength tests, additional information was obtained by measuring deformation with deflection dial gauges. A four-point least-squares fitting technique was developed to calculate and plot the static-chord modulus of elasticity (Figure 2) from axial load and axial deformation data.

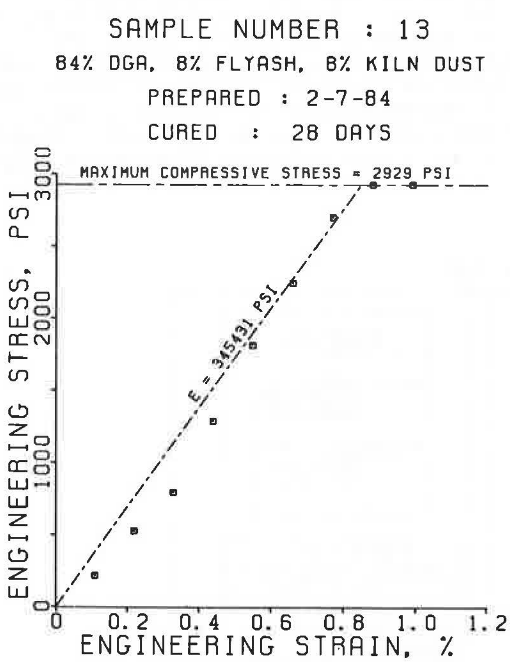

FIGURE 2 Example determination of static-chord modulus.

Attempts were made to measure lateral deformation during compressive strength testing for the purpose of obtaining data from which Poisson's ratio could be estimated. Poisson's ratio was estimated from the ratio of the slopes of the axial stress-axial strain curve and the axial stress-lateral strain curves. Techniques used to measure lateral strains, however, did not produce consistent and reliable results. Therefore, the literature was searched to determine the experience of others (10,15). Values from 0.08 to 0.3, depending on stress level as a percentage of the ultimate, were indicated. A Poisson's ratio of 0.15 was assumed for all pozzolanic base mixtures (16) until sufficient reliable test data for Kentucky mixtures could be accumulated. A summary of results of recent laboratory analyses in Kentucky is given in Table 1.

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 (7). A review of the literature has indicated considerable variability among suggested layer coefficients for pozzolanic materials (10,17,18). The range of suggested coefficients varies from 0.20 to 0.44 with most recommendations on 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 (19) 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 Modulus for Pozzolans

Early thickness designs using pozzolans in Kentucky were restricted to low-volume city street applications (20) and related well to other design methodologies. The same evaluations using thickness design procedures (based on static-chord modulus) for low-fatigue city street applications resulted in somewhat unrealistic thickness requirements when applied to high-fatigue design levels. Comparisons with other design methodologies also indicated reasonable correlations at low fatigue levels but wide variations for high-fatigue applications. However, there was concern that elastic layer parameters determined from some laboratory and field analyses did not completely account for the characteristics of pozzolanic materials.

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

Initial estimates of elastic moduli in this study were determined by the static-chord method [ASTM C 469(65)] and generally were relatively low (30,000 to 300,000 psi) (Figure 3). Elastic moduli for lime-fly ash mixtures reported elsewhere (10) were on the order of 100,000 to 500,000 psi for similar levels of compressive stresses (see Figure 3). Even greater magnitudes of elastic moduli (1,600,000 to 3,300,000 psi) have been reported by others (12).

TABLE 1 Strength Parameters for Various Pozzolanic Mixtures and for Various Curing Conditions

MIXTURE COMPONENTS (percent) FLY ASH	LIME KILN DUST	BY-PRODUCT LIME	SCRUBBER SLUDGE	RIVER SAND	DENSE-GRADED AGGREGATE	POND ASH	OPTIMUM MOISTURE CONTENT (percent)	MAXIMUM DRY DENSITY (pcf)	MIXTURE SOURCE (a)	CURING CONDITION (b)	UNCONFINED COMPRESSIVE STRENGTH (psi)	MODULUS OF ELASTICITY (psi)	SPLITTING TENSILE STRENGTH (psi)
--	--	--	10	--	--	90	11.8	126.2	field	No. 1	186	14,453	---
--	--	--	10	--	--	90	11.8	126.2	field	No. 7	557	83,836	---
--	--	--	15	--	--	85	9.5	143.6	field	No. 1	309	24,185	---
--	--	--	15	--	--	85	9.5	143.6	field	No. 7	670	37,526	---
--	--	--	20	--	--	80	11.7	133.5	field	No. 1	264	18,067	---
--	--	--	20	--	--	80	11.7	133.5	field	No. 7	560	29,029	---
--	--	--	30	--	--	70	12.4	128.3	field	No. 1	211	14,302	---
--	--	--	30	--	--	70	12.4	128.3	field	No. 7	393	58,306	---
--	--	--	100	--	--	--	43.7	71.6	field	No. 3	71	9,564	---
--	--	--	100	--	--	--	43.7	71.6	field	No. 1	98	11,430	---
--	--	--	100	--	--	--	43.7	71.6	field	No. 6	166	21,369	---
--	--	--	100	--	--	--	43.7	71.6	field	No. 7	130	10,814	---
--	--	--	100	--	--	--	43.7	71.6	field	No. 11	155	21,007	---
--	--	--	10	--	--	90	10.3	150.5	lab	No. 1	99	8,870	13
--	--	--	10	--	--	90	10.3	150.5	lab	No. 7	826	77,471	62
--	--	--	10	--	90	--	9.9	133.7	lab	No. 1	153	7,159	4
--	--	--	10	--	90	--	9.9	133.7	lab	No. 7	286	26,124	10
--	--	--	15	--	--	85	11.2	151.4	lab	No. 1	160	10,285	7
--	--	--	15	--	--	85	11.2	151.4	lab	No. 7	646	59,187	68
--	--	--	15	--	85	--	10.9	130.6	lab	No. 1	189	7,782	6
--	--	--	15	--	85	--	10.9	130.6	lab	No. 7	275	17,700	12
--	--	--	20	--	--	80	11.0	132.9	lab	No. 1	196	15,512	12
--	--	--	20	--	--	80	11.0	132.8	lab	No. 7	617	55,834	9
--	--	--	20	--	80	--	11.8	124.9	lab	No. 1	168	10,080	9
--	--	--	20	--	80	--	11.8	124.9	lab	No. 7	254	17,576	---
--	--	--	100	--	--	--	50.4	65.2	lab	No. 1	107	9,508	---
--	--	--	100	--	--	--	50.4	65.2	lab	No. 7	207	14,955	---
--	--	12	--	--	88	--	6.5	142.1	lab	No. 1	646	35,038	---
--	--	12	--	--	88	--	6.5	142.1	lab	No. 2	738	44,431	---
--	--	16	--	--	84	--	7.3	140.6	lab	No. 1	636	23,295	---
--	--	16	--	--	84	--	7.3	140.6	lab	No. 2	515	25,157	---
--	--	20	--	--	80	--	6.8	135.8	lab	No. 1	315	11,589	---
--	--	20	--	--	80	--	6.8	135.8	lab	No. 2	232	6,377	---
8	8	--	--	--	84	--	5.6	134.3	field	No. 1	1,192	87,545	---
8	8	--	--	--	84	--	5.6	134.3	field	No. 4	922	74,445	---
8	8	--	--	--	84	--	---	----	cores	No. 9	585	62,980	---
8	8	--	--	--	84	--	---	----	cores	No. 10	1,570	216,524	---
8	8	--	--	--	84	--	7.4	139.6	lab	No. 1	1,987	166,618	226
8	8	--	--	--	84	--	7.4	139.6	lab	No. 2	2,403	202,027	---
8	8	--	--	--	84	--	7.4	139.6	lab	No. 4	897	96,608	---
8	8	--	--	--	84	--	7.4	139.6	lab	No. 6	3,222	259,895	387
8	8	--	--	--	84	--	7.4	139.6	lab	No. 8	308	----	---
5	5	--	--	--	90	--	7.5	139.2	lab	No. 1	1,291	94,669	---
5	5	--	--	--	90	--	7.5	139.2	lab	No. 2	1,526	150,962	---
5	5	--	--	--	90	--	7.5	139.6	lab	No. 4	228	----	---
5	5	--	--	--	90	--	7.5	139.6	lab	No. 5	280	18,314	---
6	4	--	--	--	90	--	6.4	146.3	lab	No. 1	488	37,634	---
10	10	--	--	--	80	--	8.0	133.1	lab	No. 1	296	17,619	---
8	4	--	--	--	88	--	6.9	142.1	lab	No. 1	1,116	----	---
8	6	--	--	--	86	--	8.1	150.8	lab	No. 1	1,290	----	---
8	8	--	--	10	74	--	7.5	141.0	lab	No. 1	1,255	----	---
8	8	--	--	10	74	--	7.5	141.0	lab	No. 2	280	----	---
8	8	--	--	10	74	--	7.5	141.0	lab	No. 3	134	----	---
8	8	--	--	10	74	--	7.5	141.0	lab	No. 4	136	----	---
8	8	--	--	25	59	--	7.6	138.7	lab	No. 1	1,272	----	---
8	8	--	--	25	59	--	7.6	138.7	lab	No. 2	356	----	---
8	8	--	--	25	59	--	7.6	138.7	lab	No. 3	82	----	---
8	8	--	--	25	59	--	7.6	138.7	lab	No. 4	123	----	---
8	8	--	--	50	34	--	7.1	135.6	lab	No. 1	923	----	---
8	8	--	--	50	34	--	7.1	135.6	lab	No. 2	157	----	---
8	8	--	--	50	34	--	7.1	135.6	lab	No. 3	105	----	---
8	8	--	--	50	34	--	7.1	135.6	lab	No. 4	79	----	---
10	10	--	--	80	--	--	10.1	110.9	lab	No. 1	89	9,139	---
8	8	--	--	42	42(c)	--	7.0	138.1	lab	No. 1	349	14,956	---
8	8	--	--	32	52(d)	--	6.6	137.5	lab	No. 1	157	4,237	---
8	8	--	--	--	84(e)	--	8.0	135.1	lab	No. 6	1,317	127,193	---
8	8	--	--	42	42(e)	--	7.2	133.6	lab	No. 1	1,194	----	---
8	8	--	--	42	42(e)	--	7.2	133.6	lab	No. 2	69	----	---
8	8	--	--	42	42(e)	--	7.2	133.6	lab	No. 3	439	----	---

a. "Lab" refers to samples mixed from dry components in the laboratory, "field" refers to samples mixed in the laboratory with components from a field situation, "cores" refers to samples obtained by coring an existing pavement.

b. Curing conditions :

No. 1 -- 7 days at 100 F in a sealed container (ASTM C 593-79)
No. 2 -- 7 days at 100 F in a sealed container and then 7 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 100 F in a sealed container
No. 7 -- 7 days at 100 F in a sealed container and then 21 days at room temperature in air
No. 8 -- 28 days at room temperature in air
No. 9 -- 49 days ambient curing (field conditions) followed by a 14 day soaking period
No. 10 -- 132 days ambient curing (field conditions) followed by a 14 day soaking period
No. 11 -- 62 days at room temperature in air

c. No. 11 aggregate substituted for dense-graded aggregate.

d. Aggregate substituted for dense-graded aggregate consists of 32% No. 11, 20% aggregate meal.

e. Mines screenings substituted for dense-graded aggregate.

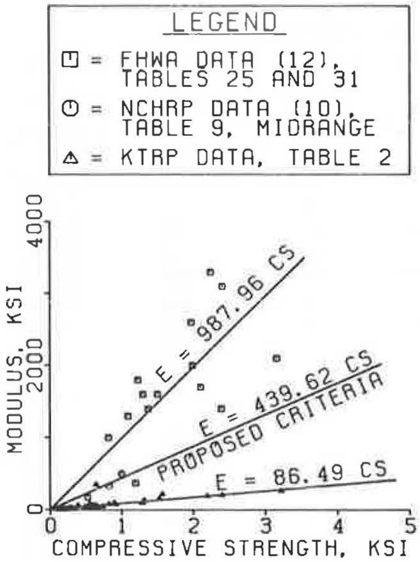

FIGURE 3 Modulus of elasticity versus compressive strength for various data sources.

Least-squares regression analyses were used to evaluate trends of modulus of elasticity versus unconfined compressive strength and tensile strength for the various sources of data (Figure 3). The Kentucky relationship (for the static-chord modulus) is most conservative and was developed for a number of pozzolanic base mixtures evaluated in Kentucky. Resilient moduli presented in the FHWA report showed the greatest rate of change of modulus per unit of compressive stress whereas data from the NCHRP report indicated a somewhat lesser rate of change. Data presented in the FHWA report (12) are resilient moduli determined by repeated load testing for a range of fly ash-kiln dust ratios and also a variety of sources of fly ash and kiln dusts (lime and cement kiln dusts). Figures 3 and 4 show trends of resilient modulus as a function of unconfined compressive strength and splitting tensile strength for all data. Additional plots have been developed for specific mixture proportions or components. Elastic moduli presented elsewhere (10, Table 9) were determined from plate load tests. Median moduli and compressive strengths were used to develop the relationship shown in Figure 3. The relationship of compressive strength versus modulus of elasticity based on data reported in the NCHRP synthesis was selected as a "middle-of-the-road" criterion to determine design elastic moduli. Additional research is necessary to verify and refine this design criterion.

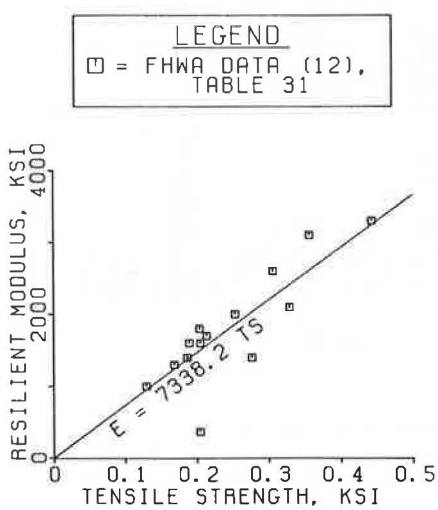

FIGURE 4 Resilient modulus as a function of splitting tensile strength.

Kentucky laboratory analyses were based on ASTM C 469(65) and resulting values were essentially static moduli of elasticity. 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. However, preliminary analyses indicate backcalculated moduli of 1 million to 3 million psi.

Ahlberg and Barenburg (15) reported flexural moduli of elasticity from 1,500,000 to 2,500,000 psi. Resilient moduli reported by Collins and Emery (12) varied from 370,000 to 3,300,000 psi. Others (10) have reported ranges of moduli from 100,000 psi at a compressive strength of 400 psi to a modulus of 500,000 psi at a compressive strength of 1,000 psi.

The modulus of elasticity of asphaltic concrete varies as a function of temperature and frequency of loading (21,22). On the other hand, granular cohesionless materials have relatively constant moduli for frequencies of 0.1 to 50 Hz (23). For a soil that may be considered to behave as a linear viscoelastic solid, the elastic modulus is a function of frequency (24). Hardin and Black (25,26) have demonstrated dramatic variations of elastic moduli of cohesive soils at low frequencies (less than 0.1 Hz) because of creep phenomena. This partly explains observed variations in elastic moduli from static and dynamic tests. Furthermore, it also has been demonstrated that modulus varies as a function of strain amplitude (23,25,26), which varies considerably among test procedures.

Static moduli were not considered representative of actual traffic loading conditions. Resilient moduli are determined on the basis of repeated load tests at 1 to 2 Hz. Road Rater deflections were obtained at 25 Hz using a 600-lb-force dynamic load and a 1,670-lb-force static load. Others (15) estimated elastic moduli from tests for flexural strength. In view of the significant variations of both frequency and strain amplitude of actual traffic loadings, the need at this time for conservative design moduli is apparent. In addition, Kentucky thickness design procedures, although predicated on a limiting strain-repetitions criterion, were verified initially by Benkelman beam deflection behavior where rebound deflections were obtained at low (creep) vehicle speeds (0.5 to 1.0 Hz) for an 18,000-lb axle load and were matched with theoretical deflections calculated using the Chevron N-layer program (27). Thus an interim criterion relating compressive strength and modulus of elasticity is shown in Figure 3.

Suggested Design Methodology

Thickness design procedures (flexible and rigid) in Kentucky have been developed on the basis of limiting strain-repetitions criteria. 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 (1,2,28). The rigid pavement design criterion is an expression of a stress-ratio fatigue criterion (3-5) in terms of tensile strain versus repetitions for various combinations of modulus of elasticity and modulus of rup-

ture. The same approach was used to develop a tensile strain-repetitions criterion for pozzolanic base materials (Figure 5).

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 (15). Based on current Kentucky specifications of a minimum compressive strength of 600 psi, the flexural stress for pozzolanic base materials is

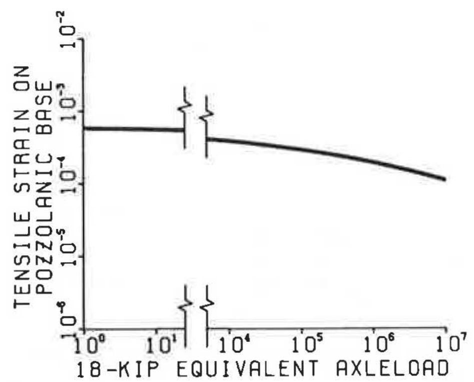

FIGURE 5 Limiting tensile strain at bottom of the pozzolanic base versus repetitions of an 18,000-lb axle load.

150 psi. The minimum design modulus of elasticity from Figure 3 is 250,000 psi. The assumed Poisson's ratio of 0.15 for pozzolanic materials (16) is near the value used to develop rigid pavement designs in Kentucky. The shape of the fatigue envelope used by the Portland Cement Association (6) for portland cement concrete pavements was applied to pozzolanic materials and defines the relationship of ratio of allowable tensile stress to repetitions of an 18,000-lb single axle load (3-5). The allowable tensile stress versus repetitions relationship for pozzolanic materials is the Portland Cement Association curve shifted according to the following relationship:

Tensile strain = (Flexural strength) x (Stress ratio) ÷ (Modulus of elasticity)

where the stress ratio value corresponds to a specific value for repetitions of an 18,000-lb equivalent axle load. More specifically, for pozzolanic base mixtures,

Tensile strain = (150 psi) x (Stress ratio) ÷ (250,000 psi)

These equations convert the ratio of allowable stress ratio to allowable tensile strain at the bottom of the pozzolanic base (see Figure 5). The resulting criterion, compared to one proposed by Thompson (19), is slightly more conservative. Experience with pozzolanic pavements in Kentucky has been limited; the proposed criterion also may be adjusted on the basis of field performance.

Recent studies (3-5) have involved the application of work and energy principles to combine all strain components into a single resultant. Strain energy density is the energy at a point in a body to resist the energy imposed on that body by an outside load and is equal and opposite to the work at that point, as defined by classical physics (29,30). The strain energy density for each point in the pavement structure must be summed (integrated) to obtain the total strain energy, which would equal the total work caused by the external force. Strain energy density, or work, at a given location within the structure may be used as the basis of design instead of using a single strain, such as the vertical compressive strain, as the criterion. Recent investigations of both flexible and rigid pavements have used concepts of equal work as the basis of thickness designs.

To develop a design procedure that uses pozzolans in the pavement structure, the elastic layer theory embodied in the Chevron N-layer computer program (27) was used first to determine thickness requirements for conventional designs (1/3 asphaltic concrete and 2/3 crushed stone base) using traditional materials (1,2,28). Work at critical locations--bottom of the asphaltic concrete or top of the subgrade, or both--was determined and used as the controlling fatigue value for the respective materials at their critical locations.

Elastic layer theory then was used to determine strains and work for a matrix of thicknesses of pozzolanic base (of varying moduli of elasticity) combined with several thicknesses of asphaltic concrete surfacing. Results of those analyses were used to develop a series of graphs similar to Figures 6 and 7.

Determination of an equivalent structural thickness design that uses pozzolans may be achieved by matching the critical strains and work for a conventional pavement design with companion work and strains for some combination of thicknesses of asphaltic concrete surfacing and pozzolanic base. The work and the vertical compressive strain at the top of the subgrade for the control (conventional) pavement were used in combination with Figures 6 and 7 (and other similar graphs) to determine thicknesses of pozzolanic bases corresponding to specific elastic moduli and various thicknesses of asphaltic concrete surfacing. Thicknesses of pozzolanic bases will be slightly increased using a criterion based on work compared to vertical compressive strain. Resultant thicknesses of pozzolanic bases were used to develop Figure 8. The specific thickness of pozzolanic base may then be determined depending on the desired modulus of elasticity and the desired thickness of asphaltic concrete surfacing. Modulus of

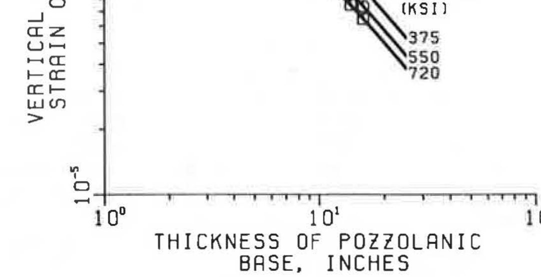

FIGURE 6 Vertical compressive strain at top of subgrade versus thickness of pozzolanic base for constant modulus of elasticity of pozzolanic base and constant thickness of asphaltic concrete surfacing.

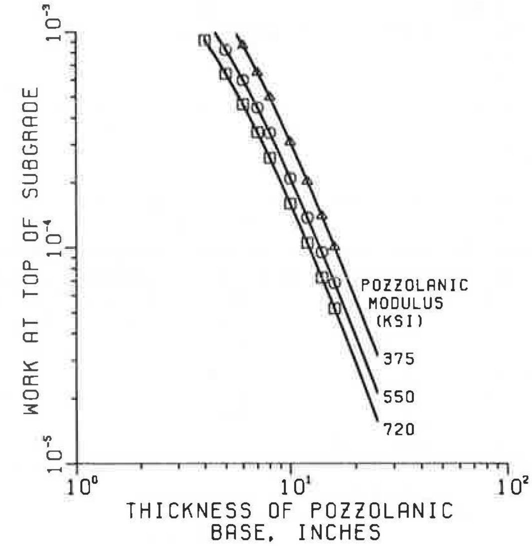

FIGURE 7 Work at top of subgrade versus thickness of pozzolanic base for constant modulus of elasticity of pozzolanic base and constant thickness of asphaltic concrete surfacing.

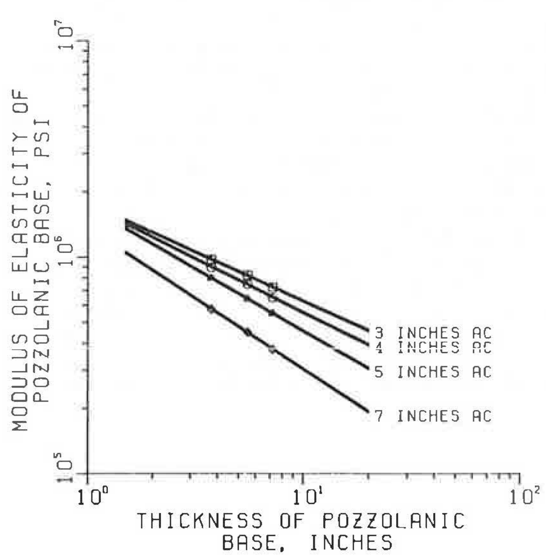

FIGURE 8 Equivalent thickness designs using pozzolanic base for varying thicknesses of asphaltic concrete and moduli of pozzolanic base materials.

elasticity may be related to compressive strength by Figure 3.

Design thicknesses (based on the work at the top of the subgrade) were checked against the limiting tensile strain criterion (1,2,28) at the bottom of the asphaltic concrete and the limiting tensile strain at the bottom of the pozzolanic base (Figure 5) to verify that fatigue of the asphaltic concrete and pozzolanic base was not controlling. Experience in Kentucky has shown that tensile strain at the bottom of the asphaltic concrete layer is normally not the controlling design criterion because the relatively "stiff" moduli of pozzolanic bases limit the magnitude of tensile strains at the interface between asphaltic concrete and pozzolanic base.

EXAMPLE DESIGN

Phase 1 of the design procedure involves the determination of thickness requirements for a conventional asphaltic concrete pavement. Consider, for example, the following design conditions:

Design 18-kip equivalent axle loads (EALs) = 5 million

Design subgrade = California bearing ratio (CBR) 9

Using Kentucky thickness design curves, a conventional asphaltic concrete pavement would be as follows:

Asphaltic concrete surface or base, or both = 7 in.

Dense-graded aggregate base = 14 in.

In Phase 2, structurally equivalent designs using pozzolanic base materials are determined. Critical strains and work for conventional designs are determined using the Chevron N-layer computer program (Figures 6 and 7). Limiting strains corresponding to those of conventional structures may be used to determine thickness requirements for pozzolanic bases for a constant thickness of asphaltic concrete. Analyses of a number of asphaltic concrete thicknesses may be used to develop Figure 8. The specific thickness design is based on estimated elastic modulus obtained from an analysis of compressive strength data (Figure 3).

The major benefit associated with the use of a pozzolanic base is the substitution of a less expensive material for a portion of a more expensive component of the pavement structure. Pozzolanic bases may be especially advantageous as alternatives to very thick conventional asphaltic concrete pavements or thick full-depth asphaltic concrete pavements for which deep rutting may be a potential problem. Pozzolanic bases also may be a cost-effective alternative to some rigid pavements. This, however, has not been considered in Kentucky.

OTHER FACTORS

Effects of Curing

Effects of curing were detected in the field by deflection measurements. Table 2 gives a summary of deflection data obtained directly on a 6-in. layer of lime kiln dust-fly ash-dense-graded aggregate base for three city street projects. Design proportions for Sites 1 and 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

TABLE 2 Road Rater Deflections on 6-in. Pozzolanic Bases

	Deflections (in. x 10^{-5}) at Sensor Number		
Project No.	1	2	3
1	53.8	29.8	15.1
2	118.5	46.0	24.8
3	147.2	56.9	24.3

all sites: 7 to 9 days after placement. Prior laboratory and field data indicated that subgrade conditions were similar for the three projects (CBR 4).

Site 1 was placed in mid-August and curing conditions were 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 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 greater strengths resulted from more favorable curing conditions. Deflection data also indicated the influence of the bituminous curing membrane on proper curing and associated strength gains.

Both laboratory (Table 1) and the field data (Table 2) 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 above 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

Another aspect associated with low-strength pozzolanic base materials is the potential for reflective cracking of the overlying asphaltic concrete surfacing. It is anticipated that greater amounts of cracking will occur during curing of higher strength pozzolans.

Results of the deflection testing of the three test sites stimulated additional interest in the effects of curing and autogenous healing. A series of lime kiln dust-fly ash-dense-graded aggregate mixtures was prepared in 6-in.-diameter by 12-in. cylinders and cured at room temperature for 28 days. Compressive strengths of those specimens were 231 psi for Mixture A and 209 psi for Mixture B. The aggregate portion for Mixture A consisted of 84 percent dense-graded limestone; Mixture B contained 42 percent sand and 42 percent limestone mine screenings. The fine portions of both mixtures contained 8 percent each of fly ash and lime kiln dust. That was considerably less than for specimens compacted and cured according to ASTM C 593 (4-in.-high by 4.6-in.-diameter cylinder cured at 100°F for 7 days). Compressive strengths in those cases were 1,501 psi for Mixture A and 1,194 psi for Mixture B. The 6- by 12-in. cylinders tested for compressive strengths at 7 days were not destroyed but were sealed in plastic bags and cured to an age of 240 days. The cylinders were again subjected to compressive testing. Compressive strengths at that time were 870 psi for Mixture A and 1,367 psi for Mixture B. Significant strength gains may be partly attributable to long-term strength gain characteristics of pozzolanic materials and also to autogenous healing of the initial failure locations.

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

CLOSING COMMENTS

Experience in Kentucky with 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 could not be determined at this time. This paper represents initial efforts to develop a thickness design methodology for pozzolanic pavements that is related to performance histories in Kentucky as well as to laboratory test data characterizing the properties of pozzolans. It is anticipated that the procedures presented herein may be the nucleus for the development of a complete set of thickness design curves using pozzolanic or other low-strength base materials.

Additional research on and experience with 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 overlying asphaltic concrete layers associated with shrinkage cracking in the pozzolanic base. Additional evaluations are currently ongoing. One technique involves the use of stress-relief layers between the pozzolanic and the asphaltic concrete layers. Benefits are yet to be determined.

REFERENCES

1. J.H. Havens, R.C. Deen, and H.F. Southgate. Design Guide for Bituminous Concrete Pavement Structures. Research Report UKTRP-81-17. Kentucky Transportation Research Program, University of Kentucky, Lexington, Aug. 1981.
2. H.F. Southgate, R.C. Deen, and J.H. Havens. Development of a Thickness Design System for Bituminous Concrete Pavements. Research Report UKTRP-81-20. Kentucky Transportation Research Program, University of Kentucky, Lexington, Nov. 1981.
3. H.F. Southgate, J.H. Havens, R.C. Deen, and D.C. Newberry, Jr. Development of a Thickness Design System for Portland Cement Concrete Pavements. Research Report UKTRP-83-5. Kentucky Transportation Research Program, University of Kentucky, Lexington, Feb. 1983.
4. H.F. Southgate and R.C. Deen. Thickness Design Curves for Portland Cement Concrete Pavements. Research Report UKTRP-84-3. Kentucky Transportation Research Program, University of Kentucky, Lexington, Feb. 1984.
5. H.F. Southgate and R.C. Deen. Thickness Design Procedure for Portland Cement Concrete Pavements. Research Report UKTRP-84-6. Kentucky Transportation Research Program, University of Kentucky, Lexington, March 1984.
6. Thickness Design for Concrete Pavements. Portland Cement Association, Skokie, Ill., 1966.

7. Interim Guide for Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, DC, 1972 and 1981.
8. The AASHO Road Test: Report 5--Pavement Research. Special Report 61E. HRB, National Research Council, Washington, D.C., 1962.
9. Fly Ash Stabilized Bases. Special Provision 83(70). Kentucky Transportation Cabinet, Frankfort.
10. Lime-Fly Ash-Stabilized Bases and Subbases. NCHRP Synthesis of Highway Practice 37. TRB, National Research Council, Washington, D.C., 1976.
11. Chemical Lime Facts. Bulletin 214, 3rd ed. National Lime Association, Arlington, Va., 1973.
12. R.J. Collins and J.J. Emery. Kiln Dust-Fly Ash Systems for Highway Bases and Subbases. Report FHWA/RD-82/167. FHWA, U.S. Department of Transportation, Sept. 1983.
13. Standard Specifications for Road and Bridge Construction. Kentucky Transportation Cabinet, Frankfort, 1983.
14. Slake Durability Index Test. Method KM-64-513-76. Kentucky Transportation Cabinet, Frankfort, 1976.
15. H.L. Ahlberg and E.J. Barenburg. Pozzolanic Pavements. Bulletin 473. Engineering Experiment Station, University of Illinois, Urbana, 1965.
16. E.J. Barenburg. Lime-Fly Ash-Aggregate Mixtures in Pavement Construction. National Ash Association, Washington, D.C., 1974.
17. M. Gomez and M.R. Thompson. Structural Coefficients and Thickness Equivalency Ratios. University of Illinois, Urbana, June 1983.
18. T.D. Larson and E.S. Lindow. An Evaluation of Pennsylvania's Flexible Pavement Design Methodology. Pennsylvania Transportation Institute, Pennsylvania State University, State College, Dec. 1974.
19. M.R. Thompson. Concepts for Developing a Nondestructive Testing Based Asphalt Concrete Overlay Thickness Design Procedure. University of Illinois, Urbana, June 1982.
20. G.W. Sharpe, H.F. Southgate, and R.C. Deen. Development of Pavement Thickness Designs Using Pozzolanic Base Materials. Research Report UKTRP-83-25. Kentucky Transportation Research Program, University of Kentucky, Lexington, Oct. 1983.
21. B.F. Kallas and J.C. Riley. Mechanical Properties of Asphalt Pavement Materials. Proc., Second International Conference on the Structural Design of Asphalt Pavements, University of Michigan, Ann Arbor, 1967.
22. J.F. Shook and B.F. Kallas. Factors Influencing Dynamic Modulus of Asphaltic Concrete. Proc., Association of Asphalt Paving Technologists, Vol. 38, 1969.
23. V.P. Drnevich and J.P. Jent. Response of Saturated Sands to Cyclic Shear at Earthquake Amplitudes. Water Resources Research Institute Report 87. University of Kentucky, Lexington, Oct. 1975.
24. F.E. Richart, Jr., J.R. Hall, Jr., and R.D. Woods. Vibrations of Soils and Foundations. Prentice-Hall, Inc., Englewood Cliffs, N.J., 1970.
25. B.O. Hardin and W.L. Black. Vibration Modulus of Normally Consolidated Clay. Journal of the Soil Melchanics and Foundation Division, ASCE, Vol. 94, No. SM2, March 1968.
26. B.O. Hardin and W.L. Black. Closure to Vibration Modulus of Normally Consolidated Clay. Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 95, No. SM6, Nov. 1969.
27. J. Michelow. Analysis of Stresses and Displacements in an N-Layered Elastic System under a Load Uniformily Distributed on a Circular Area. Chevron Research Corporation, Richmond, Calif., Sept. 1963.
28. R.C. Deen, H.F. Southgate, and J.H. Havens. Structural Analysis of Bituminous Concrete Pavements. Research Report 305. Division of Research, Kentucky Department of Highways, Lexington, May 1971.
29. I.S. Sokolnikoff. Mathematical Theory of Elasticity. 2nd ed. McGraw-Hill Book Co., New York, 1956.
30. R.C. Deen, H.F. Southgate, and J.G. Mayes. The Effect of Truck Design on Pavement Performance. Proc., Association of Asphalt Paving Technologists, Vol. 49, 1980.

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