NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
SYNTHESIS OF HIGHWAY PRACTICE 37

LIME-FLY ASH-STABILIZED BASES AND SUBBASES

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:
- PAVEMENT DESIGN
- CONSTRUCTION
- AIR TRANSPORT

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1976

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Research Council was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as: it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state, and local governmental agencies, universities, and industry; its relationship to its parent organization, the National Academy of Sciences, a private, nonprofit institution, is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the Academy and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are responsibilities of the Academy and its Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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PREFACE

There exists a vast storehouse of information relating to nearly every subject of concern to highway administrators and engineers. Much of it resulted from research and much from successful application of the engineering ideas of men faced with problems in their day-to-day work. Because there has been a lack of systematic means for bringing such useful information together and making it available to the entire highway fraternity, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize the useful knowledge from all possible sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series attempts to report on the various practices without in fact making specific recommendations as would be found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available concerning those measures found to be the most successful in resolving specific problems. The extent to which they are utilized in this fashion will quite logically be tempered by the breadth of the user's knowledge in the particular problem area.

FOREWORD

By Staff
Transportation Research Board

This synthesis will be of special interest and usefulness to design, construction, materials, and maintenance engineers seeking technical information on the use of lime-fly ash–aggregate materials for stabilized pavement bases and subbases. Detailed information is presented on the materials, mixture properties, selection of proportions, construction procedures, and pavement behavior and performance. Applications and limitations for the use of lime-fly ash–aggregate materials in pavement construction are outlined.

Administrators, engineers and researchers are faced continually with many highway problems on which much information already exists either in documented form or in terms of undocumented experience and practice. Unfortunately, this information often is fragmented, scattered, and unevaluated. As a consequence, full information on what has been learned about a problem frequently is not assembled in seeking a solution. Costly research findings may go unused, valuable experience may be overlooked, and due consideration may not be given to recommended practices for solving or alleviating the problem. In an effort to resolve this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of synthesizing and reporting on common highway problems—a synthesis being identified as a composition or combination of separate parts or elements so as to form a whole greater than the sum of the separate parts. Reports from this endeavor constitute an NCHRP report series that collects and assembles the various forms of information into single concise documents pertaining to specific highway problems or sets of closely related problems.

In recent years, energy and environmental considerations have brought about an increased interest in the use of lime and fly ash in pavement construction. A well-developed technology now exists for the stabilization of bases and subbases with these materials. However, lime-fly ash–aggregate materials are sometimes not used when they might because technical information has not been conveniently available.

This report of the Transportation Research Board describes current technology for construction of stabilized bases and subbases using lime-fly ash–aggregate materials. Typical specifications for mixing and placing the materials are described.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the researchers in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that which is now at hand.

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John W. Guinnee, Engineer of Soils, Geology, and Foundations, Transportation Research Board, assisted the Special Projects staff and the Topic Panel.

Information on current practice was provided by many transportation agencies. Their cooperation and assistance was most helpful.

LIME-FLY ASH-STABILIZED BASES AND SUBBASES

SUMMARY

Technology for stabilizing aggregates with lime and fly ash has been increasing during the past 20 years. Many state and federal agencies now include this paving material in their specifications. However, because this technology is not widely known, many agencies with sources of lime and fly ash do not make extensive use of these materials. Factors that are likely to influence the future use of lime-fly ash aggregates (LFA) are:

- Increase in use of coal for fuel.
- Low energy requirements for producing LFA mixes.
- New technology for LFA use.
- Widespread availability of lime and fly ash.

Most commercially available hydrated limes are suitable for LFA mixes. In addition, some by-product limes can be used. Although there are differences in the properties of these limes, most of them can be used in LFA mixtures, but each lime source should be validated before approval for use.

Fly ash is the fine residue that results from the combustion of coal; it is collected from flue gases. It has been estimated that the U.S. fly ash production will approach 40 million tons (3.6×10^7 metric tons) annually by 1980. Fly ashes are pozzolans with little cementitious value but in the presence of moisture will chemically react with calcium hydroxides at ordinary temperatures to form compounds with cementitious properties.

"Dry" fly ash is taken directly from the precipitator or from dry storage. Fly ash stockpiled in the open requires the addition of water to prevent dusting. As a result of alkalies present in some fly ashes, dampened fly ashes may set up and require crushing and screening before use. Fly ash is also stored in slurry ponds, and may segregate during settlement causing the final product to be more variable.

The physical and chemical properties of fly ash are highly variable. Some of these are:

- Shape (spherical, solid, or hollow).
- Glass content.
- Composition (silica and aluminum plus carbon; and oxides of iron, calcium, magnesium, and sulfur).
- Size (1 μm to 80 μm).
- Color (gray, tan, black).

Agencies usually cite ASTM Specification C 593 in their specification for fly ash.

The quality of a lime-fly ash-stabilized mixture is highly dependent on the

aggregates used; crushed stone, gravel, and granular sands produce better mixes than silts. (Fine-grained clays are not normally used with LFA mixtures.) Other factors that affect the stability and strength of the LFA mixture include: gradation, plasticity index, liquid limit, soundness, compactive effort, and curing.

Generally, the lime-plus-fly ash content of a mixture ranges from 12 percent to 30 percent with lime-to-fly ash ratios of 1:3 to 1:4 being common.

Critical engineering properties of LFA mixtures include: strength (compressive, flexural, and shear), modulus of elasticity, Poisson's ratio, fatigue, autogenous healing, volume changes, and durability. These properties are influenced by the constituent materials (lime, fly ash, aggregate, and soil) and by proportions, processing, compaction, and curing (time, temperature, and moisture).

Test data on full-scale and model pavements correlated with theoretical analyses confirm that the load distribution characteristics of pavements with layers of LFA mixes are essentially those of a slab. The ultimate strength of LFA slabs under static load has been shown to far exceed the strength predicted by elastic slab theory. The stiffer LFA layers distribute the load over large areas of the subgrade by the slab action, thus transmitting low vertical stress to the subgrade.

The primary factors affecting the performance of LFA pavements are: loading and the interrelationships between load, slab thickness, and material strength; durability of the LFA material as related to the environment in which it must serve; quality of construction, including uniformity of the final product; and subsurface drainage of the pavement system.

Performance of LFA pavements has been studied using: scale models in laboratory and quasi-laboratory conditions, short-term evaluation of full-scale pavements with vehicle loading under simulated service conditions, and performance of pavements in service for a number of years under normal traffic. One objective of the studies of LFA performance has been the identification of structural coefficients to be used in designing LFA pavement systems.

Distress in LFA pavements is normally surface cracking caused by excessive loads on the pavement and deterioration of the LFA material, which is often the result of excess moisture and inadequate density (particularly near the pavement edge).

Two types of construction-related distress have been observed on LFA pavements. One type occurs when the material becomes saturated before setting and shoves under traffic. This is corrected by reshaping and compacting the unstable layer and permitting it to dry. A more harmful type of distress is caused when construction traffic causes cracks in partially set LFA material. Under favorable curing conditions these cracks will reheal, but with a reduction in the ultimate strength of the pavement.

Exact proportions of lime, fly ash, and aggregate are included in LFA mixtures to satisfy specific requirements. For a given set of materials, several proportions may provide mixtures of satisfactory quality.

Advantages of using LFA mixtures in pavement construction include ease of construction and the ability to use conventional construction equipment. The essential construction requirements are thorough mixing, uniform spreading, and compaction to a high density. LFA materials can be blended or mixed on the roadbed or in a central plant. Compacted layers of LFA materials should be sealed as soon as possible.

LFA materials are most effective when used under proper conditions and within specified limitations. Some general guidance is offered:

- LFA materials can be used for a wide range of pavement systems from low volume to heavy volume.
- LFA materials can be used as a base or subbase for flexible pavements, or as a subbase for rigid pavements.
- The key to good performance with LFA materials is good mixture selection and sound construction techniques.
- Durability is the single most important consideration in the performance of LFA materials, especially in areas of cyclic freezing and thawing and where use of deicing salt is heavy.
- Procedures are required for establishing cutoff dates for late-season construction with LFA materials.
- High relative density is critical for high strength and durability.

CHAPTER ONE

INTRODUCTION

The technology for stabilizing aggregates with lime and fly ash has been growing at a significant pace during the past 20 years. The state of the art in this technology has developed to the point that lime-fly ash–aggregate (LFA) materials are included in many state and federal agency specifications as a conventional type of paving material.

Despite the development of the technology, its acceptance has not been universal. In several areas of the country, substantial quantities of LFA materials are used annually in the construction of pavement bases and subbases, but in other areas with excellent resources of lime and fly ash, the materials are not used extensively. There are, no doubt, a number of reasons for this, including the fact that the technology for use of this material is not widely known in the profession. One of the primary functions of this synthesis is to summarize the current state of the art on LFA materials in a readily available format.

Use of LFA materials has increased in recent years because of concern for energy resources and the environment. Currently, more than 35 million tons of fly ash are disposed of annually in an environmentally acceptable manner. As more utilities are forced to shift from gas and oil to coal as a source for fuel, quantities of fly ash will increase. In addition, because LFA mixes generally require only 2 to 5 percent lime, the total energy used in the production of LFA mixes is very low. These two factors along with a well-developed technology should make an attractive case for the expanding use of LFA mixes.

Lime and fly ash are not locally available in all areas of the United States. The distribution of lime and fly ash is widespread enough, however, that LFA materials could be produced in most areas of the country with local materials and reasonable haul distances of only one of the components. Appendix A shows the availability of lime and fly ash around the country.

CHAPTER TWO

MATERIALS

Lime and fly ash can be used to stabilize aggregates and soils to produce acceptable quality base and subbase materials. The characteristics of the lime, fly ash, and aggregate or soil being stabilized significantly influence the quality and properties of the stabilized material. Thus, it is important to carefully evaluate and consider all of the various mixture components (lime, fly ash, and aggregate or soil) and their interactions.

LIME

In general, the term lime refers to oxides and hydroxides of calcium and magnesium, but not to carbonates. There are various types of lime commercially available. Calcitic quicklime (CaO) and dolomitic quicklime (CaO + MgO) are produced by calcining calcitic and dolomitic limestone, respectively. By the controlled addition of water to quicklime, three types of hydrated lime can be produced: high-calcium, $Ca(OH)_2$; monohydrated dolomitic, $Ca(OH)_2 + MgO$; and dihydrated dolomitic, $Ca(OH)_2 + Mg(OH)_2$. Only hydrated high-calcium and monohydrated dolomitic limes are used in lime-fly ash stabilization. Quicklimes are not used. For a comprehensive consideration of lime and the lime production process, consult Boynton (*1*). Typical properties of commercial varieties of quicklime and hydrated limes are summarized in Table 1.

By-product lime also provides a source of lime that is

TABLE 1

PROPERTIES OF COMMERCIAL LIMES *

(a) Quicklime

Constituent (percent)	High Calcium	Dolomitic
CaO	92.25 - 98.00	55.70 - 57.50
MgO	0.30 - 2.50	37.60 - 40.80
CO_2	0.40 - 1.50	0.40 - 1.50
SiO_2	0.20 - 1.50	0.10 - 1.50
Fe_2O_3	0.10 - 0.40	0.05 - 0.40
Al_2O_3	0.10 - 0.50	0.05 - 0.50
H_2O	0.10 - 0.90	0.10 - 0.90
Specific Gravity	3.2 - 3.4	3.2 - 3.4
Specific heat at 100 F (38 C)	Btu/lb 0.19 (J/kg) (440)	Btu/lb 0.21 (J/kg) (488)
Bulk Density, pebble lime	pcf 55-60 (kg/m^3) (880-960)	pcf 55-60 (kg/m^3) (880-960)

(b) Hydrates

	High Calcium	Monohydrated Dolomitic	Dihydrated Dolomitic
Principal constituent	$Ca(OH)_2$	$Ca(OH)_2$+MgO	$Ca(OH)_2$+$Mg(OH)_2$
Specific gravity	2.3 - 2.4	2.7 - 2.9	2.4 - 2.6
Specific heat at 100 F (38 C)	Btu/lb 0.29 (J/kg) (675)	Btu/lb 0.29 (J/kg) (675)	Btu/lb 0.29 (J/kg) (675)
Bulk density	pcf 25-35 (kg/m^3) (400-560)	pcf 25-35 (kg/m^3) (400-560)	pcf 30-40 (kg/m^3) (480-640)

*Data taken from "Chemical Lime Facts," Bulletin 214 (3rd ed.), National Lime Association (1973).

often suitable for use in stabilization. This type of lime is usually available from various manufacturing processes. Two types of by-product limes commonly available are: (*a*) that collected from the draft of the calcining process in lime production operations (flue dust), and (*b*) the by-product of acetylene gas production from calcium carbide. By-product lime may be a very economical source of lime; however, these limes may be nonuniform in quality (*2*).

Although many by-product limes may be similar to virgin limes in terms of chemical composition, other important properties may be considerably different. For example, commercial hydrates generally are more finely divided and have higher specific surfaces than carbide limes.

A new form of lime for stabilization has recently been developed in the Chicago area. The material is a by-product hydrate produced by hydrating a mixture of flue dust and normal quicklime. Although the by-product hydrate is not chemically equivalent to normal commercial hydrated lime, it has been successfully used in lime-fly ash stabilization. By-product hydrate is less expensive and more readily available in the Chicago area.

There is some concern as to whether calcitic lime, $Ca(OH)_2$, or monohydrated dolomitic lime, $Ca(OH)_2 + MgO$, is the more effective lime for use in lime-fly ash stabilization. Studies indicate that monohydrated dolomitic lime is more effective than high-calcium lime (*3, 4*), but both limes produce long-term strengths of approximately equal magnitude. Other investigators (*4, 5, 6*) have found that high-calcium lime gives higher strengths, especially at low lime contents. Thus, it can probably be concluded that either high-calcium or monohydrated dolomitic lime is, in general, satisfactory for use in lime-fly ash stabilization. Laboratory testing may be used to indicate the effectiveness of any of the lime types, but it should be emphasized that the quality of the fly ash has a much greater influence on the lime-fly ash pozzolanic reaction than does lime type.

It can be stated that most types of lime (exclusive of dihydrated dolomitic) are appropriate if a quality lime-fly ash-stabilized product meeting strength, durability, and economic criteria can be obtained. Appropriate quality control testing should be conducted during the course of a project to ensure the quality and uniformity of the lime being incorporated into the LFA mixture. Typical lime specifications are summarized in the following section.

Lime Specifications for Lime-Fly Ash–Aggregate Mixtures

Illinois Department of Transportation

The lime, either high-calcium or dolomitic hydrate, shall comply with the requirements of ASTM C 207, Hydrated Lime for Masonry Purposes, Type N, with the following modifications:

(1) Total calcium and magnesium oxides (nonvolatile basis) minimum percent 90
(2) Calcium oxide in hydrated lime (as-received basis) maximum percent 5
(3) Magnesium hydroxide (as-received basis) maximum percent 5
(4) Mechanical moisture in hydrated lime (as-received basis) maximum percent 4
(5) The sieve analysis of the lime residue shall be as follows:

SIEVE	MAXIMUM PERCENT RETAINED
No. 4 (4.75 mm)	0
No. 30 (600 μm)	2.5
No. 100 (150 μm)	15

Pennsylvania Department of Transportation

Lime shall meet the requirements of ASTM Designation C 207, Type N, Sections 2, 3(a), 6, and 7(a) and shall be capable of producing a mixture meeting Pennsylvania aggregate-lime-pozzolan mixture requirement.

Ohio Department of Transportation

Hydrated lime shall meet the requirements prescribed in standard specification for Hydrated Lime for Masonry, ASTM C 207, Type N for chemical composition, residue, sampling, inspection and methods of test. (Sections 3.1, 2, 4, and 5 are not relevant to the intended usage.)

Federal Aviation Administration

The lime shall meet ASTM Specification C 207, Type N, Sections 2 and 3(a) when sampled and tested in accordance with Sections 6 and 7. The above requirements may be waived if it is demonstrated that a mixture of comparable quality and reliability can be produced with lime and/or fly ash which do not meet the above criteria.

FLY ASH

Fly ash is "the finely divided residue that results from the combustion of ground or powdered coal and is transported from the boilers by flue gases" (ASTM Specification C 593). Fly ash is collected from the flue gases by either mechanical or electrostatic precipitation devices.

Large quantities of fly ash are produced in the U.S. Past and future fly ash production trends, as presented by Brackett (*8*), are shown in Figure 1.

Fly ash is a pozzolan and is defined as "a siliceous or siliceous and aluminous material, which in itself possesses little or no cementitious value but which will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties" (*7*).

Fly ash is available in different conditions. "Dry" fly ash is taken directly from the precipitator or from dry storage. If the fly ash is stockpiled in the open atmosphere, it is normally conditioned by adding water to prevent dusting. Some conditioned stockpiled fly ashes may develop cementitious materials and "set up" in the stockpile. If the fly ash has set up, crushing and screening may be required prior to use in stabilized mixtures. In some instances the collected fly ash is slurried into storage pond areas and must subsequently be reclaimed from the pond for use.

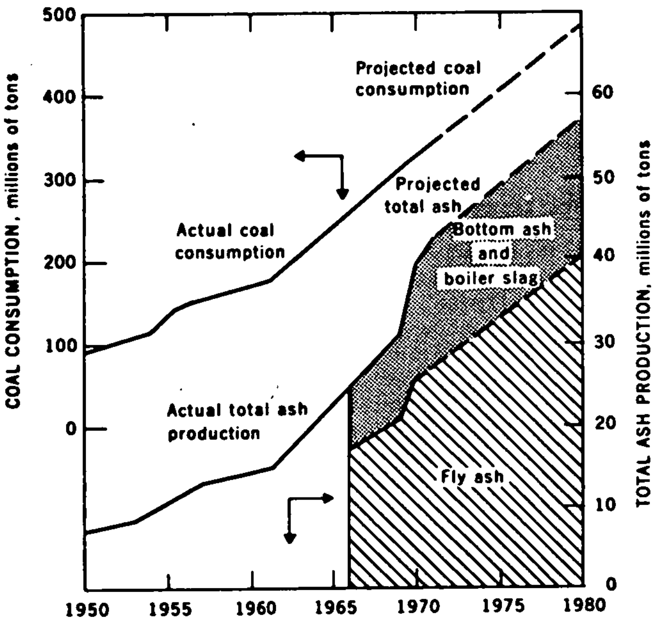


Figure 1. Coal consumption and ash production (8).

Fly ash particles are predominantly spherical, solid or hollow in nature, and amorphous. British studies (*9*) of seven different fly ashes indicate that the glass content (amorphous material) ranges from 71 to 88 percent. Minnick (*10*) indicated that the amorphous components of fly ash are the main components involved in the lime-fly ash pozzolanic reactions.

Fly ash particles are primarily composed of silica and alumina. Secondary ingredients are carbon and oxides of iron, calcium, magnesium, and sulfur. Oxides of sodium, potassium, titanium, manganese, and phosphorus may also be present. Table 2 gives typical ranges of values for the chemical composition of fly ash. Manz's data (*11*) for several United States lignite fly ashes indicate that substantial quantities of calcium oxide (20 to 40 percent) and magnesium oxide (5 to 10 percent) are typically present. Fly ashes produced in plants where the SO_2 emission control processes use lime or limestone may also contain significant amounts of calcium or magnesium oxides and other products. It may also contain selenium.

TABLE 2

TYPICAL COMPOSITION OF FLY ASH

Principal Constituents	Percent
SiO_2	28-52
Al_2O_3	15-34
Fe_2O_3	3-26
CaO	1-10
MgO	0- 2
SO_3	0- 4
Loss on Ignition	1-30

Brackett (*12*) reports that most fly ashes contain more than 85 percent of alumina, silica, iron oxide, and magnesia with the percentage of any one constituent varying over a wide range of values, depending upon the character of the particular ash being characterized. The broad ranges of fly ash composition are also evident from the studies of Minnick (*13*), Vincent et al. (*14*), and Watt and Thorne (*9*).

According to Brackett (*12*), the size of fly ash particles varies from 1 to 80 microns (3.94×10^{-5} to 3.15×10^{-3} in.) The carbon particles tend to be concentrated in the larger sizes (*12, 13*). Specific surface values for fly ash are quite variable but are generally in the range of 2 000 to 8 000 cm^2/g (8,800 to 35,000 $in.^2/oz$).

Fly ash is usually light gray in color, but can vary from light tan through shades of gray to black. The tan color is usually associated with the presence of iron oxide; darker colors are indicative of carbon or, in some cases, magnetic iron oxide or magnetite (*12*).

Fly Ash Specifications for Lime-Fly Ash–Aggregate Mixtures

Illinois Department of Transportation

The pozzolan, prior to dampening thereof to alleviate the dust problem, shall comply with the requirements of ASTM C 593. The maximum loss on ignition, determined in accordance with the procedures of ASTM C 311 shall be 10 percent. The tests prescribed by this specification shall be performed at the option of the engineer. At the time of mixing the pozzolan shall be, when dry sieved, in a finely divided condition, as follows:

SIEVE	MINIMUM PERCENT PASSING
½ inch (12.5 mm)	100
⅜ inch (9.5 mm)	95
No. 10 (2.00 mm)	75

The moisture content of dampened pozzolan shall not exceed 35 percent.

Pennsylvania Department of Transportation

Pozzolan shall meet the requirements of ASTM Designation C 593 and shall be capable of producing a mixture meeting Pennsylvania aggregate-lime-pozzolan mixture requirements.

Ohio Department of Transportation

Fly ash shall meet the requirements of ASTM C 593, with the exception of Section 7 for plastic mixes. The maximum loss on ignition shall be 10 percent as determined in accordance with ASTM C 311.

Federal Aviation Administration

The fly ash shall meet ASTM Specification C 593, Section 3.2, when sampled and tested in accordance with Sections 4, 6, and 8. An exception is noted that the water-soluble fraction shall not be determined. The above requirements may be waived if it is demonstrated that a mix of comparable quality and reliability can be produced with lime and/or fly ash which do not meet the above criteria.

AGGREGATES AND SOILS

The quality of a lime-fly ash-stabilized product depends to a large extent on the material being stabilized. High-clay-content fine-grained soils are less desirable for stabilization with lime-fly ash than are silts and more granular sands, gravels, and crushed stones.

A wide range of aggregate types and gradations have been used successfully including sands, gravels, crushed stones, and several types of slag. Aggregates should be of such gradation that, when mixed with lime, fly ash, and water, the resulting mixture is mechanically stable under compaction equipment and capable of being compacted in the field to a high density. The aggregate should be free from deleterious organic or chemical substances that may interfere with the desired chemical reaction and should consist of hard, durable particles, free from soft or disintegrated pieces.

Aggregate mixtures with greater fines contents have generally produced materials of greater durability than coarser grained mixtures. However, mixtures with coarser aggregate gradations are generally more mechanically stable.

Although laboratory studies have demonstrated the feasibility of stabilizing fine-grained soils (*4, 6*), lime-fly ash is not commonly used to stabilize such materials. Such considerations as (*a*) the difficulty of incorporating lime and fly ash with these materials under field conditions, (*b*) the low-level strength development (as compared to stabilized granular materials), (*c*) the increased lime-plus-fly ash content requirements, and (*d*) the use of alternate stabilizing agents, have hindered the widespread use of lime-fly ash stabilization of fine-grained soils.

Lime-fly ash stabilization specifications vary substantially regarding the properties of the materials to be stabilized. ASTM C 593, which is widely used, does not consider aggregate or material properties [gradation, plasticity index (PI), soundness, etc.] at all, but rather specifies mixture quality as evaluated by cured strength and durability.

Specifications used by some agencies consider aggregate gradation and quality. The specifications currently used by Illinois, Pennsylvania, and Ohio are summarized in Table 3. Specifications used by the Federal Aviation Administration can be found in the next section.

FAA Aggregate Gradation Requirements

A wide range of aggregate gradations are permitted with these base materials provided appropriate mix design procedures are followed. If the maximum particle size in the aggregate exceeds 0.75 in. (19.0 mm), the aggregate shall meet the gradation requirements given in Table 4 when tested in accordance with AASHTO T 11 and T 27.

The gradation in the table sets limits which shall determine the general suitability of the aggregate from a source of supply. The final gradations selected for use shall be within the limits designated in the table, and shall also be well graded from fine to coarse and shall not vary from high to low limits on subsequent sieves.

In addition to the gradations given in the table, clean sands and sand-sized materials such as boiler slags can be used. Also, if the aggregate has a substantial portion (75 percent) passing the No. 4 (4.75-mm) mesh sieve, the gradations in the table can be waived and the aggregate gradation adjusted with the fly ash and fines contents to produce the maximum dry density in the compacted mixture.

LIME-FLY ASH REACTIONS

The reactions that occur in the lime-fly ash–water system to form cementitious materials are complex. However, several studies provide basic information pertaining to the reactions.

Minnick (*10*) presents an illustrative list of reactions, as follows, and acknowledges that other reactions are also possible.

1. $RO \xrightarrow{H_2O} R(OH)_2$
2. $RO \xrightarrow{H_2O;\ CO_2} RCO_3 + H_2O$
3. $R(OH)_2 \xrightarrow{CO_2} RCO_3 + H_2O$
4. $R(OH)_2 + SiO_2 \xrightarrow{H_2O} xRO \cdot ySiO_2 \cdot zH_2O$
5. $R(OH)_2 + Al_2O_3 \xrightarrow{H_2O} xRO \cdot yAl_2O_3 \cdot zH_2O$
6. $R(OH)_2 + Al_2O_3 + SiO_2 \xrightarrow{H_2O} xRO \cdot yAl_2O_3 \cdot zSiO_2 \cdot wH_2O$
7. $R(OH)_2 + SO_3^{--} + Al_2O_3 \xrightarrow{H_2O}$ $xRO \cdot yAl_2O_3 \cdot zRSO_4 \cdot wH_2O$

Note: $R = Ca^{++}$ or Mg^{++} or combinations of these ions.

Based on his own studies as well as others documented in the literature, Minnick (*10*) indicates that the major cementing compounds formed in lime-fly ash mixtures are probably calcium silicate hydrates and possibly ettringite. Low-sulfate sulfoaluminate may also be formed.

The amorphous glassy materials in fly ash are the constituents that react to form complex silicates and aluminates (*10, 15, 16, 17*). The strength developed as a result of the lime-fly ash reaction is dependent on the quantity of cementing materials produced (*18*). Mullite may also be an important reactant. For a given lime-fly ash composition, increased quantities of pozzolanic reaction products are produced by extending the curing time and/or increasing the curing temperature. The effects of curing time and temperature on the strength development of a typical lime-fly ash–aggregate mixture are shown in Figure 2.

The pozzolanic reactivities of fly ashes are quite variable. Several studies have considered the relation between fly ash properties and pozzolanic reactivity (*14, 15, 19*). The following factors are indicative of good pozzolanic reactivity:

1. Increased percentage of fly ash passing the No. 325 (45-µm) sieve (*14*) or increased surface area (*15, 19*).
2. Increased SiO_2 (*14, 15*), $SiO_2 + R_2O_3$ (*14*), and $SiO_2 + Al_2O_3$ (*15*) contents.
3. Low carbon content (*14*) or loss on ignition (*19*).
4. Increased alkali contents (*19*).

Minnick et al. (*19*) emphasize that "no single test on fly ash will predict the performance of that material in compositions in which it is used," but that "it is far more preferable to combine factors or develop multiple factors in making performance predictions."

In addition to the primary reaction between the lime and the fly ash, the lime may also react with the "fines" in the material being stabilized. Soil-lime reactions that may occur are cation exchange, flocculation/agglomeration, and a soil-lime pozzolanic reaction.

Cation exchange and flocculation/agglomeration reactions take place quite rapidly and cause decreased plasticity of the fines and some "immediate" strengthening. The plasticity reduction improves workability and allows easier mixing with materials that contain substantial quantities of plastic fines.

Reaction products from the soil-lime pozzolanic reaction contribute to the development of the cementitious matrix in the stabilized mixture. Similar secondary soil-lime reactions have been noted for soil-cement mixtures containing "lime-reactive" fines.

TABLE 3
AGGREGATE SPECIFICATIONS FOR LIME-FLY ASH MIXTURES

a) Gradation

Sieve	% Passing			
	Illinois	Pennsylvania		Ohio
2" (50.0 mm)	—	100	100	100
1 1/2" (38.1 mm)	100	—	—	—
1" (25.0 mm)	90-100	—	—	75-100
3/4" (19.0 mm)	—	52-100	70-100	—
1/2" (12.5 mm)	60-100	—	—	50-85
3/8" (9.5 mm)	—	36-70	58-100	—
No. 4 (4.75 mm)	40-70	24-50	45-80	35-60
No. 8 (2.36 mm)	—	—	—	15-45
No. 16 (1.18 mm)	—	10-30	25-50	10-35
No. 40 (425 µm)	0-25	—	—	—
No. 50 (300 µm)	—	—	—	3-18
No. 100 (150 µm)	—	—	6-20	—
No. 200 (75 µm)	0-10 (Gravel) 0-15 (Crushed Stone & Slag)	0-10	—	1-7

b) Other Typical Requirements

Property	Illinois	Penn	Ohio	FAA
Sodium Sulfate Soundness (AASHTO - T104)	<25%	< 20%	<15%	<12%
Los Angeles Abrasion (AASHTO - T96)	<45%	< 55%	—	—
Plasticity Index	< 9	< 6	—	< 6
Liquid Limit	—	< 25	—	<25

TABLE 4

REQUIREMENTS FOR GRADATION OF AGGREGATE FOR THE PLANT-MIX BASE COURSE

Sieve designation (square openings)	Percentage by weight passing sieves		
	A	B	C
2" (50.0 mm)	100	-	-
1-1/2" (38.1 mm)	-	100	-
1" (25.0 mm)	55 - 85	70 - 95	100
3/4" (19.0 mm)	50 - 80	55 - 85	70 - 100
No. 4 (4.75 mm)	40 - 60	40 - 60	40 - 65
No. 40 (425 μm)	10 - 30	10 - 30	15 - 30
No. 200 (75 μm)	5 - 15	5 - 15	5 - 15

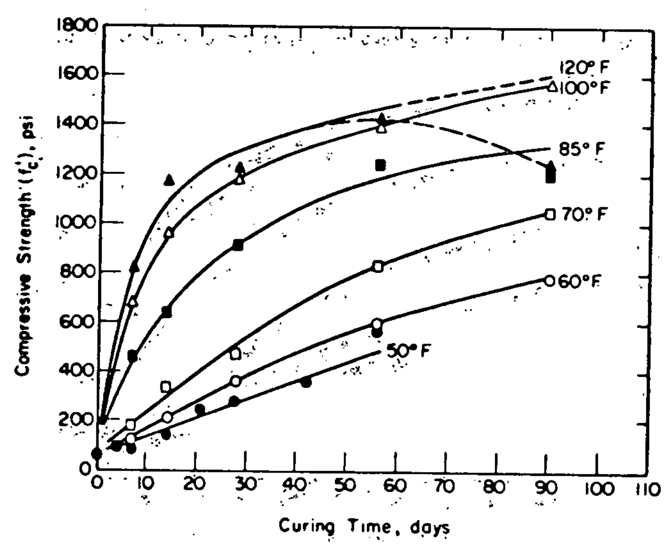


Figure 2. Effects of curing temperature and curing time on the compressive strength development of an LFA mixture (22).

CHAPTER THREE

MIXTURE AND MIXTURE PROPERTIES

RANGE OF COMPOSITION

The composition of a lime-fly ash–aggregate (LFA) mixture can be defined by designating the total amount of lime plus fly ash and the lime-to-fly ash ratio. Lime and fly ash contents are generally designated as a percent by dry weight of the total mixture (i.e., 4 percent lime, 16 percent fly ash, 80 percent aggregate).

Lime-plus-fly ash contents depend on many variables, but generally range from 12 to 30 percent. Fine-grained soils generally require higher percentages of lime plus fly ash and requirements for well-graded aggregates generally fall at the lower end of the range. Aggregates of angular shape and rough surface texture require larger quantities of lime plus fly ash than rounded and smooth aggregate particles.

Lime-to-fly ash ratios vary substantially, but the range is generally from 1:10 to 1:2 with ratios of 1:3 to 1:4 being common. Factors that tend to increase the lime requirement are (*a*) greater fines content [minus No. 200 (—75 μm)], (*b*) increased PI, and (*c*) increased pozzolanic reactivity of the fly ash.

Lime-fly ash mixture proportions can not be established based on an analysis of the properties of the lime, fly ash, and material to be stabilized. Proportions are determined using laboratory-based mixture design procedures (see Chapter Five) using the component materials to be incorporated into the field mixture.

ENGINEERING PROPERTIES OF LIME-FLY ASH-STABILIZED MATERIALS

Many properties must be considered in lime-fly ash mixture proportion selection and pavement structural analysis. LFA mixture properties vary depending on lime and fly ash characteristics, mixture proportions, stabilized material, density, and curing conditions. Many of the properties vary for a given mixture depending on curing conditions. Thus, it is necessary to define mixture curing conditions (time, temperature, moisture) when reporting mixture property data.

TABLE 5

RANGES OF COMPRESSIVE STRENGTH FOR THE LIME-FLY ASH-STABILIZED MATERIALS

Material	28 Day Immersed Compressive Strength psi	(kPa)
Gravels	400-1300	(2800-9000)
Sands	300- 700	(2100-4800)
Silts	300- 700	(2100-4800)
Clays	200- 500	(1400-3400)
Crushed Stones and Slag	1400-2000	(10,000-14,000)

Compressive Strength

Unconfined compressive strength is frequently used to evaluate the quality of cured LFA mixtures. A general range of typical strengths for various LFA mixtures is given in Table 5. Barenberg (*22*) states that standard ASTM C 593 curing [7 days at 100 F (38 C)] develops mixture compressive strengths ranging from about 500 to 1200 psi (3400 to 8300 kPa). ASTM Procedure C 593-69 requires a minimum compressive strength of 400 psi (2800 kPa) for lime-fly ash used in paving-type mixtures.

Compressive strength development continues in LFA mixtures for a substantial period following placement. Figure 3 shows core strength data for a typical LFA mixture constructed in Chicago (*23*).

Shear Strength

The shear strength of LFA mixtures has not been extensively considered. Unconfined compressive strength data for typical mixtures indicate that shear strength failures are not likely for normal pavement applications.

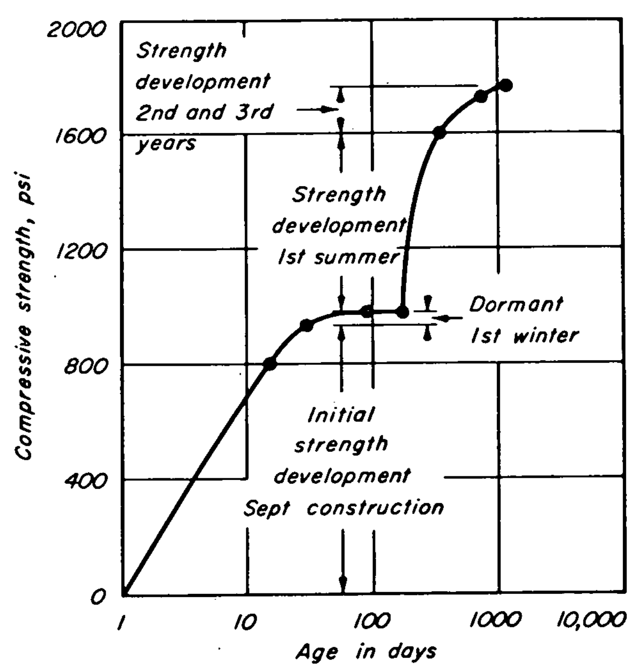


Figure 3. Compressive strength development of lime-fly ash-stabilized mixture in Chicago area (23).

Typical shear strength data (*24*) for lime-fly ash–gravel mixtures (lime-plus-fly ash contents of 13.5 to 18 percent, lime-to-fly ash ratios of 1:2.5 to 1:3.5) indicate that angles of shearing resistance varied from 49° to 53° and cohesions ranged from 55 to 128 psi (380 to 880 kPa). Mixture curing was equivalent to 28-day moist-sand curing at approximately 70 to 75 F (21 to 24 C).

Flexural Strength

The flexural strength of LFA mixtures is substantially lower than the corresponding compressive strengths. Similar trends have been noted for other cementitious-stabilized materials such as soil-cement or soil-lime mixtures.

Laboratory test procedures similar to ASTM D 1635-63 can be used to evaluate the flexural strength of LFA mixtures. Figure 4 illustrates typical flexural strength-cure time relations for two mixtures.

The ratio of flexural strength to compressive strength for most LFA mixtures is between 0.18 and 0.25. A value of 20 percent of the compressive strength is a conservative engineering estimate of the flexural strength of LFA mixtures (*22*).

Split-tensile and double-punch procedures have also been proposed for evaluating the tensile strength of stabilized materials (*25, 26*). The tensile strengths determined by these procedures are approximately one-half the flexural strengths. LFA mixture pavement response and performance studies indicate that the mixtures can sustain repeated flexural stresses that are greater than the split-tensile or double-punch strength of the mixtures. Thus, the double-punch and split-tensile strength data are of limited value in pavement analysis and design procedures.

Modulus of Elasticity

According to Ahlberg and Barenberg (*23*) the elastic moduli for LFA mixtures are different depending on whether the modulus is determined from compressive or flexural testing procedures. They indicate that the flexural modulus is somewhat lower than the compressive modulus.

Flexural moduli values are recommended for pavement design calculations (*23*). A moment-curvature plot for a typical LFA mixture is shown in Figure 5. Ahlberg and Barenberg (*23*) indicate that flexural moduli for granular LFA mixtures range from 1.5×10^6 to 2.5×10^6 psi (10×10^6 to 17×10^6 kPa) after a reasonable curing time. LFA mixtures containing greater fines contents generally have lower moduli values.

LFA mixture moduli values increase as mixture strength increases. Figure 6 illustrates the development of compressive strength, flexural strength, and flexural modulus for the LFA mixture used in an extensive University of Illinois test track study (*23*). Studies using "pulse velocity" evaluation procedures (*27*) for laboratory and field conditions also suggest increasing moduli values with the development of mixture strength. Figure 7 illustrates the relation between unconfined strength and pulse velocity for 14 different soils ranging from AASHTO classes of A-1-a to A-5.

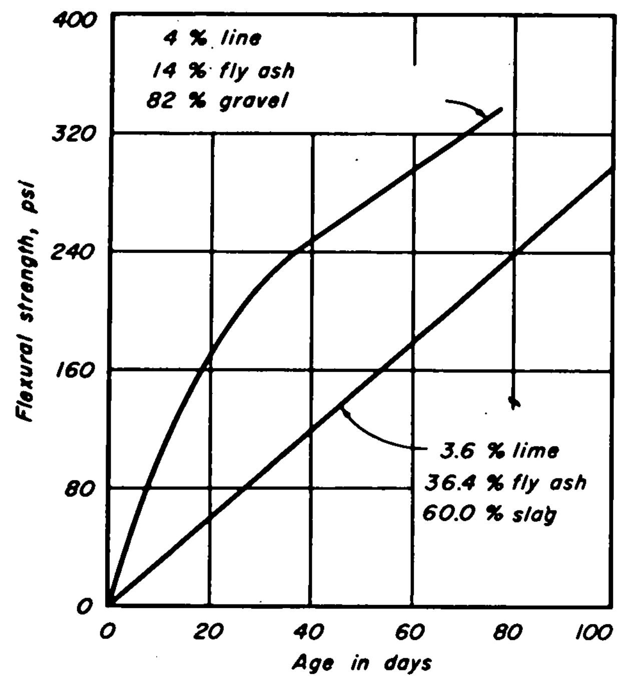


Figure 4. Flexural strength development of typical lime-fly ash-stabilized mixtures (laboratory curing) (23).

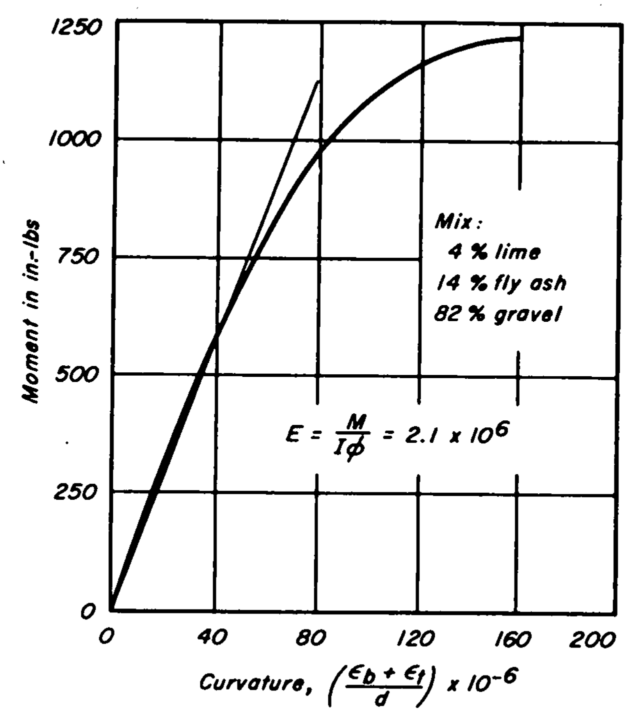


Figure 5. Moment-curvature relationship for lime-fly ash–aggregate mixture (23).

Poisson's Ratio

Poisson's ratio of a material usually varies with the intensity of the applied stress. However, this ratio usually remains relatively constant at stress levels below about 70 percent of the ultimate stress. Figure 8 is a plot of Poisson's ratio

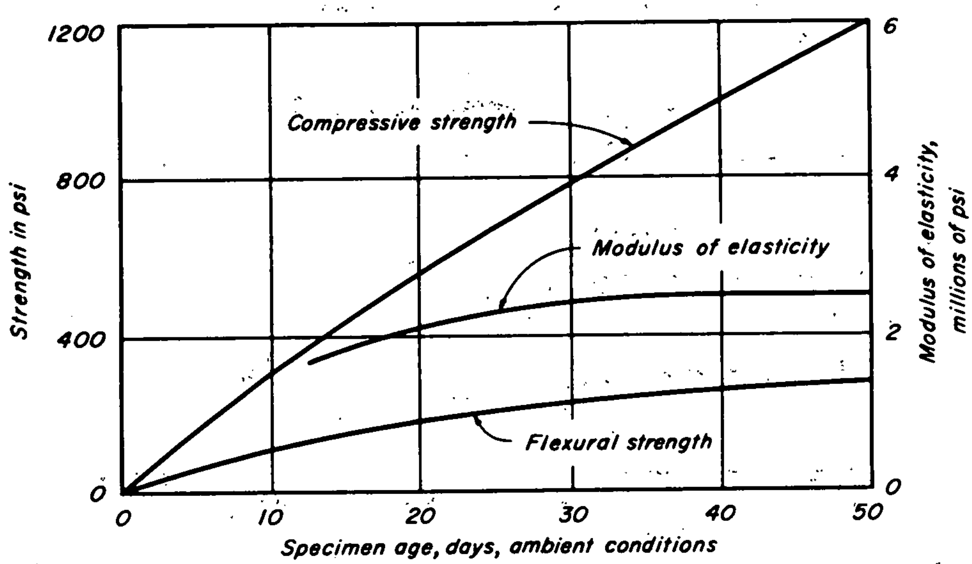


Figure 6. Effect of curing (approximately 73 F) on the engineering properties of a lime-fly ash–aggregate mixture (82 percent aggregate, 14 percent fly ash, 4 percent lime) (23).

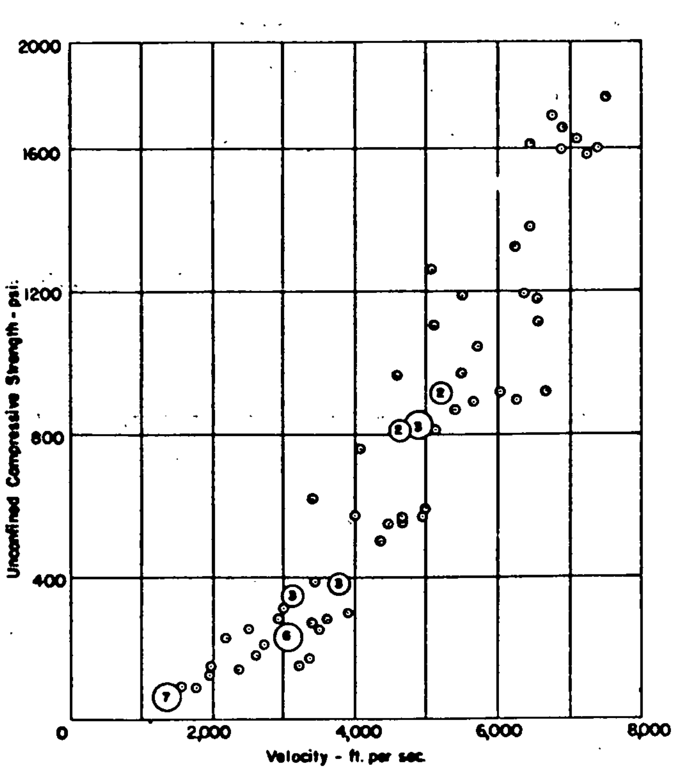


Figure 7. Relationship between cured compressive strength and velocity for lime-fly ash-stabilized soils (27).

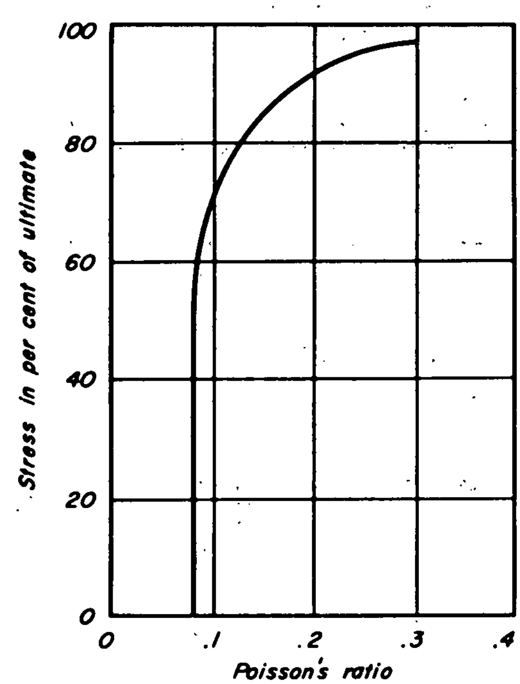


Figure 8. Effect of stress level on Poisson's ratio for a lime-fly ash–gravel mixture (23).

versus stress level for a typical LFA mixture (*23*). The value for Poisson's ratio remains relatively constant at about 0.08 for stress levels below approximately 60 percent of ultimate strength and then increases at high stress levels, reaching a value of approximately 0.3 at failure.

Barenberg (*22*) indicates that for most pavement design calculations, Poisson's ratio for cured LFA mixtures can be taken as between 0.10 and 0.15 without appreciable error.

Fatigue Properties

The fatigue properties of LFA mixtures are important in pavement design analysis. Because the compressive stresses developed in most pavements with LFA mixture layers are small compared to LFA mixture compressive strength, compressive fatigue behavior is not considered to be of any consequence. Flexural fatigue is of major importance, however.

The flexural fatigue properties of lime-fly ash–aggregate mixtures have been studied and reported by Ahlberg and McVinnie (*28*). Results from these tests are summarized in Figure 9. The results are presented as a relationship between the ratio of applied stress to the modulus of rupture of the material and the number of load applications to failure. All tests were conducted on beam specimens with loads applied continuously at a rate of approximately 450 applications per minute.

In analyzing the fatigue properties of LFA mixtures, the influence of the strength gain with time must be recognized. Because flexural strength increases with time, the stress level (expressed as a percent of the ultimate flexural strength) decreases. Thus, as the time required to accumulate the number of load applications to failure becomes longer, the number of load applications to failure becomes greater. If the gain in strength is sufficiently rapid or the stress level is small, the probability of the material failing in fatigue is minimized.

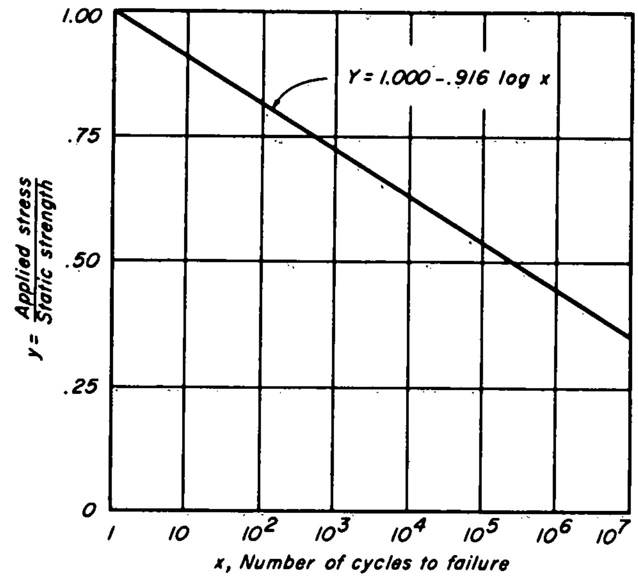


Figure 9. Flexural fatigue behavior of lime-fly ash–aggregate mixture (28).

Autogenous Healing

Autogenous healing refers to the phenomenon by which a crack in a material heals or re-cements itself by a self-generating mechanism. The continuing pozzolanic reaction between lime and fly ash in LFA mixtures provides the potential for autogenous healing.

Laboratory tests by Callahan et al. (*29*) proved that autogenous healing can take place to a significant extent in lime-fly ash–aggregate mixtures. Several cases in which this phenomenon has occurred in the field have been observed. The degree of healing is dependent on the age at which the fracture occurs, the degree of contact of the fractured surfaces, and the curing conditions. Although it can not be expected that healing will occur across wide cracks, autogenous healing provides the potential for improved durability and fatigue resistance in LFA mixtures.

Volume Changes

LFA mixtures that are properly proportioned, constructed, and cured will have a good resistance to "frost heave." Thus, the major volume changes in the LFA mixtures are induced by thermal and moisture changes. Temperature and moisture shrinkage may develop sufficient tensile stress to initiate cracking in the LFA mixture pavement layer.

Miller and Couturier (*30*) investigated the thermal expansion characteristics of lime-fly ash–aggregate mixtures. Their data indicate that the coefficients of thermal expansion for the LFA mixtures ranged from approximately 5 to $7 \times 10^{-6}/°F$ (9 to $13 \times 10^{-6}/°C$). Increased dry density and larger percentages of lime and fly ash tended to increase the coefficient of thermal expansion. Figures 10 and 11 illustrate the density and lime content effect. Barenberg reports similar values for the coefficient of thermal expansion for LFA mixtures (*31*).

There is no published information on moisture-related volume change in LFA mixtures. It is known from field experience that LFA mixtures do exhibit drying shrinkage tendencies.

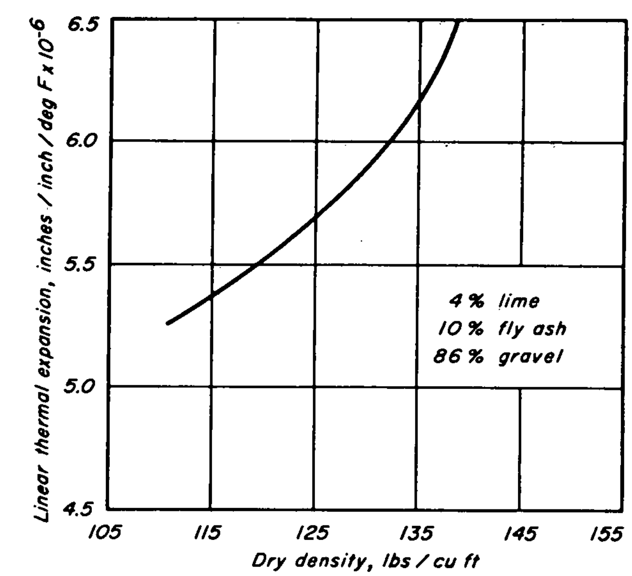


Figure 10. Effect of dry density on thermal expansion properties (30).

Durability

Cyclic freeze-thaw action is the major durability factor that must be considered for LFA mixtures. The extent of cyclic freeze-thaw action is dependent on the location of material in the pavement structure, geographic location, climatic variability, and pavement system characteristics (*32*).

Cyclic freeze-thaw and brushing tests (ASTM C 593) have been extensively used for evaluating LFA mixture durability. ASTM C 593 criteria require less than 14 percent weight loss following 12 freeze-thaw cycles.

The Iowa freeze-thaw test (*33*) has also been used to evaluate the freeze-thaw durability of LFA mixtures. The index of resistance, R_f, is used as a measure of durability.

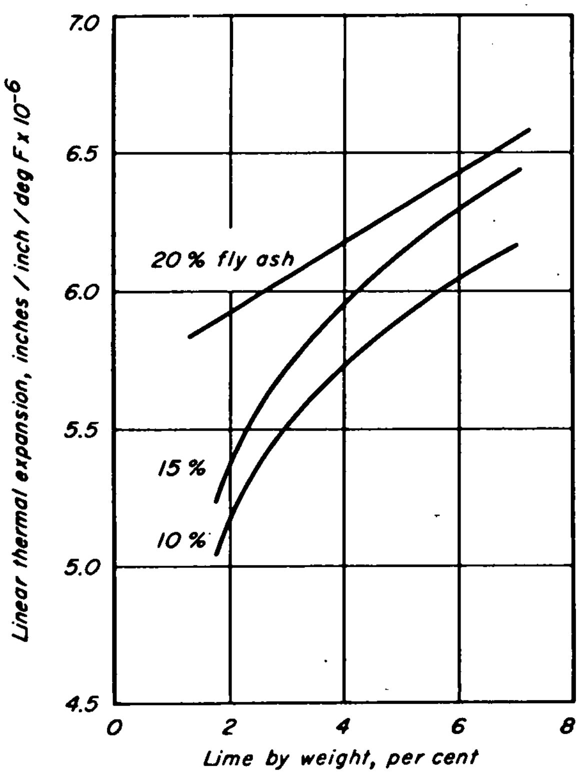


Figure 11. Effect of lime content on thermal expansion properties (30).

$$R_f = \frac{100\ P_f}{P_c} \tag{1}$$

in which

R_f = index of resistance to freeze-thaw;

P_f = unconfined compressive strength following 28 days moist curing at 71 F (22 C), 24 hours immersion in water, and 10 freeze-thaw cycles; and

P_c = unconfined compressive strength of "control specimens" following 28 days moist curing at 71 F (22 C) and 11 days immersion in water.

For Iowa climatic conditions, it has been proposed that R_f should be greater than 80 for stabilized soils (*34, 35*).

Recent stabilized materials durability studies at the University of Illinois (*36*) have resulted in the development of a freeze-thaw testing procedure that closely simulates field conditions. The standard "freeze-thaw" testing cycle developed for Illinois is shown in Figure 12. Typical compressive strength–freeze-thaw cycle response data for LFA mixtures are presented in Figure 13. The mixtures were cured 7 days at 100 F (38 C) in accordance with ASTM C 593 prior to freeze-thaw testing. It is apparent from Figure 13 that the CA-6, Plainfield sand, and CA-10 LFA mixtures possess a high degree of freeze-thaw resistance, and that Ridgeville sand and pit-run gravel mixtures are less durable.

Dempsey and Thompson (*36*) have developed general relations between the compressive strengths of stabilized materials (including LFA mixtures) subjected to 5 or 10 freeze-thaw cycles and the compressive strengths of the

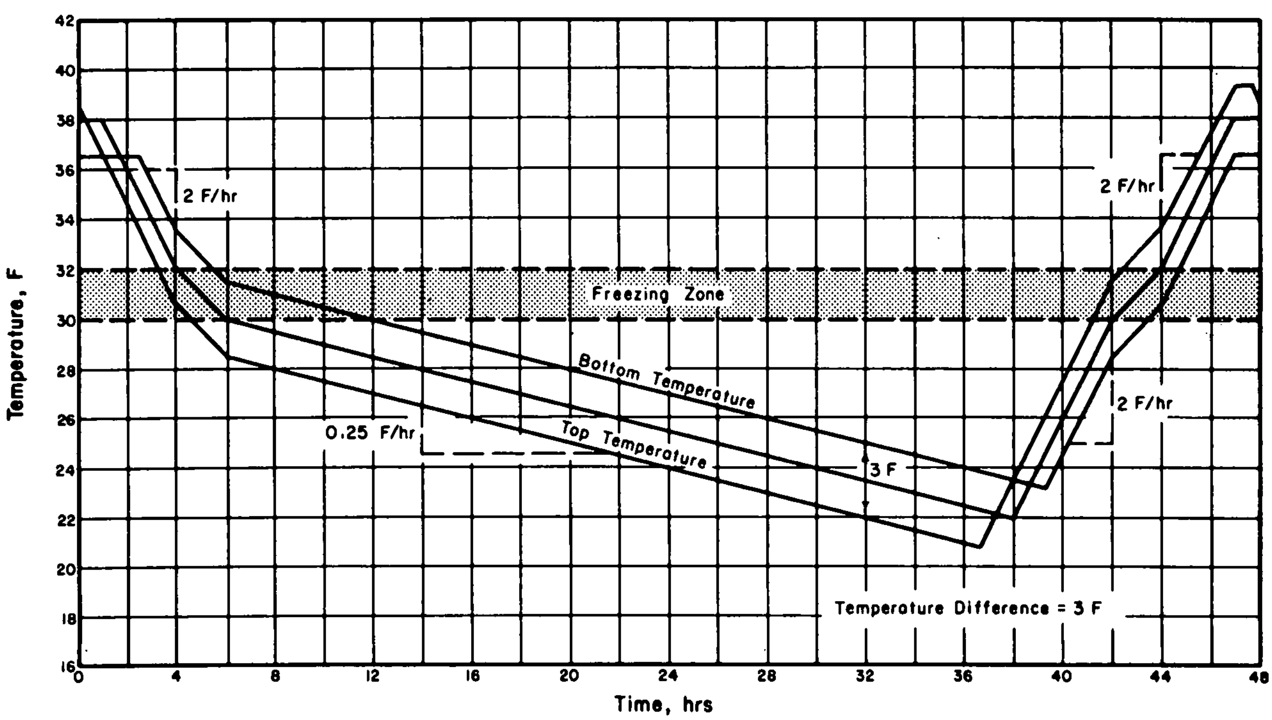


Figure 12. Standard freeze-thaw cycle for Illinois.

cured mixtures prior to freeze-thaw testing. The relationships for 5 and 10 freeze-thaw cycles are shown in Figures 14 and 15, respectively. It is apparent that freeze-thaw durability resistance is significantly related to cured strength.

In a follow-up study (*37*), Dempsey and Thompson demonstrated that the compressive strength of cured stabilized materials subject to vacuum saturation provides a better indication of the 5- or 10-cycle freeze-thaw compressive strength. The relationship between vacuum saturation and freeze-thaw strength is shown in Figures 16 and 17. ASTM Committee C7.07 has revised ASTM C 593 to incorporate the vacuum saturation testing procedure. The standard freeze-thaw brushing test was deleted from ASTM C 593.

In conjunction with the development of improved freeze-thaw evaluation procedures, Thompson and Dempsey extended the residual strength concept (*32*). Residual strength is the strength of a stabilized material following the equivalent of the first winter's freeze-thaw cycles. If the residual strength can ensure the desired level of structural pavement response and the material displays a projected strength-time history that ensures that the field strength will always be greater than some minimum strength requirement, then satisfactory pavement performance can be attained. The residual strength concept is illustrated in Figure 18. Field experience with LFA mixtures indicates that if the cured material possesses sufficient durability to survive the first winter's freeze-thaw cycles, the probability of experiencing durability problems during subsequent years is quite low. The additional curing developed during the summer following construction and during subsequent summers is beneficial in developing additional strength in the LFA mixture.

FACTORS INFLUENCING LFA PROPERTIES

There are many factors that significantly affect LFA mixture properties. Figure 19 illustrates that mixture properties are dependent on the interaction of many variables. The variables can be grouped into four categories: materials, proportions, processing, and curing.

Materials

Fly ash, lime, aggregate, and soil characteristics have a strong influence on the ultimate nature of LFA mixture properties. A general discussion of these materials is presented in Chapter Two. There are substantial property variations in all of the ingredients incorporated into an LFA mixture and there is also a wide range in mixture quality as evidenced by the typical compressive strength data given in Table 5.

Proportions

The total quantity of lime plus fly ash and the lime-to-fly ash ratio are the proportioning variables that can be altered in LFA mixture proportion selection. It is generally accepted that when coarse-textured materials are stabilized, the mixture properties are primarily controlled by the quality of the matrix material [lime + fly ash + fraction

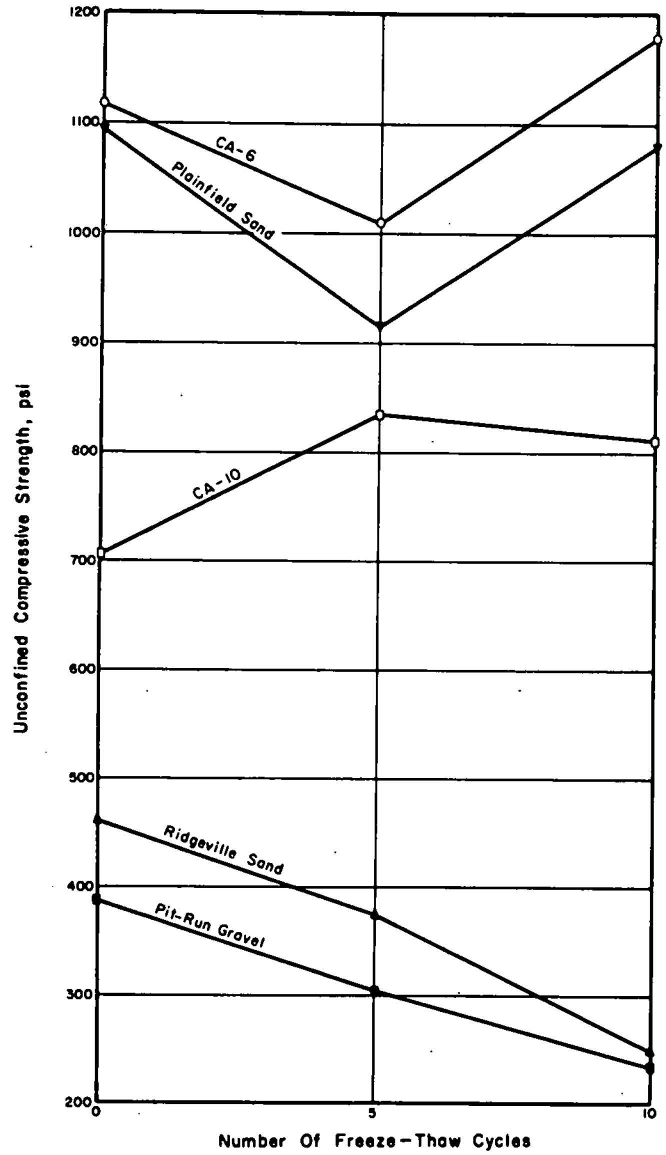


Figure 13. Effect of freeze-thaw on the compressive strength of lime-fly ash-stabilized mixtures.

passing No. 4 (4.75-mm) sieve]. Sufficient matrix material must be present in the mixture to "float" the coarse aggregate (plus No. 4) to ensure high strength and good durability. If insufficient matrix material is present in the mixture, adequate compacted density is not achieved in the matrix material, even though the over-all LFA mixture density is high.

The effects of the lime-plus-fly ash content on the strength and durability of an A-3 sand and a pit-run gravel, A-1-b(0), are shown in Figures 20 and 21, respectively (*36*). Similar data are shown in Figure 22 for a well-graded crushed stone (*36*).

Additional data (*6*) for three fine-grained soils and a

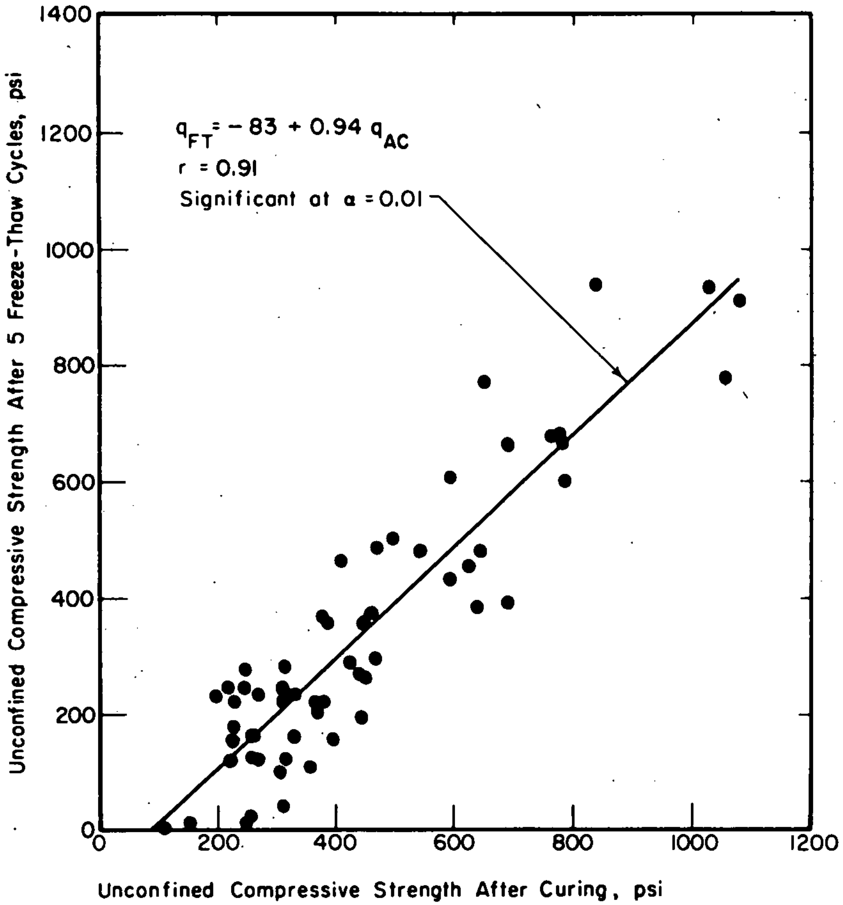


Figure 14. Relationship between after-curing strength and 5-cycle freeze-thaw strength (all data adjusted to equivalent l/d = 2*).*

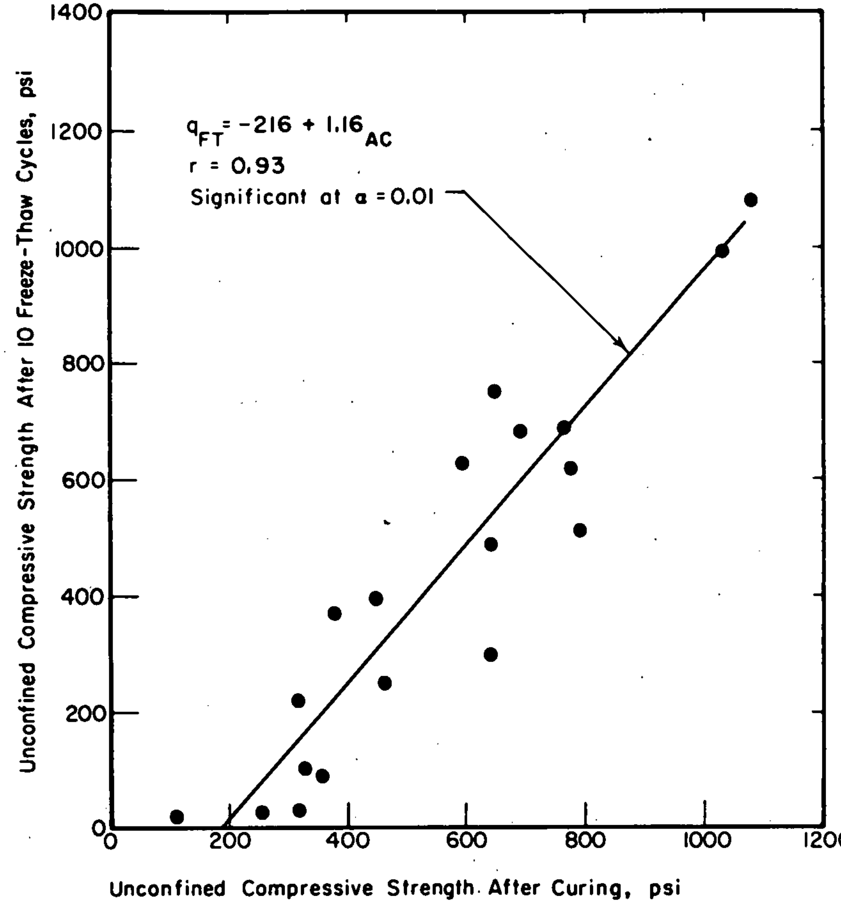


Figure 15. Relationship between curing strength and 10-cycle freeze-thaw strength (all data adjusted to equivalent l/d = 2*).*

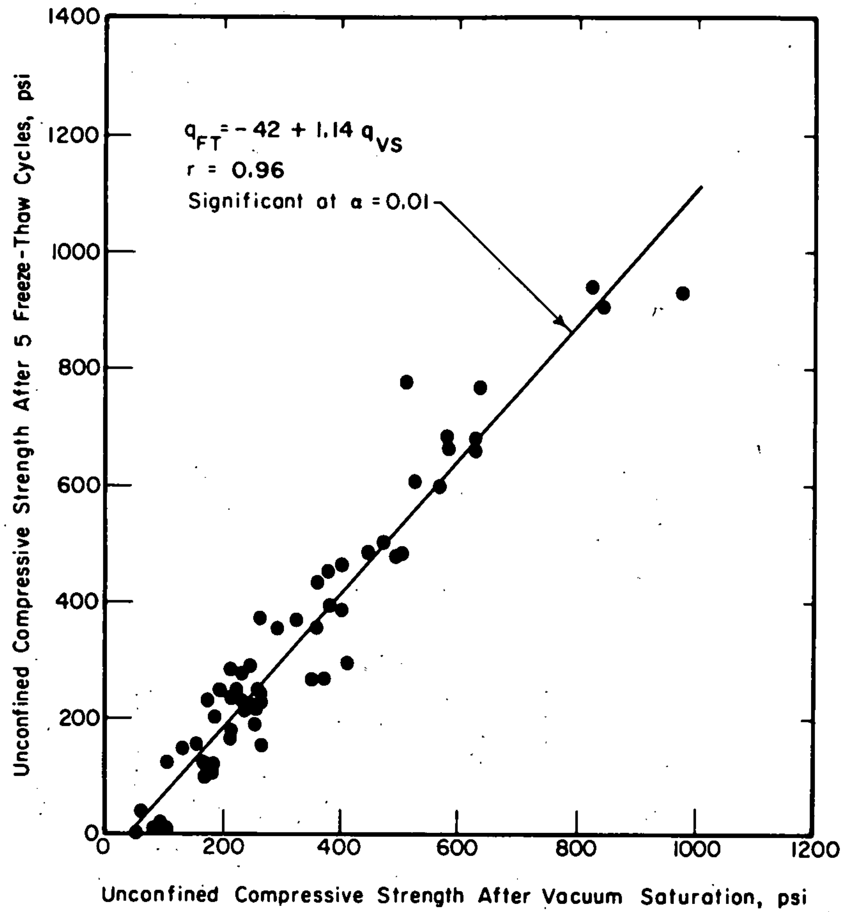


Figure 16. Relationship between vacuum saturation strength and 5-cycle freeze-thaw strength (all data adjusted to equivalent l/d = 2*).*

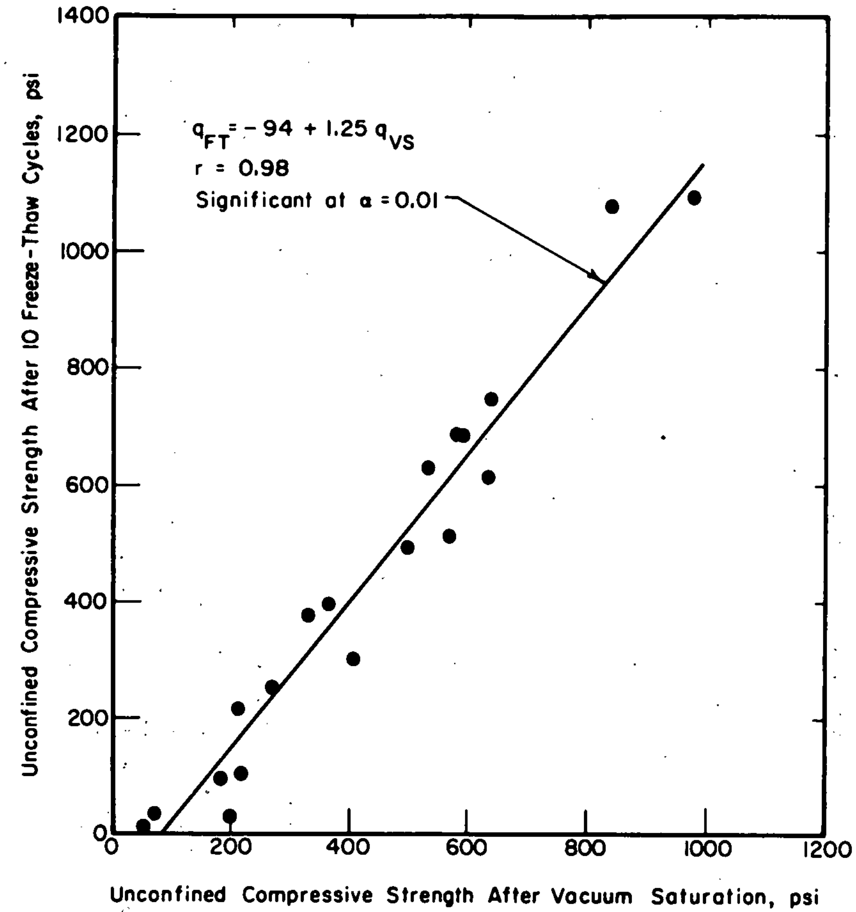


Figure 17. Relationship between vacuum saturation strength and 10-cycle freeze-thaw strength (all data adjusted to equivalent l/d = 2*).*

sand are shown in Figure 23. It is apparent that increased lime-plus-fly ash content effects increased cured strength. Normal proportioning operations are directed toward achieving a satisfactory quality LFA mixture at a minimum cost. In most situations, lime and fly ash are more expensive than the aggregate or soil being stabilized. Lime-plus-fly ash contents are established at the lowest level consistent with achieving satisfactory mixture quality.

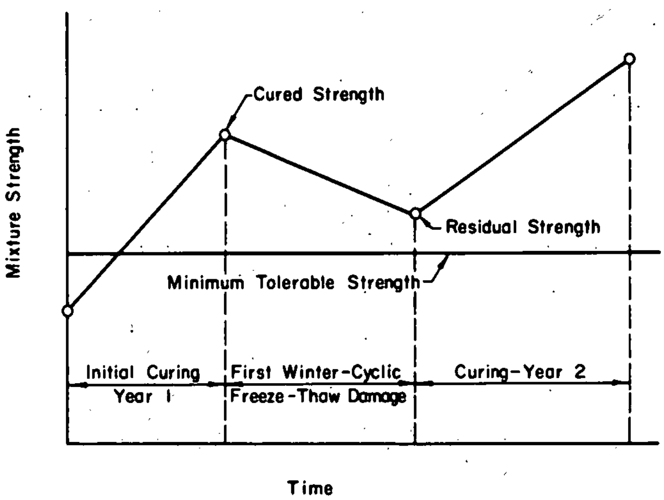


Figure 18. The residual strength concept of freeze-thaw durability.

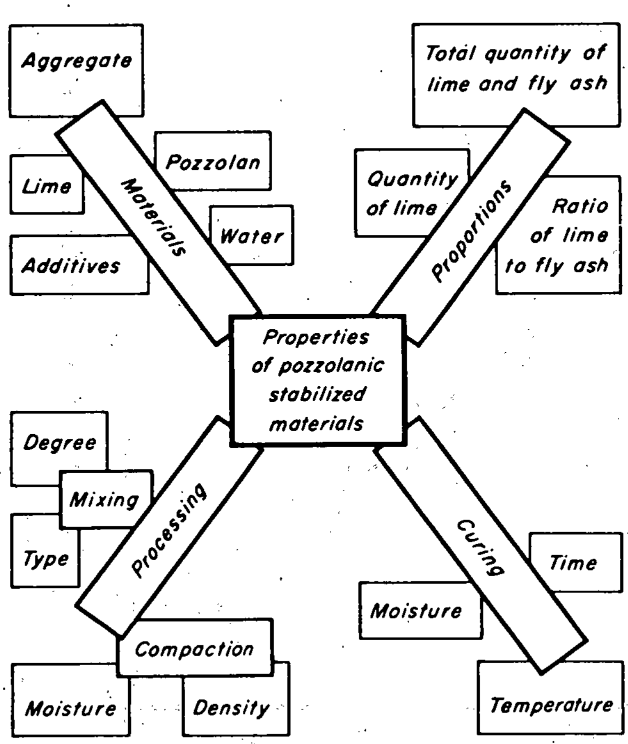


Figure 19. Factors influencing the properties of lime-fly ash-stabilized materials (33).

The effects of the lime-to-fly ash ratio (*6, 24*) on compressive strength development of typical LFA mixtures are shown in Figures 24 and 25. There is generally an appropriate lime-to-fly ash ratio that produces adequate strength development in a given mixture. Even though the fly ash source remains the same, optimum lime-to-fly ash ratios for a mixture vary depending on the characteristics of the material being stabilized. Factors that normally increase the lime requirement are increased silt and clay contents, and increased plasticity. The additional lime is required to satisfy the lime demand of the greater quantity of plastic fines.

In general, lime-to-fly ash ratios are set at the lowest possible level consistent with maintaining LFA mixture quality. Lime is the most expensive constituent in an LFA mixture.

Processing

Mixing

Maximum effectiveness of lime-fly ash stabilization is achieved when all of the materials are completely mixed. With fine-grained materials, adequate pulverization is needed to minimize the occurrence of clay balls in the mixture.

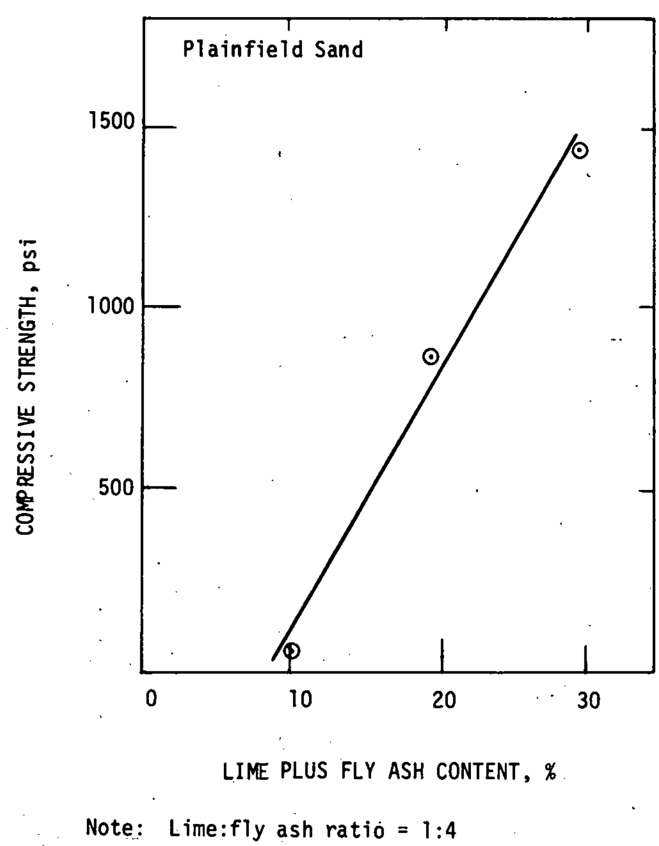


Figure 20. Lime-plus-fly ash-content effects on compressive strength development (Plainfield sand) (7-day curing at 100 F) (36).

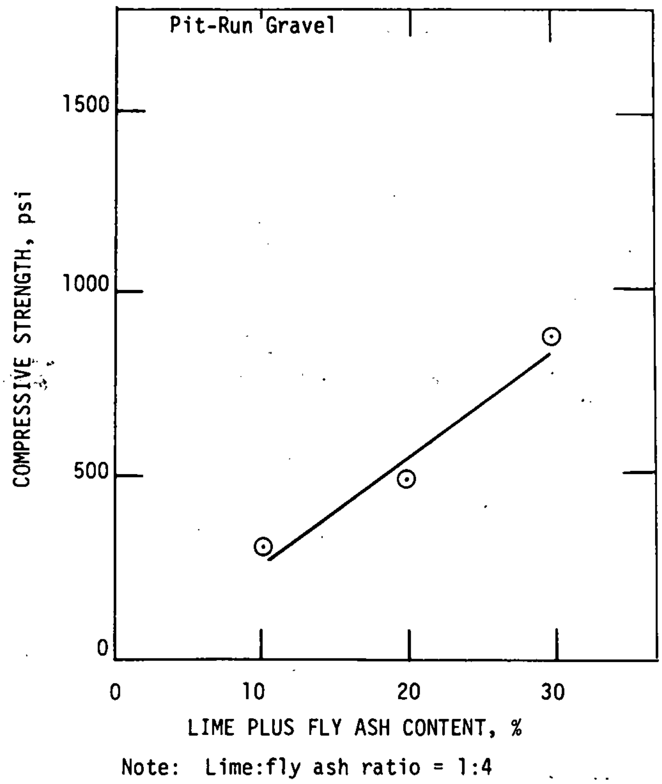


Figure 21. Lime-plus-fly ash-content effects on compressive strength development (pit-run gravel) (7-day curing at 100 F) (36).

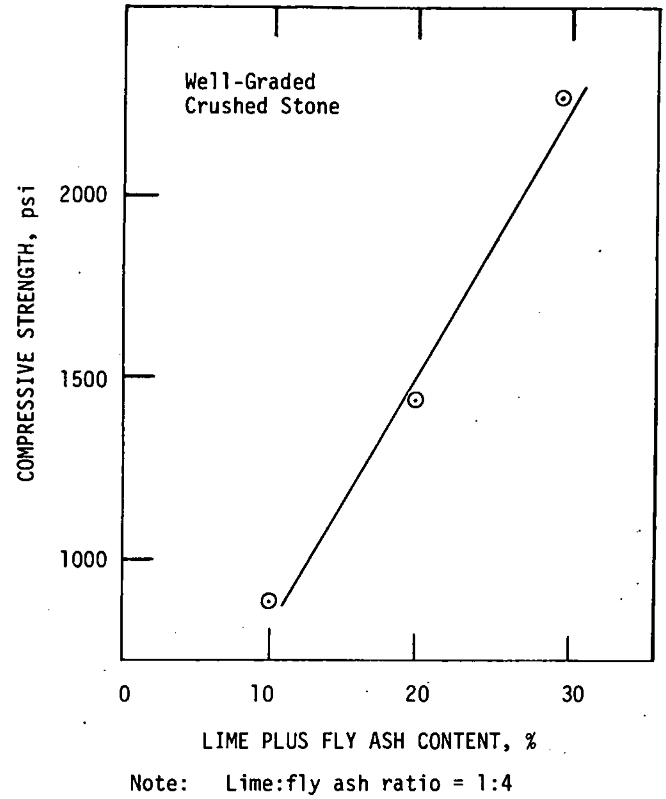


Figure 22. Lime-plus-fly ash-content effects on compressive strength development (well-graded crushed stone) (7-day curing at 100 F) (36).

The effects of mixing time on strength development of laboratory mixtures have been considered (*24*). Generally, after a minimal mixing time for achieving an intimate mixture, additional mixing does not produce significant benefits as indicated in Figure 26. More thorough and uniform mixing can be achieved in plant-mix operations than with mixed-in-place procedures.

An extensive study of plant-mixed LFA mixture operation in the Chicago area (*20*) indicates that the coefficients of variation for cured compressive strength (laboratory-prepared specimens of plant-mixed material) range from 7.7 to 18.2 percent with an average of approximately 11.5 percent. According to Thompson and Dempsey (*32*), a reasonable range of coefficient of variation for field-mixed stabilized materials is 20 to 25 percent.

Density

The compacted density of a given LFA mixture significantly influences the strength and durability of the cured mixture. Increased density improves strength and durability. The effect of density on the strength and durability of typical LFA mixtures is illustrated in Figures 27 and 28.

It is apparent that careful consideration must be directed to density in any construction quality control program. Small changes in compacted density effect substantial changes in strength and durability. Similar trends have been noted for other forms of cementitious stabilizers (soil-cement, soil-lime).

Curing

Proper conditions (moisture, temperature, time) are essential for curing LFA mixtures. Without proper curing, adequate mixture designs may not perform satisfactorily in the field.

Maintaining adequate moisture in the mixture is essential for proper curing. The lime-fly ash pozzolanic reaction requires water. If the compaction moisture content (generally around optimum) is approximately maintained in the LFA mixture during curing, sufficient water is available for the pozzolanic reaction.

The lime-fly ash pozzolanic reaction is time- and temperature-dependent. Figure 29 illustrates the effect of various curing temperatures and times on compressive strength development for a typical LFA mixture (*20*). The effect of temperature on the lime-fly ash pozzolanic reaction is not linear. Curing temperatures in excess of approximately 80 F (27 C) accelerate the reaction to a greater extent than do lower temperatures (*20, 26*). Strength development is sub-

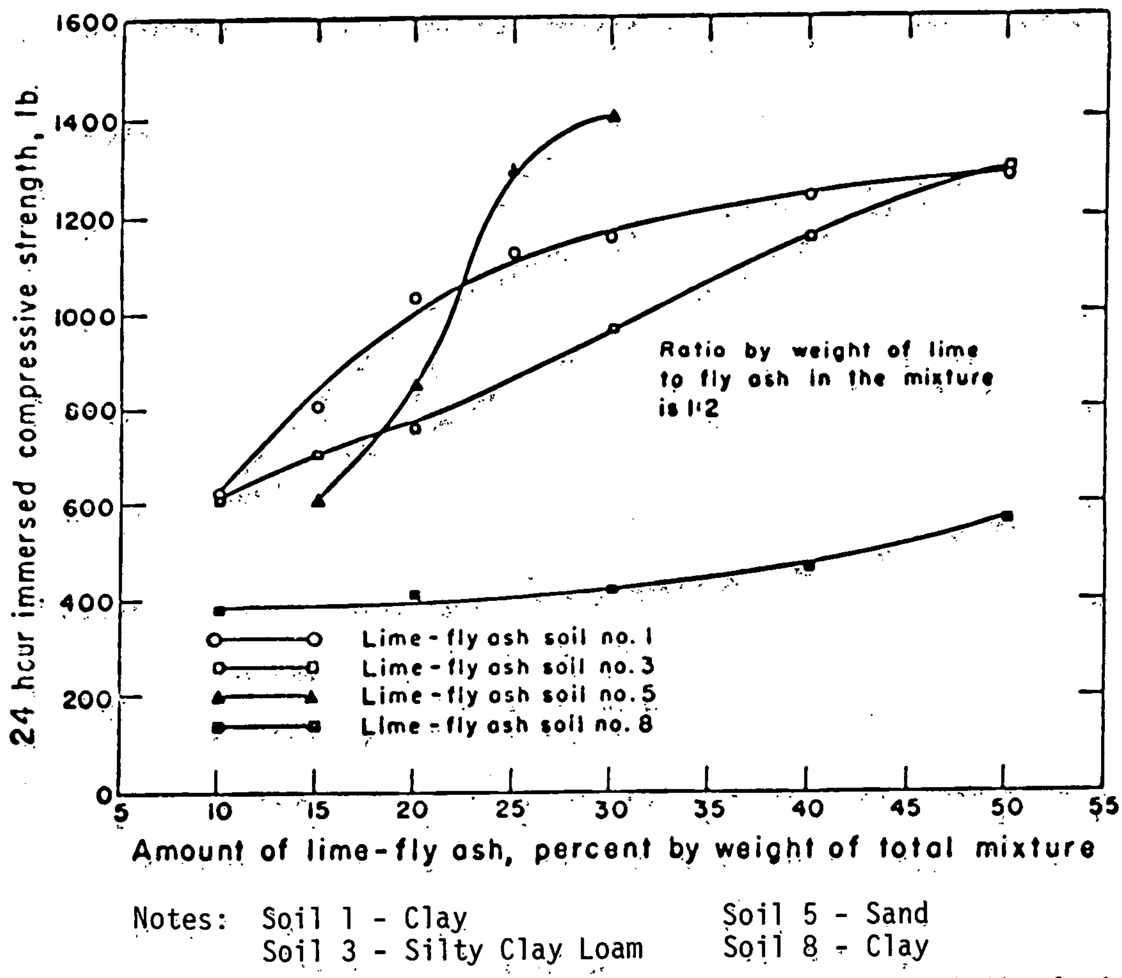


Figure 23. Effect of variations in the amount of lime-fly ash on the 28-day compressive strength of lime-fly ash-stabilized soils (6).

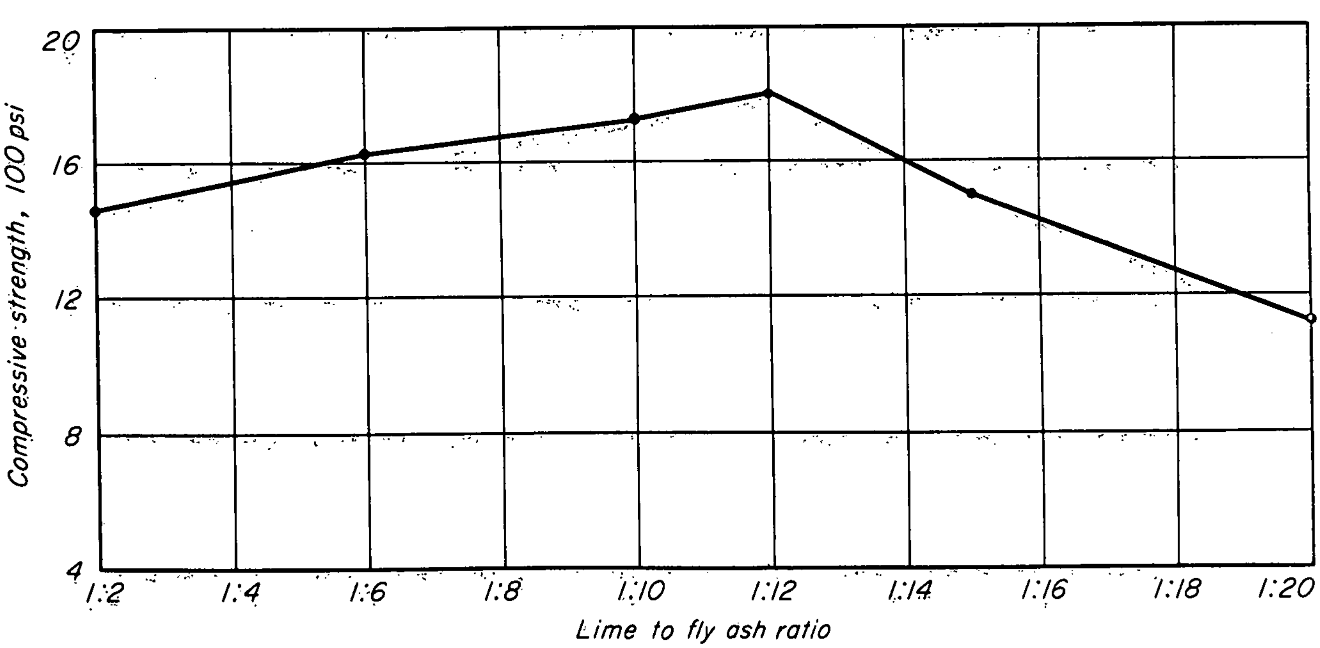
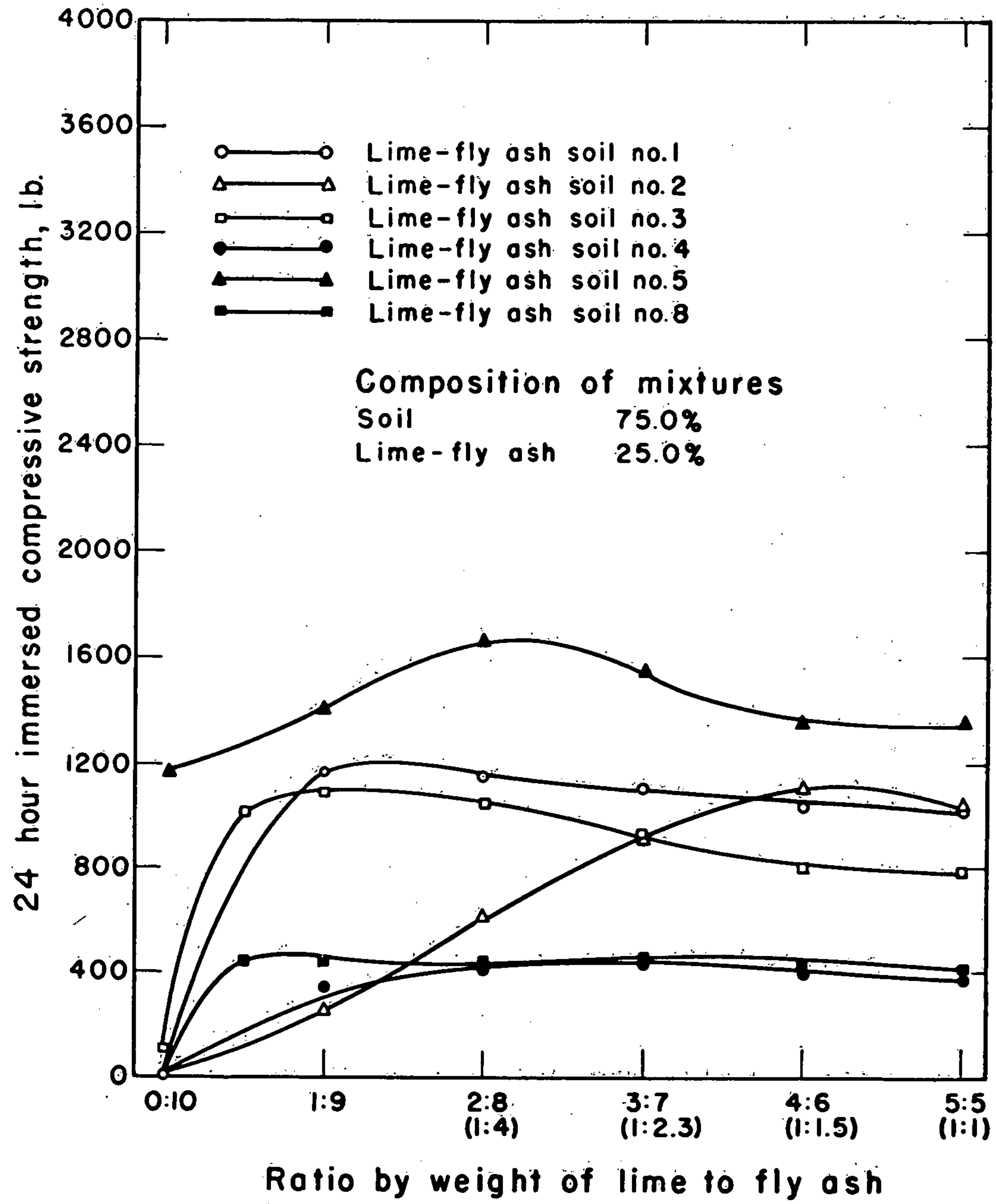


Figure 24. Effect of lime-to-fly ash ratio on the compressive strength of a lime-fly ash–slag mixture (60 percent slag, 40 percent lime plus fly ash) (24).



Notes:

Soil No.	AASHTO Classification
1	A-7-6(20)
2	A-7-5(18)
3	A-4(8)
4	A-6(8)
5	A-2-4(0)
8	A-7-5(15)

Figure 25. Effect of lime-to-fly ash ratio on compressive strength development (28-day curing, 100 percent humidity, 70 F) (6).

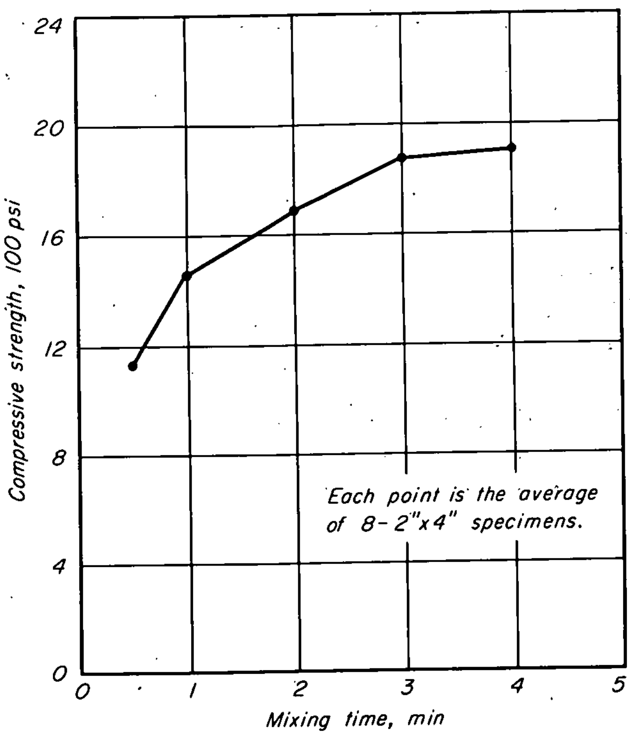


Figure 26. Effect of mixing time on compressive strength development for a lime-fly ash–slag mixture (24).

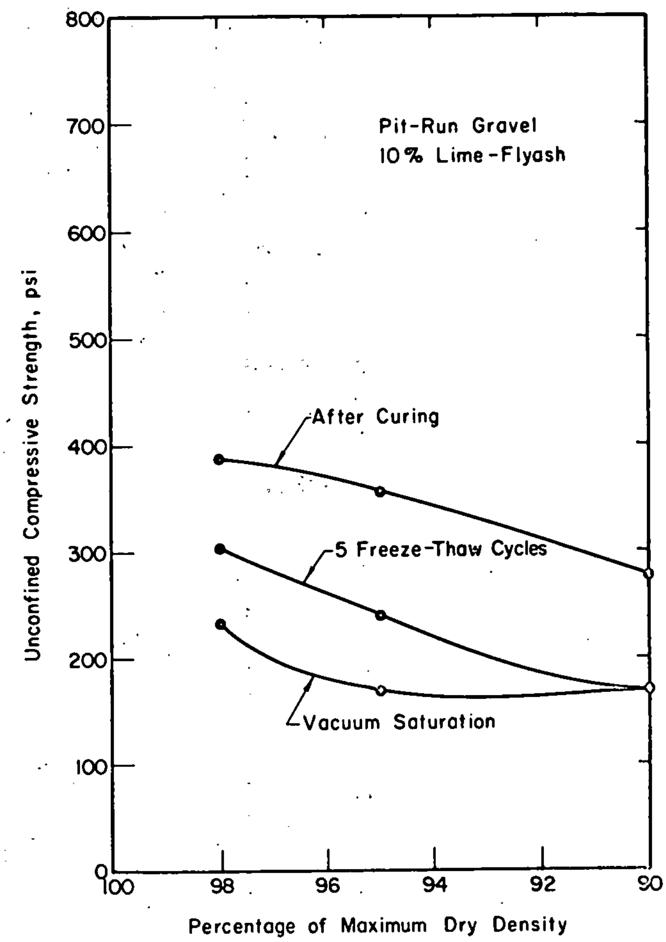


Figure 28. Influence of density on the strength and durability of a lime-fly ash–aggregate mixture (36).

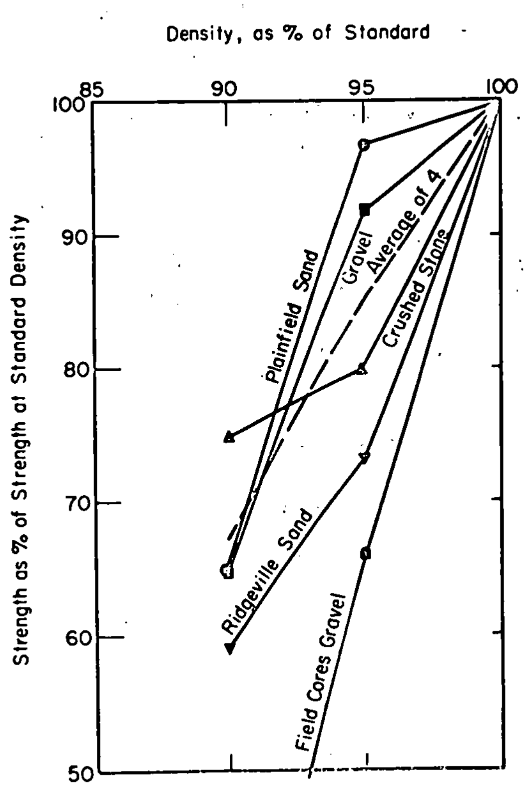


Figure 27. Effect of density on the compressive strength of lime-fly ash-stabilized mixtures (22, 36).

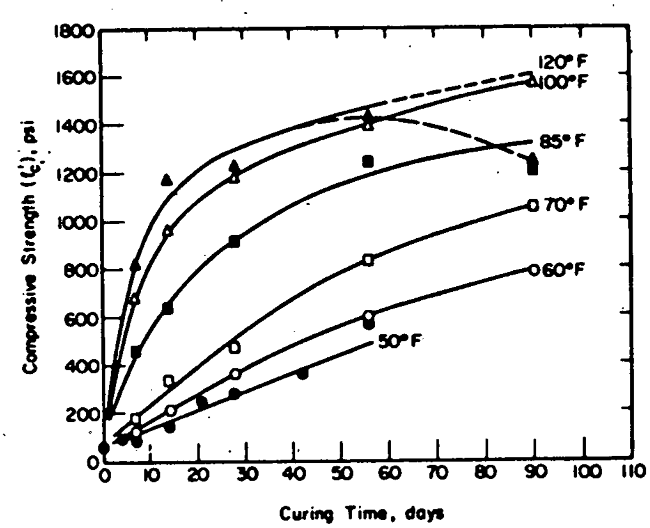


Figure 29. Effects of curing time and temperature on the strength development of a lime-fly ash–aggregate mixture (20).

stantially retarded at low temperatures [less than approximately 40 F (4 C)].

Field curing occurs at varying temperatures. The "degree day" concept (*20, 26*) can be used to consider "mixed" curing conditions as illustrated in Figure 30. It is important to note that higher temperature curing data do not produce the same "strength-degree day" relation as lower temperature curing.

Strength development will continue in LFA mixtures after the termination of the formal curing period. Figure 3 illustrates field strength development for a typical LFA mixture constructed in Chicago (*23*).

If LFA mixtures are used in areas where low temperatures may occur, it is necessary to consider time-temperature curing effects on fall construction. Adequate curing must be obtained prior to the beginning of cyclic freeze-thaw to assure satisfactory field performance (*32*). If a minimum LFA mixture strength has been established for a particular situation, curing requirements (temperature-time) must be established accordingly.

It is emphasized that the strength-degree day relation is a property of a particular LFA mixture. Fly ash, lime, and aggregate properties, as well as mixture composition, will influence the temperature-time dependent strength development.

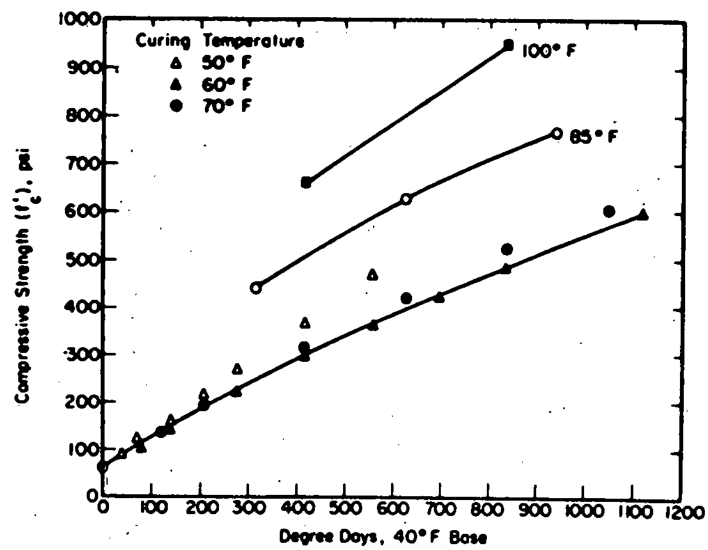


Figure 30. Degree day–strength relationships for a lime-fly ash–aggregate mixture (20).

CHAPTER FOUR

PAVEMENT BEHAVIOR AND PERFORMANCE

PAVEMENT BEHAVIOR

Although most pavements with lime-fly ash–aggregate (LFA) materials are classified and designed as flexible pavements, as indicated in Chapter Three, these materials, when cured, can develop moduli values to well over two million psi (14 000 MPa). With moduli values of this magnitude, it can be expected that pavements with cured LFA mixes will behave essentially as slabs rather than as "flexible" pavements. Test data on full-scale and model pavements confirm that the theoretical load distribution characteristics of pavements with layers of LFA mixes are essentially those of a slab (*23, 39*). Because of the rigid nature of the material, slabs are susceptible to cracking caused by thermal and moisture changes in the pavements.

Detailed studies on behavior of LFA pavements are reported by Ahlberg and Barenberg (*23*) and Barenberg (*40*). In these studies, model pavements from approximately 3 to 6 in. (75 to 150 mm) thick were loaded at different stages of curing with static plate and dynamic wheel loads. In the plate load tests, surface deflections were determined by measuring the loaded plate deflections, and the deflection basin determined by measuring the pavement surface deflection at points outside the loaded areas. Some typical results of these tests are shown in Figures 31 through 34. Figure 31 shows the relative load deflection patterns for an LFA slab compared with conventional flexible pavements. Figure 32 shows the load distribution effects of a 4-in. (100-mm) LFA slab compared with a 9-in. (230-mm) conventional flexible pavement. Figure 33 shows a comparison between typical measured deflection patterns and the theoretical deflections indicated by Westergaard slab theory for interior loading conditions. Figure 34 shows similar data for loads applied near the edge of the slabs. The relative deflections of a flexible and an LFA pavement under a wheel load moving at creep speed are shown in Figure 35.

The load deflection characteristics and ultimate load carrying capacity of the LFA slabs are a function of the amount of curing of the LFA material, as indicated by the material's resistance to flexural deformation. The curves in Figure 36a reflect the load deflection curves for six 4-in.-thick (100-mm) LFA slabs tested to failure under inferior loading conditions applied through a 7-in.-diameter (175-mm) steel plate. The flexural strength of the LFA material at the time of testing for each pavement is shown in the figure. Similar information for two pavements tested under edge loading conditions is shown in Figure 36b. Failure in the slabs under interior loads consisted of a shear-type cone punching through the slabs. The shape of the load deflection curves for the interior loadings does not indicate any preliminary failure modes or abrupt changes in the load deflection properties of these pavements. Upon removal of the LFA slabs, however, the bottoms of the slabs in the region of the loaded area were severely cracked, indicating slab bending failure prior to the shear punchout (*23, 39*). For the edge load conditions, the failure pattern is different, and this is reflected by the break in the load deflection curves in Figure 36b. Failure of the LFA slabs under edge loadings follows the classical pattern outlined by Meyerhof (*47*). That is, as the load is increased, radial cracks initiate at the point of maximum stress, which is assumed to be directly under the load, and propagate upward and outward from the loaded region. With the propagation of the radial cracks, there is a redistribution of internal stresses causing an increase in the radial stresses at the pavement surface near the point of maximum negative bending. When negative bending stress reaches the flexural strength of the LFA, a semicircular crack develops on the surface near the point of maximum negative stress. The break in the load deflection curves shown in Figure 36b is probably the point at which radial cracking initiated under the applied edge load. Although the pavement can carry additional load, this break point in the load deflection curve is probably the load that should be considered as the design ultimate for sustained pavement performance.

The ultimate strength of the LFA slabs under static load has been shown to far exceed the strength predicted by elastic slab theory. Figure 37 shows the effect of flexural strength of the LFA mix on the theoretical and observed strength of the LFA slabs under interior and edge loading conditions. Note that the observed ultimate load under edge loading agrees well with that predicted by the Meyerhof theory (*47*), whereas for the interior loading condition the observed ultimate load is greater even than that predicted by Meyerhof's collapse load theory. For both edge and interior load conditions, the theoretical ultimate load failure condition is nearly two and one half times greater than the corresponding failure load as determined by the Westergaard theory. Thus, results from the plate load tests clearly show that the behavior of pavement with LFA materials is essentially that of a slab, but that the load-carrying capacity of this slab under single static load is likely to be significantly greater than predicted by elastic slab theory.

The reason for the greater load capacity of the LFA slab is understandable in light of the known properties of these materials. LFA materials, like concrete and other similar paving materials, are assumed to be quite brittle in nature. Although the stress-strain curves of these materials in flexure tests make them appear brittle, carefully controlled tests show them to have rather poorly defined yield stress levels. Beyond this level, the materials undergo significant plastic

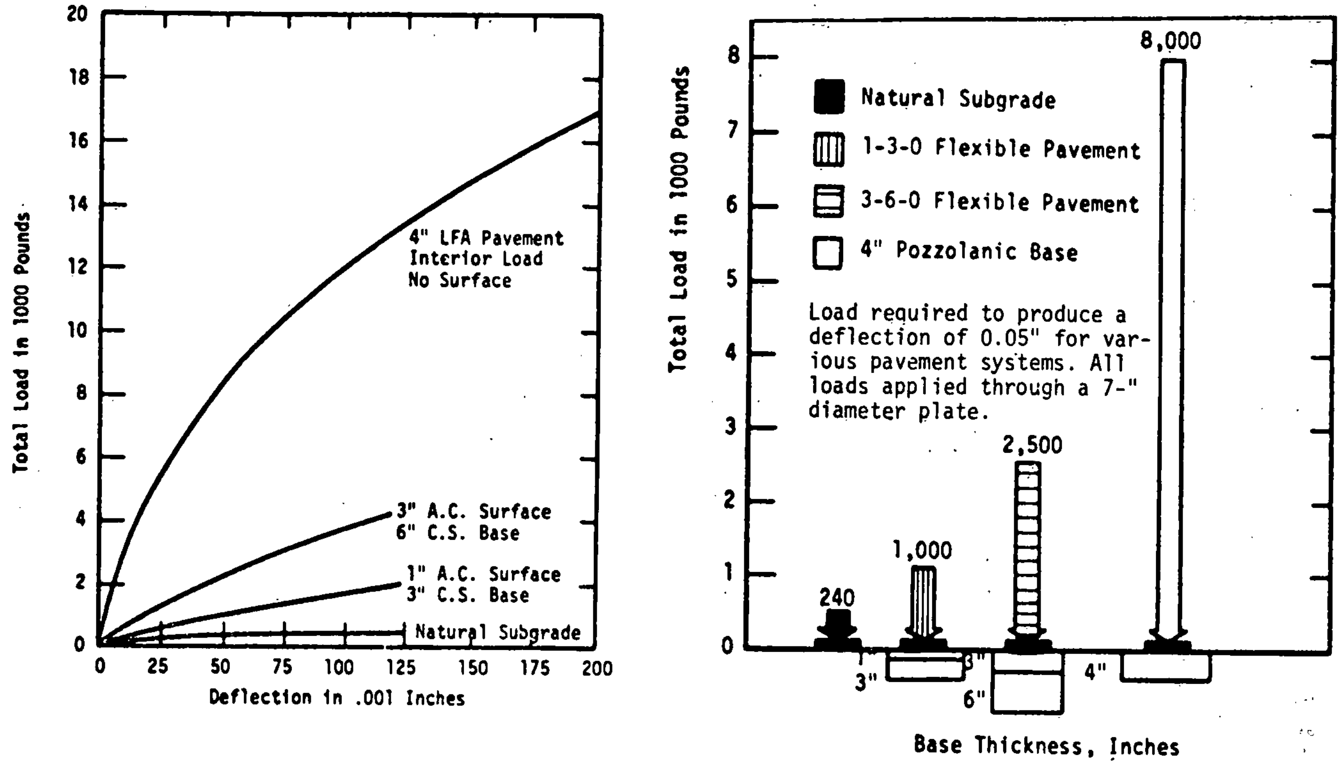


Figure 31. Comparison of load deflection characteristics of LFA pavements with typical flexible pavements.

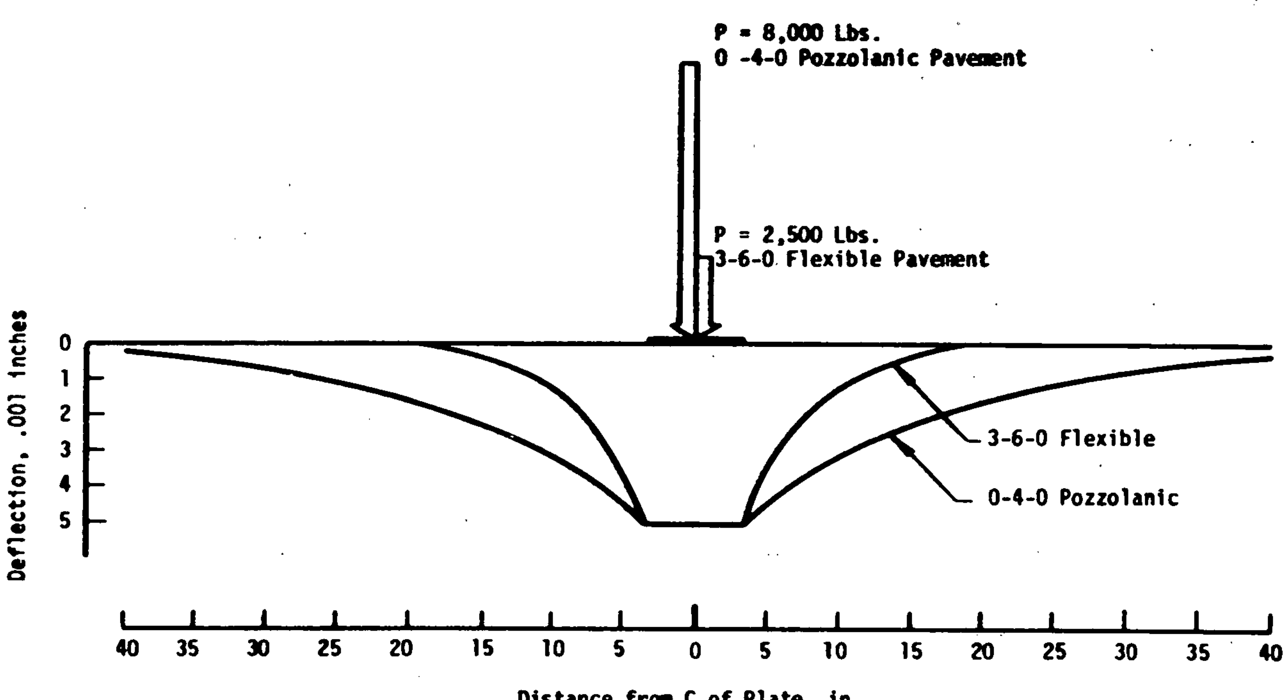


Figure 32. Deflection profiles for an LFA slab and a typical flexible pavement under a static load applied through a 7-in.-diameter plate.

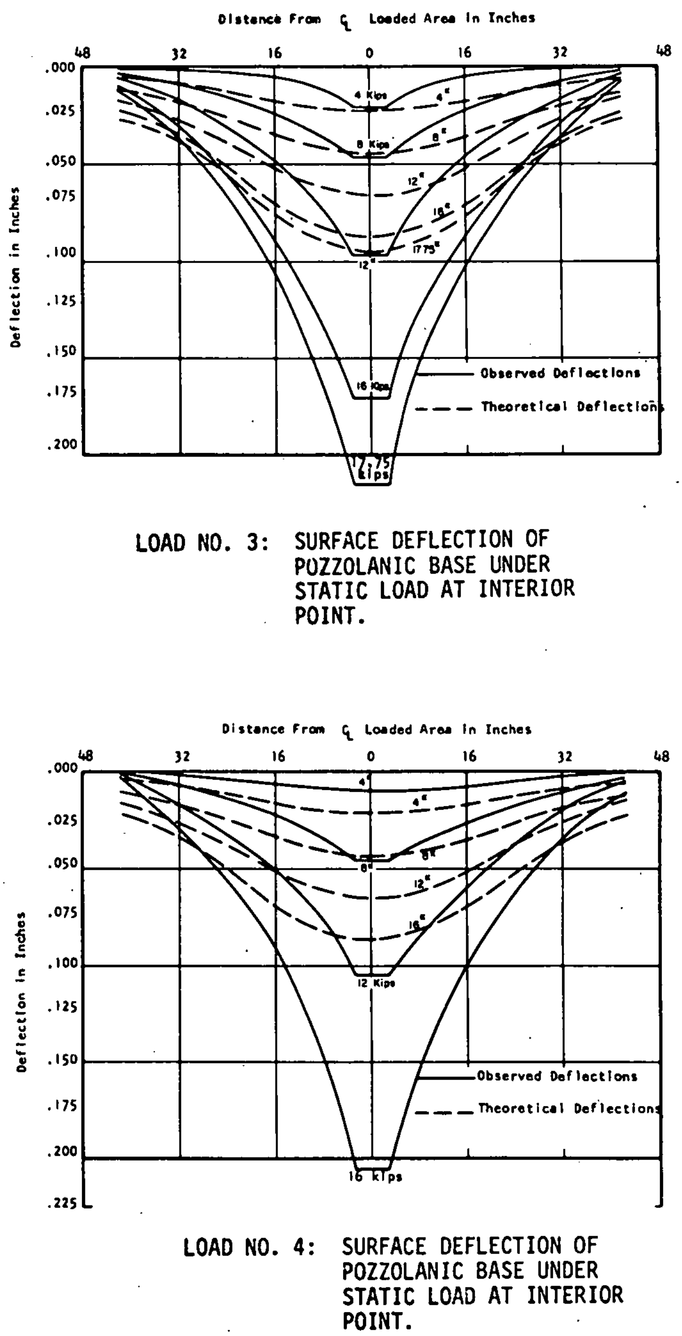


Figure 33. Typical load deflection patterns for LFA slabs under static plate loads (interior).

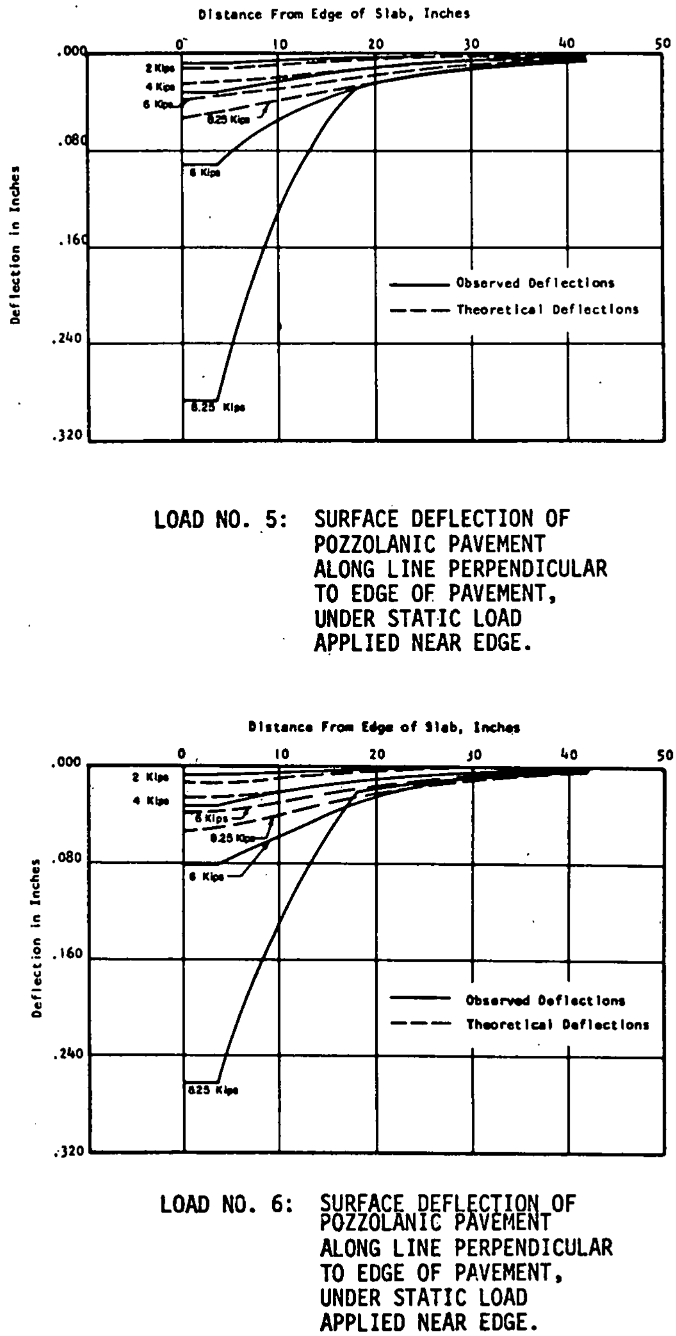


Figure 34. Typical load deflection patterns for LFA slabs under static plate loads (edge).

flow without apparent gross rupture. During this yielding process, there is a concurrent redistribution of stresses within the slab that accounts for the slab's supporting substantially higher loads than indicated by the elastic slab theory. Although gross rupture does not usually occur at initial yielding, microscopic rupture is probably occurring within the material mass that, under repeated load applications, may lead to fatigue failure. Thus, for reliable design, stress levels in the slab should be kept in the elastic (pre-yield) range.

Recent field studies reported by Hirst, Fang, and Schmidt (*45*) also emphasize the slab behavior of the LFA materials. Pavement sections with typical base course materials, including LFA materials, used by the Pennsylvania Department of Transportation (PENNDOT) were evaluated by deflection techniques in these studies. Dyna-

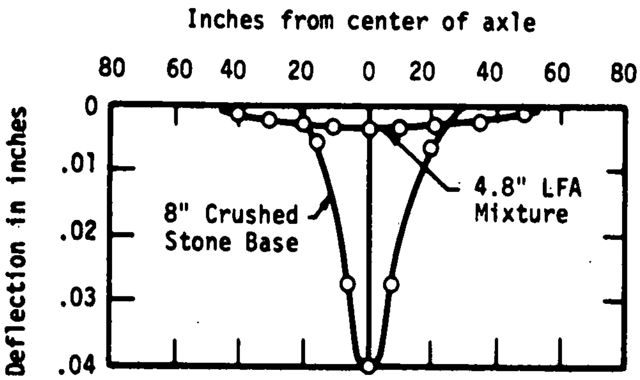
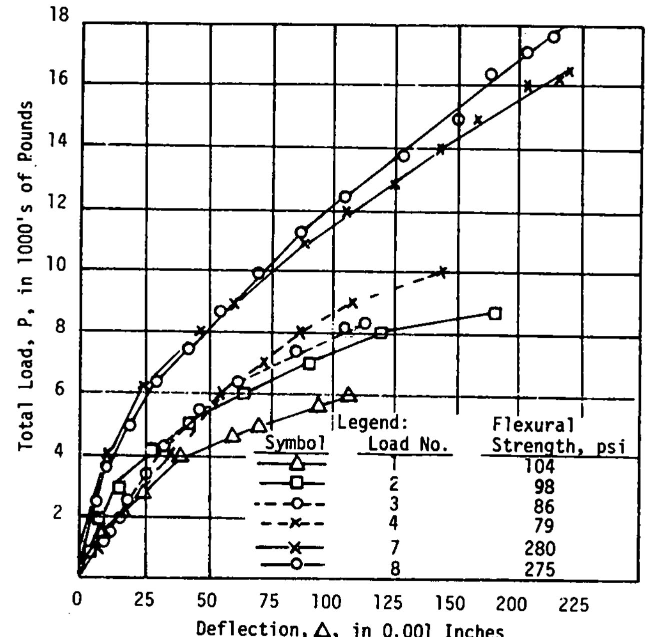
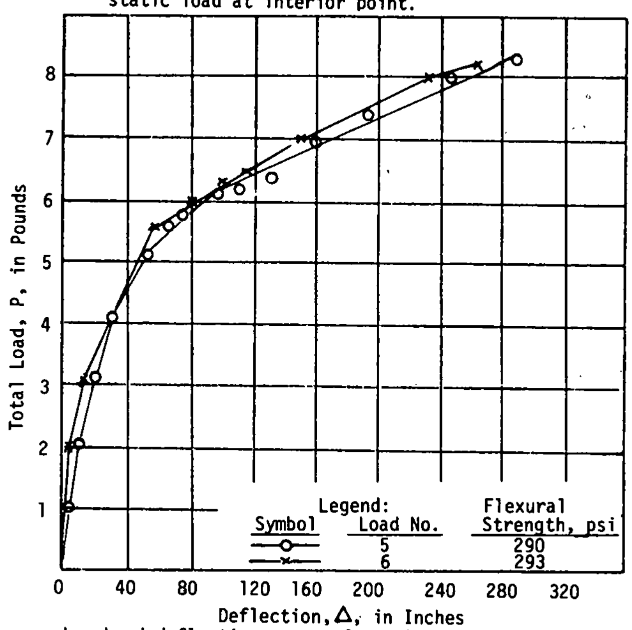


Figure 35. Typical base deflection pattern under a moving load.



a. Load Deflection curves for LFA pavements under static load at interior point.



b. Load deflection curves for LFA pavements under static loads applied near an edge of the slab.

Figure 36. Load deflection behavior of LFA slabs under static loading conditions.

flect and plate load tests were run on a number of pavement sections, and relative structural coefficients were determined based on the ability of these pavements to resist deformation due to load. Moduli values for the different base materials were calculated by correlating measured deflection values with theoretical values from elastic layer theory and finite element methods of analyses. Results from these studies show that the effective modulus for the LFA base materials is approximately 1,000,000 psi (6 900 MPa). Corresponding structural coefficients calculated from these deflection data are in the order of 1.3 to 1.6 compared with values of 0.10 to 0.16 for dense-graded crushed stone bases.

Creep-speed wheel load tests on full-scale airport pavements with LFA layers totaling up to 32 in. (800 mm) thick, are reported by Yang (*49, 50*). In these tests, a test vehicle weighing 187,000 lbs (84 000 kg) was used to simulate the load from the landing gear of the Boeing 747 aircraft. Up to 5,000 passes with the test vehicle were made over instrumented test sections of these pavements to evaluate the behavior and performance of the LFA layers.

A significant finding from this study is that the use of the LFA material in lieu of a crushed stone as the base material does not significantly reduce the total pavement surface deformation, but does greatly improve the deformation recovery of the pavement. This is indicated in Figure 38. As Yang observed, "The total surface deformation is largely contributed by the deformation of the subgrade. However, the longer, smooth deflection configuration of the stabilized base (LFA) will produce a more durable and better performing pavement" (*50*).

Because the aircraft loads are large and are distributed over such a large area, the effect of pavement stiffness on total pavement deformation is small. This is because nearly all of the deformations (>80 percent) occurs in the subgrade. The stiffer LFA layers, however, distribute the load over larger areas of the subgrade, thus reducing the maximum vertical stress transmitted to the subgrade as well as the amount of permanent rutting in the pavement. Figure 39 shows the permanent rutting trend versus pavement thickness observed in the Newark Airport pavement studies.

Plate load tests were also run on various layers of a heavy-duty LFA pavement constructed for the Portland Port Authority, Portland, Oregon, for one of their marine terminals. Pavements were constructed with up to 20 in. (500 mm) of LFA materials in three layers with a 3.5-in. (89-mm) asphaltic concrete surface. Approximately six months after placement of the LFA layers, the surface

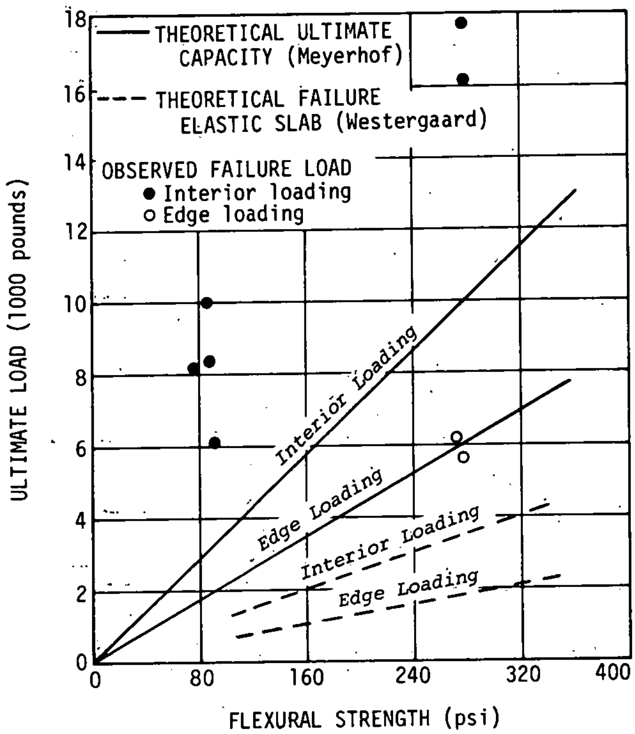


Figure 37. Comparison between the theoretical and observed load-carrying capacity of LFA pavement slabs.

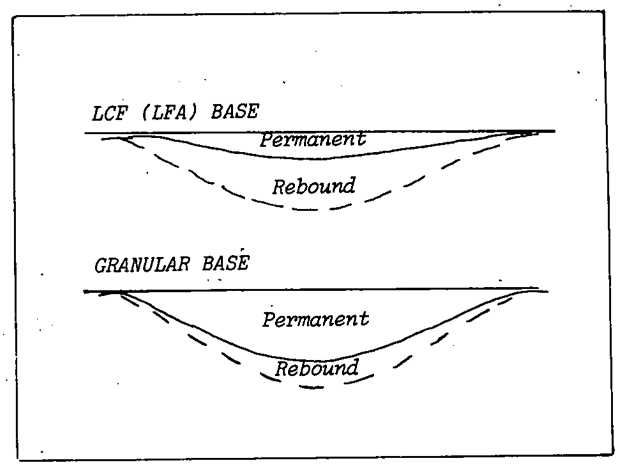


Figure 38. Relationship between permanent and rebound deformations observed in the Newark Airport test sections.

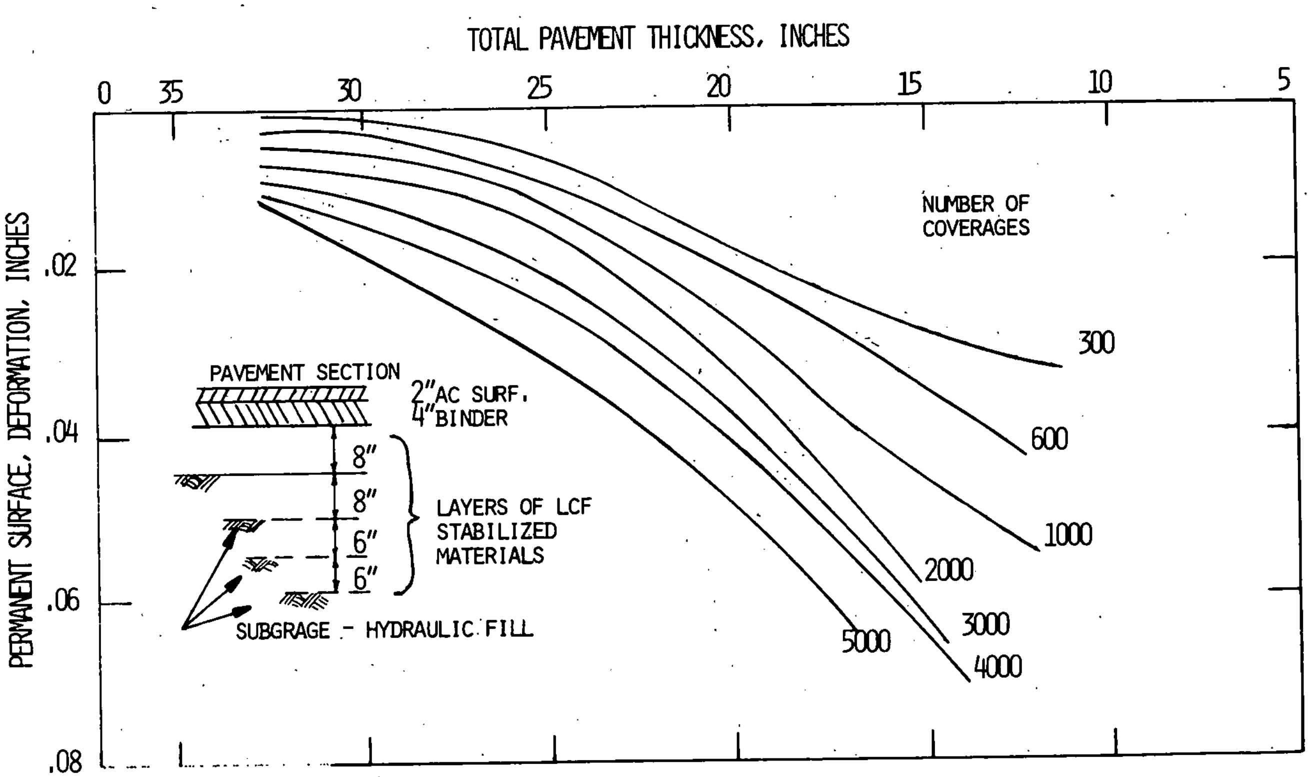


Figure 39. Effect of pavement thickness on the performance of pavements for the Newark Airport.

and succeeding layers were carefully removed and plate load tests run on the three layers of LFA base material as well as on the top of the subgrade. Using the elastic layer theory, the effective moduli values were determined for each of the three layers (see Table 6) (*48*).

Each of the three mixes used in the pavement had different proportions, and the corresponding compressive strengths at the time of testing were 1,670 psi (11 500 kPa), 560 psi (3 900 kPa), and 1,760 psi (12 000 kPa) for the A, B, and C mixes, respectively. There was no viable explanation why mix B was weaker than mixes A and C; the mixes supposedly increased in quality from the top to the bottom of the pavement (i.e., from mix A to C).

Based on the responses of pavements with LFA materials reported in the literature, it seems reasonable to conclude that these pavements distribute loads essentially by slab action. The effect of the slab action on total pavement deflection is not consistent, however, because it depends upon the manner in which the load is applied. With the applied loads distributed over very large areas, such as with aircraft gear, the effect of the slab action on total pavement deflection is much less than when the loads are applied over relatively small loaded areas.

TABLE 6

EFFECTIVE MODULI VALUES OF LFA PAVEMENT LAYERS AT PORTLAND PORT AUTHORITY (*48*)

LAYER	MIX	EFFECTIVE MODULI, PSI (MPa)
Top	A	580,000 (4 000)
Middle	B	112,000 (770)
Bottom	C	630,000 (4 300)
Subgrade	Hydraulic fill	43,000 (300)

PAVEMENT PERFORMANCE

Performance of pavements with LFA materials is affected by many factors. Some of these can be taken into account directly in the design procedure, whereas others are more subtle and need to be considered only as part of the overall design approach. The primary factors affecting the performance of these pavements are: (*a*) loading and the interrelationships between load, slab thickness, material strength, etc.; (*b*) durability of the LFA material as related to the environment in which it must serve; (*c*) quality of construction, including uniformity of the final product; and (*d*) subsurface drainage of the pavement system.

In evaluating the performance of paving materials, it is important to keep several factors in mind. All paving materials change dimensionally with changes in moisture and temperature, and unless the material has enough ductility to absorb these dimensional changes, the pavements crack. In addition, there is often a great difference between assumed material properties and properties of materials as actually placed *in situ*. Many of the factors affecting pavement performance are interacting, that is, the presence of one factor such as moisture increases or decreases the effects of other factors such as loading or cyclic freezing and thawing. As a consequence, there are many subtleties involved in evaluating the performance of paving materials.

As compared to other paving materials, LFA mixes are relatively new. Thus, long-term, documented histories of pavement performance with these materials are not generally available. Several pavement performance evaluations have been reported, however, that permit the determination of the potential performance of pavements with LFA mixes, and a few reports are available with actual performance evaluations with pavements having up to 16 years of service.

Studies on performance of pavements with LFA mixes can be broken into three categories: (1) scale-model testing of pavements under laboratory and quasi-laboratory conditions (*23, 39, 41*), (2) short-term evaluation of full-scale pavements with conventional highway and aircraft loading under simulated service conditions (*43, 45, 49, 50, 52, 53*), and (3) evaluation of performance of pavements in service for a number of years under normal traffic conditions (*40, 54, 55*). Each of these types of studies adds to knowledge of the performance of pavements with LFA mixes and the factors that influence their performance.

Performance trends of scale-model LFA slabs under accelerated wheel loadings under laboratory conditions are reported by Ahlberg and Barenberg (*23, 39*), and Barenberg (*41*). Some typical results from these studies are shown in Figure 40. These results show that because of the strength-gain characteristics of LFA materials with time, if the ratio of applied to ultimate load as indicated by the Meyerhof theory is less than approximately 0.6 at the time of initial loading, these pavements do not fail in fatigue. If, on the other hand, these pavements are severely overloaded at an early age, the LFA materials crack and deteriorate under repeated loading.

These observations are valid only if the pavements are new at the time of initial loading. As the LFA material becomes more mature with added curing, the rate of strength gain decreases. For the strength-gain effects to completely affect the cumulative fatigue damage at more mature stages in development, the load-to-ultimate load ratio has to be correspondingly lower as the pavement matures. Also, for pavements with sufficient maturity to have well-developed shrinkage cracks, the ultimate load calculations have to be based on edge loading rather than interior loading conditions.

A review of the performance of LFA pavement in service by Barenberg (*40*) also shows that pavements that were not overloaded at an early age did not fail in fatigue. In this study 16 pavements, some in service for 16 years or more at the time of the study, were reviewed for performance and general distress. A summary of the pavements evaluated in the study is given in Table 7. Based on results of this survey, the author concluded that so long as the ultimate load capacity of the pavement under edge loading, as determined from the Meyerhof theory, was 1.5 to 2.0 times the applied load (load ratio of 0.5 to 0.7), the pavements did not fail in fatigue because of repeated traffic loads. Thus, the performance of these materials, insofar as load applications are concerned, can be developed around a strength-versus-thickness criterion for various traffic conditions.

TEST PAVEMENT NO. II-1
WHEEL LOAD 3200 LBS.
BASE MATERIAL POZZOLANIC
BASE THICKNESS 4.3"
SURFACE MATERIAL MORTAR
SURFACE THICKNESS NOMINAL

RATIO P/P_u 1.00

TEST PAVEMENT NO. II-2
WHEEL LOAD 3200 LBS.
BASE MATERIAL POZZOLANIC
BASE THICKNESS 4.8"
SURFACE MATERIAL MORTAR
SURFACE THICKNESS NOMINAL

RATIO P/P_u 0.80

TEST PAVEMENT NO. II-3
WHEEL LOAD 3200 LBS.
BASE MATERIAL POZZOLANIC
BASE THICKNESS 5.3"
SURFACE MATERIAL MORTAR
SURFACE THICKNESS NOMINAL

RATIO P/P_u 0.68

TEST PAVEMENT NO. II-4
WHEEL LOAD 3200 LBS
BASE MATERIAL POZZOLANIC
BASE THICKNESS 5.8"
SURFACE MATERIAL MORTAR
SURFACE THICKNESS NOMINAL

RATIO P/P_u 0.56

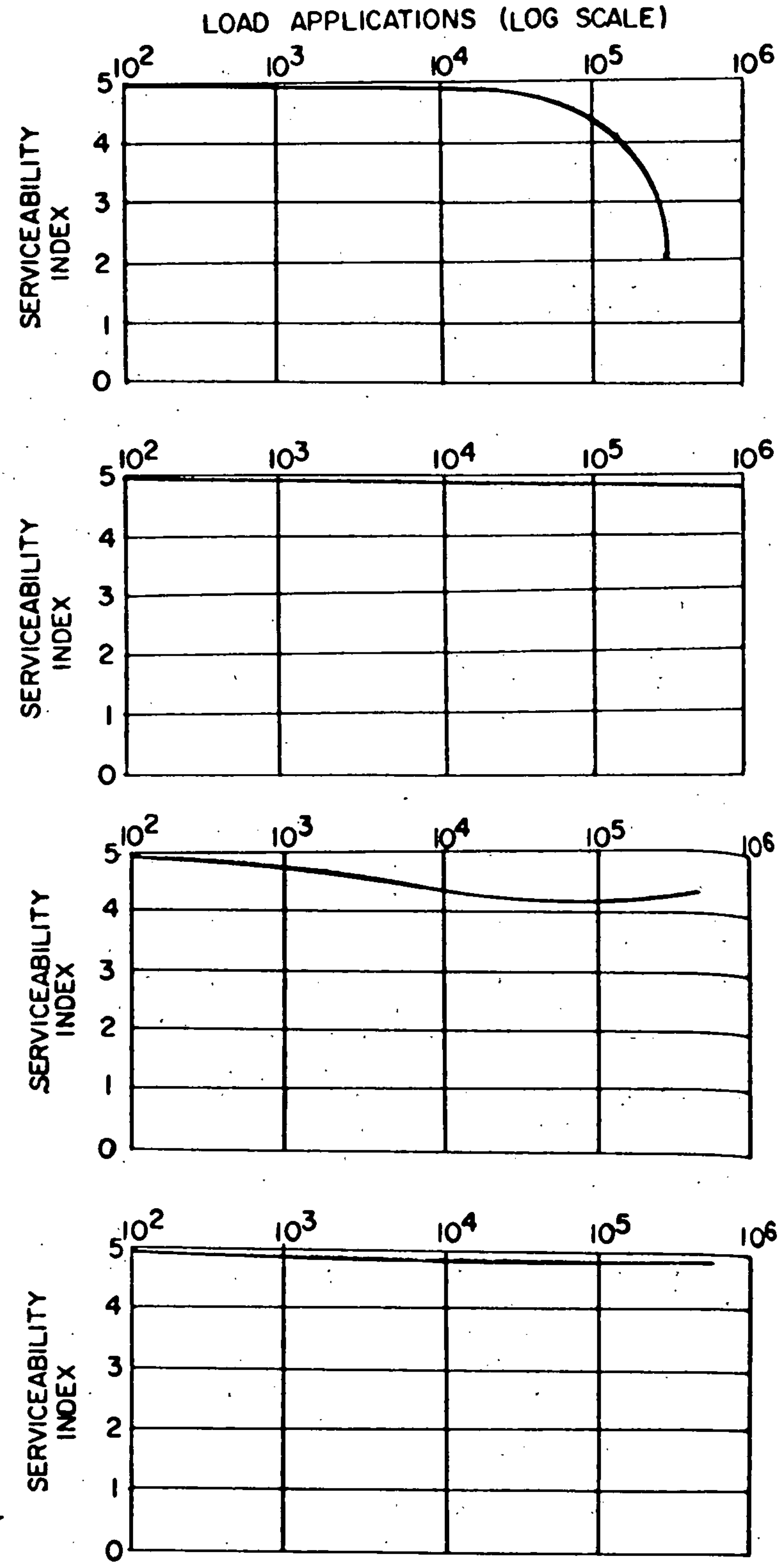
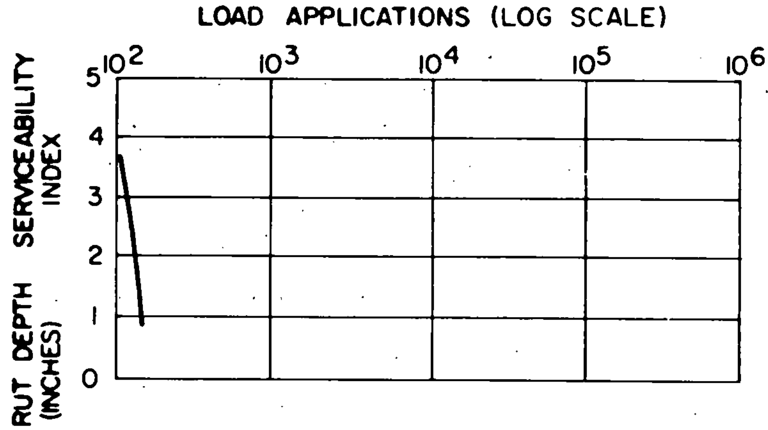


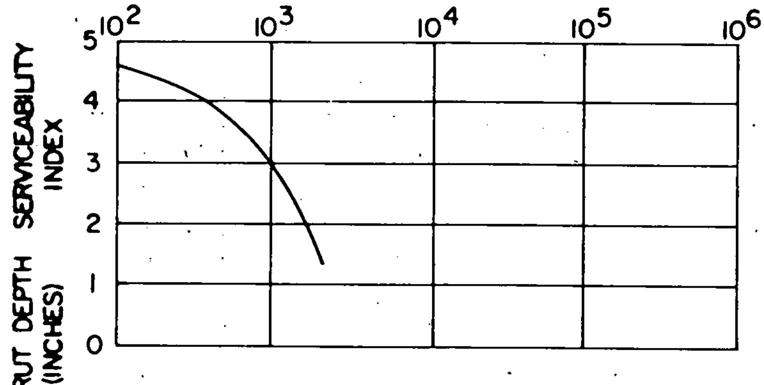
Figure 40a. Performance trends for LFA pavements under laboratory conditions.

PAVEMENT PERFORMANCE TRENDS

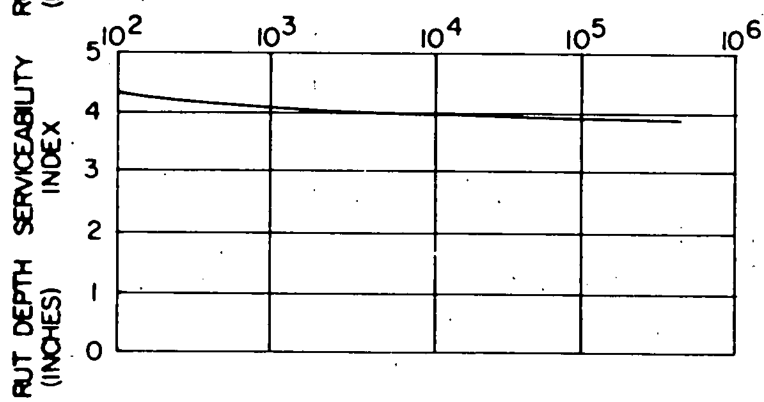
TEST PAVEMENT NO. VI-1
WHEEL LOAD 3200 LBS
BASE MATERIAL POZZOLANIC
BASE THICKNESS 4.0"
SURFACE MATERIAL NONE
SURFACE THICKNESS —
RATIO P/P_u 1.95



TEST PAVEMENT NO. VI-2
WHEEL LOAD 3200 LBS
BASE MATERIAL POZZOLANIC
BASE THICKNESS 4.0"
SURFACE MATERIAL NONE
SURFACE THICKNESS —
RATIO P/P_u 0.87



TEST PAVEMENT NO. VI-3
WHEEL LOAD 3200 LBS
BASE MATERIAL POZZOLANIC
BASE THICKNESS 4.0"
SURFACE MATERIAL NONE
SURFACE THICKNESS —
RATIO P/P_u 0.65



TEST PAVEMENT NO. VI-4
WHEEL LOAD 3200 LBS
BASE MATERIAL POZZOLANIC
BASE THICKNESS 5.5"
SURFACE MATERIAL NONE
SURFACE THICKNESS —
RATIO P/P_u 0.35

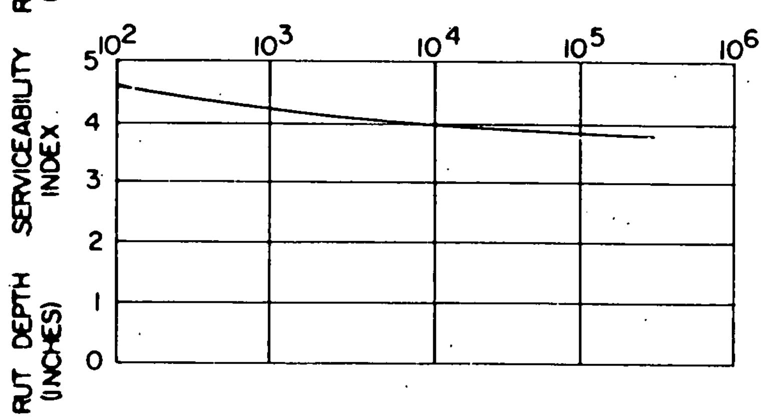


Figure 40b. Performance trends for LFA pavements under laboratory conditions.

TABLE 7

SUMMARY OF PAVEMENT SECTIONS AND PAVEMENT CONDITIONS

Pavement Number	Pavement Thickness (inches) Surface	Base	Subbase	Compressive Strength of Pozzolanic Base Material (psi) 7 Days @ 130 F	Field Cores	Age of Field Cores Years	Estimated Traffic Daily Passenger Cars	All Trucks	Heavy Trucks	Remarks
1	2	7-9 (8)	0	1000	2160	8	4000	600	600	Failed along edge for about 30 to 40% of length.
2	2	7-9	0	1000	1925	8	4000	150	10	Trench settlement failure only.
3	1.5	3-12 (8)	0	1000	3240	6	2000	75	5	15 to 20% of total area failed.
4	2.5	8	4.5	1000	1955	1	3000	325	25	Slight edge distress only.
5	3	6.5-11	4-24 (10)	1000	2600	5	400	500	200	Slight edge distress only.
6	1.5	4-13 (7)	12	--	1310	3	3000	150	50	2 to 4% of total area failed.
7	2-3	5-6	0-12	--	1720	5	--	--	--	No distress.
8	1.5	6	4	1165	1310	3	3000	700	100	2 to 3% of total area failed.
9	2	8	0	1150	1200	8	--	--	--	No failures. See discussion.
10	2	6	0	1350	1090	3	--	--	--	1 to 2% of total area failed in areas of soft subgrade only.
11	2	10	-	--	No Cores	6	5000	300	variable	1 to 2% total area distressed. See discussion.
12	3	10	-	1705	1700	7				No distress.
13	2	6	-	--	1100	7	2000	200	50	3 to 5% area distress occurred during first year, did not progress.
14	2	8	-	1104	600	7	--	--	--	No distress.
15	2.5	8	-	1795	825	7	--	250	200	Some map cracking % area not indicated.
16	2	8-10	-	740	1800	8	--	200	150	No distress.

Another way of evaluating the performance of LFA materials is to consider the structural coefficients that have been assigned by various design agencies. Such coefficients are developed around some short-term evaluations, and then adjusted on the basis of the over-all performance of pavements in service. Such coefficients have a somewhat broader connotation than mere strength, because they reflect the actual performance of the materials in service. In-service performance is a function of durability of the material and construction variability as well as the basic material characteristics.

Following the concept developed by the AASHTO Committee on Pavement Design from the results of the AASHO Road Test (*51*), the structural capacity of a flexible pavement can be defined by the relationship

$$SN = a_1D_1 + a_2D_2 + a_3D_3 \tag{2}$$

in which

SN = the structural number or structural capacity of the pavement;

D_1, D_2, D_3 = thicknesses of the surface, base, and subbase, respectively; and

a_1, a_2, a_3 = material coefficients, often referred to as structural coefficients.

This relationship indicates that to achieve a specified structural capacity there is an inverse linear relationship between the structural coefficient and the thickness of each layer in a flexible pavement system. Usually, there are minimum material standards associated with the assigned coefficients. In addition, a range of coefficients can be used to reflect different qualities for specific materials. Ahlberg and Barenberg (*23*) suggest the coefficients for LFA materials given in Table 8.

No specific limits for compressive strength or modulus of elasticity were assigned by Ahlberg and Barenberg for the three categories of LFA, but a review of the background data suggests the ranges given in Table 9. Note that the modulus of elasticity does not have a 1:1 relationship with compressive strength. Note also that even the lowest quality of LFA must meet realistic durability criteria to be acceptable.

A field study on the performance of stabilized base materials including LFA materials is reported by Dunn (*43*). Base materials tested included lime-fly ash–aggregate mixtures, cement-treated aggregates, and two grades of bituminous stabilized aggregates. These materials were tested in the test track operated by Pennsylvania State University. This test facility is a one-mile-long test loop that is loaded by trafficking the test pavements with standard truck vehicles. Both pavement behavior and pavement performance were evaluated in this study.

Figures 41 and 42 show the performance trends for pavements with LFA mixtures as reported by Dunn (*43*). Trafficking on these pavements was started in October 1972, and required nearly two years to accumulate the million-plus equivalent 18-kip (80-kN) single-axle loads shown in the figures. Private communication with supervisors of the test facility indicates that these sections were still under test and giving satisfactory performance as of mid-1975. Table 10 gives the relative structural coefficients for the four stabilized materials as determined from the Pennsylvania State University test track, and the current values used by the Pennsylvania Department of Transportation for these same materials. All values are based on a standard value of $a_2 = 0.14$ for high-quality crushed stone.

The higher-than-expected values obtained from the test track results can be explained in two ways. In evaluating these coefficients, it must be kept in mind that if any of the stabilized base materials (especially the aggregate-cement and aggregate-lime pozzolan bases) are significantly overloaded, they crack and the continuity of these bases is partially lost. Once these bases are severely cracked, there is a rapid loss in serviceability level with continued loading. Thus, some minimum thickness standard is required, along with the structural coefficient values, for sound pavement design. It is anticipated that, because of the controlled

TABLE 8

STRUCTURAL COEFFICIENTS OF LFA MATERIALS (*23*)

QUALITY OF LFA	STRUCTURAL COEFFICIENT (a_2)
High	0.34
Medium	0.28
Low	0.20

TABLE 9

RANGES OF COMPRESSIVE STRENGTH AND MODULUS OF ELASTICITY FOR LFA MATERIALS

QUALITY	COMPRESSIVE STRENGTH [7 DAYS AT 100 F (38 C)], PSI	(kPa)	MODULUS OF ELASTICITY, PSI × 10^3	(MPa)
High	>1000	(>6900)	>500	(>3400)
Medium	650-1000	(4500-6900)	250-500	(1700-3400)
Low	400-650	(2800-4500)	100-250	(690-1700)

TABLE 10

RELATIVE STRUCTURAL COEFFICIENTS OF FOUR STABILIZED MATERIALS (PENNDOT)

BORE MATERIAL	a_2 (TEST VALUE)	a_2 (CURRENT USE)
Aggregate-cement base	0.55	0.30
Aggregate-lime-fly ash base	0.51	0.30
Bituminous concrete base	0.51	0.40
Aggregate-bituminous base	0.49	0.30

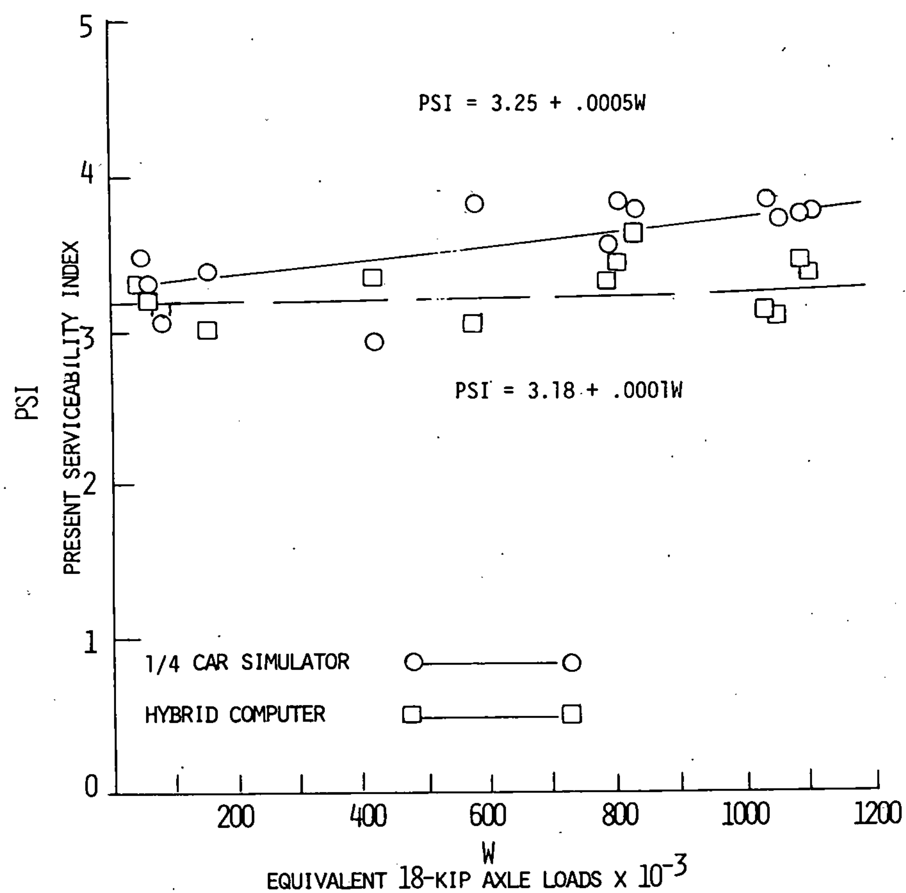


Figure 41. Pavement performance curves for Section 3; 2.5-in. AC surface, 8-in. LFA base, and 8-in. granular subbase.

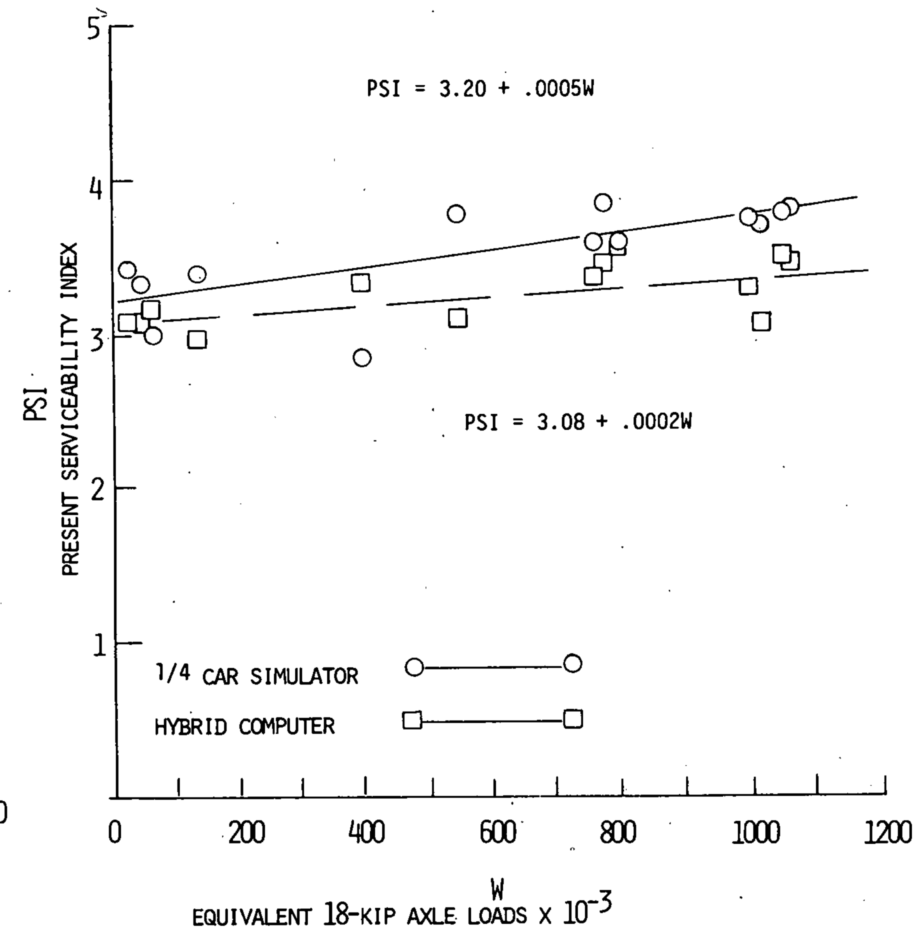


Figure 42. Pavement performance curves for Section 12; 2.5-in. AC surface, 8-in. LFA base, and 8-in. granular subbase.

loading, the test pavements were never overloaded. Thus, the high apparent structural values compared with the crushed stone.

The quality control employed during the construction process could also account for the high structural values. No specific data are available on the quality control used in construction of the test pavements, but it seems likely that because this is a test facility, the level of quality control would be greater than in normal pavement construction. Because most stabilized materials are more sensitive to variations in product quality than the unbound aggregates, it follows that the stabilized materials in the test pavements would give better relative performance than in normal use conditions.

Typical structural coefficients recommended by several states are given in Table 11. Direct comparisons of these values are not totally valid, because each agency uses slightly different values for the crushed stone base materials, which are normally considered to be the standard. However, the values shown provide a general indication of the relative performance of the LFA materials under normal service conditions.

Based on the structural coefficients used by most agencies and the results of short-term evaluations, the performance of LFA mixes of intermediate quality compares favorably with that of cement-treated aggregates having a 7-day compressive strength of 700 psi (4 800 kPa) or greater. The critical factor in the performance of both LFA and cement-aggregate mixtures is achieving density of the material in place. In this respect, the advantage lies with the LFA because there is usually a longer time period between mixing and setting during which density can be achieved. With cement-aggregate mixtures, all densification must be accomplished within two hours, and preferably within a much shorter time; but with most LFA mixes, delays of several hours do not affect the compactability of the material unless it is allowed to dry to below optimum moisture content. This is not a general rule, however, because some fly ashes are so reactive that special handling may be required. Each particular combination of materials used should be checked for reactivity and rate of reactivity before decisions are made on the allowable lead time prior to completion of the compaction process.

To thoroughly understand the potential and the limitations for LFA materials, it is necessary to review the performance and distress of several heavy-duty pavements in service. Some potential problems and causes for these problems can be illustrated through a review of the performance and distress of several highway pavements. With proper design and construction control, pavements with LFA materials have the potential to provide a high level of performance; with improper design and control during construction, the potential of this material may not be realized.

The base and subbase for runways, taxiways, and apron areas at Newark Airport were constructed with LFA materials. Figure 43 shows some typical cross sections of the pavement sections for various airport facilities. Mix proportions and properties of the cured mixes used in the construction are given in Table 12. Quality control during the construction was provided by the Port Authority of New

TABLE 11

RELATIVE STRUCTURAL COEFFICIENTS USED BY SEVERAL STATES FOR LFA BASE MATERIALS

STATE	COEFFICIENT, a_2
Illinois	0.28
Michigan	1:1 with black base
Ohio	0.25-0.30
Pennsylvania	0.30

TABLE 12

APPROXIMATE MIX PROPORTIONS USED IN THE NEWARK AIRPORT PAVEMENT

MIX DESIGNATION	INGREDIENTS, % LIME	CEMENT	FLY ASH	SAND	STONE
A	3.6	0.9	12	53.5	30
B	3.2	0.8	14	82	0
C	2.8	0.7	14	82.5	0

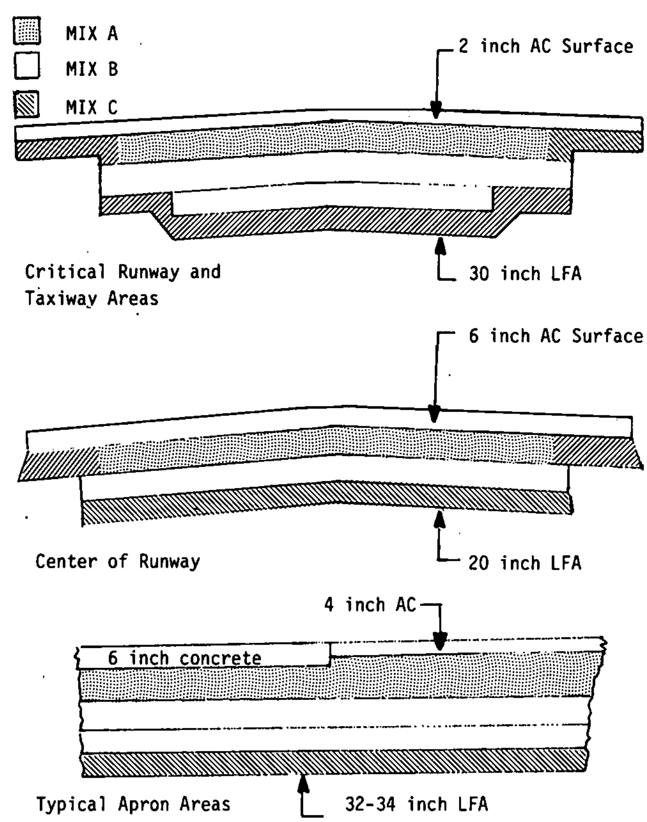


Figure 43. Typical cross sections used at Newark Airport.

York and New Jersey and is generally higher than achieved in normal highway pavement construction.

The projected compressive strength of these materials varies from around 1,000 psi (6 900 kPa) after one year to in excess of 2,000 psi (14 000 kPa) after approximately three years. Cores taken from these pavements after one to two years in place are well in excess of the 2,000 psi projected ultimate strengths.

At the time of the latest evaluation,* some of the runways and taxiways had been in service for slightly over five years. At that time, the only significant distress in the pavements was some shrinkage cracks in the shoulder area where an insufficient number of contraction joints was provided, and one crack because of subgrade settlement in an area where an old drainage ditch existed prior to the construction.

The airport facility engineers for several major airlines using Newark Airport indicate that they are satisfied with the performance of the pavements. They generally agree that despite the aforementioned minor cracking distress, the pavements at Newark Airport are smooth with a high serviceability rating and show excellent performance trends.

It should be noted that contraction joints as illustrated in Figure 44 were placed at 150- to 200-ft (46- to 61-m) intervals on the runways and taxiways. In some instances, the asphaltic concrete (AC) surface was partially sawed through, directly over the joints in the LFA base, and in other instances no saw cuts were made and the AC was allowed to crack in a random manner. The sawed joints were sealed as soon as possible, whereas the random cracks were not sealed until later when they had fully developed. These random cracks have caused the maintenance engineers some concern, but apparently have not caused any structural damage or loss of performance in these pavements.

Heavy-duty LFA pavements have also been recently completed by the Portland Port Authority of Portland, Oregon. Figure 45 is a cross section of two of the pavements constructed by the Portland Port Authority.

The marine terminal pavement has been in service for over a year with no significant problems. This pavement covers over 40 acres (162 000 m^2) and is used as temporary storage for containers during transfer from ship to land transportation. These containers, which weigh up to 40 tons (36 000 kg) each, are stacked up to three containers high with only a few inches between stacks. A transtainer, as shown in Figure 46, is used to transport these containers from one location to another. Normal wheel load for the loaded transtainer is 50,000 lb (220 000 N), but under unusual loading and wind conditions, individual wheel loads can go as high as 100,000 lb (440 000 N). Composition of the LFA materials used in the Portland Marine Terminal is given in Table 13.

Projected compressive strengths of these materials vary from approximately 800 psi (5 500 kPa) after three months, to in excess of 2,000 psi (14 000 kPa) after three years

* Private communication with airline facility engineers, Newark Airport personnel, and personal observations by Dr. E. J. Barenberg.

TABLE 13

COMPOSITION OF LIME-FLY ASH MIXES USED IN PORTLAND MARINE TERMINAL

COMPONENT	PROPORTION (%) MIX A	MIX B	MIX C
Hydrated lime, ASTM Type N	3.3	3.0	3.0
Portland cement, ASTM Type I	1.1	1.0	1.0
Fly ash	6.6	6.0	6.0
Inorganic silt	8.0	10.0	10.0
Aggregate	25.0	10.0	—
In-place fill sand	56.0	70.0	80.0

(*48*). Cores taken from the pavements after less than six months of service show that the strengths exceed the projected strengths of the materials.

After more than one year of service, the pavements at the marine terminal showed only isolated cracks in two locations. These cracks are believed to be because of nonuniform settlement of the hydraulic fill placed over the entire site before the pavements were placed. It is significant to note that Portland has a mild climate with only moderate ranges in moisture and temperature. No shrinkage cracks were observed in these pavements. Figure 47 shows the condition of the runway and taxiway extensions at the Portland Airport after almost one year of service.

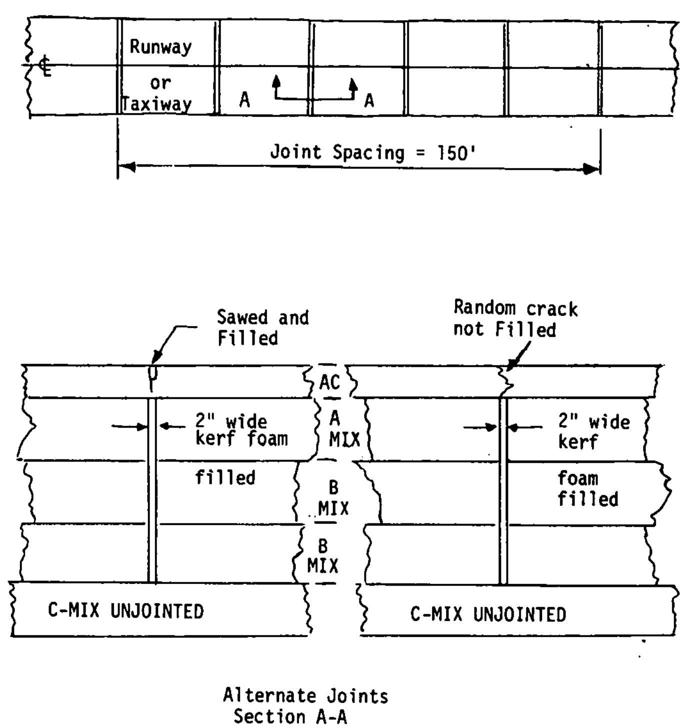


Figure 44. Jointing system used at Newark Airport.

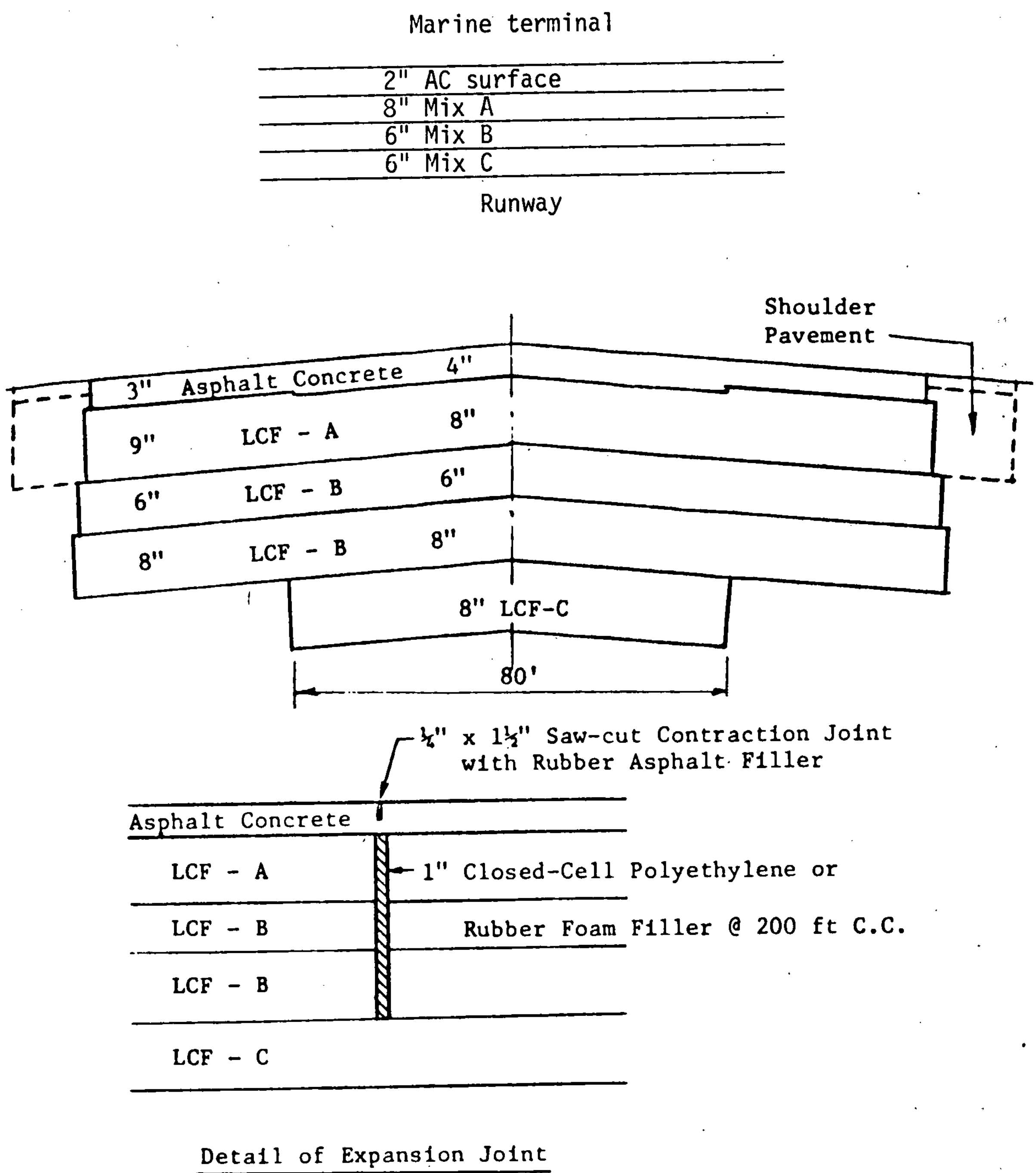


Figure 45. Typical cross sections used in the LFA heavy-duty pavements at Portland, Ore., for the marine terminal and airport.

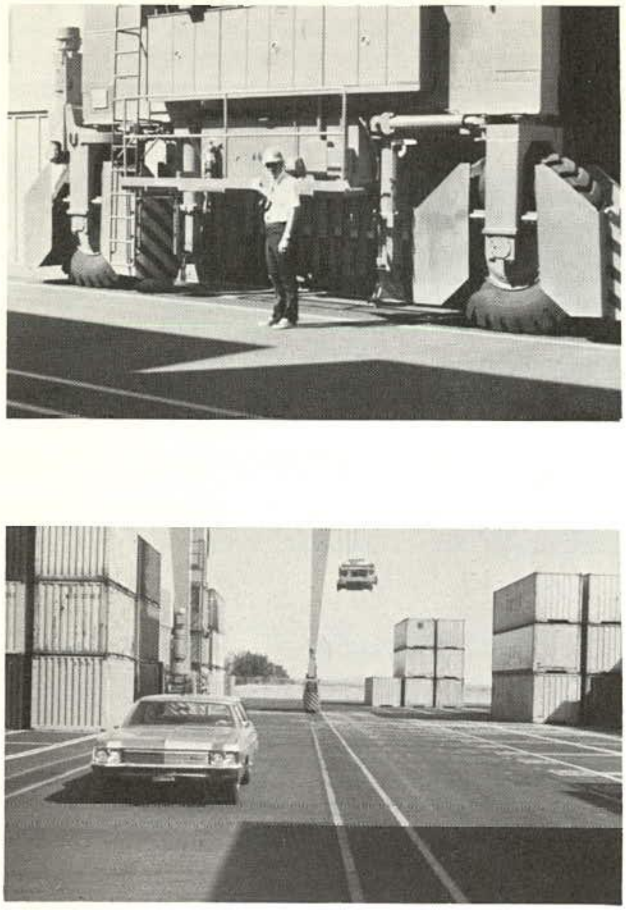


Figure 46. *Transtainer and storage bins at the Portland, Ore., Marine Terminal with LFA pavements.*

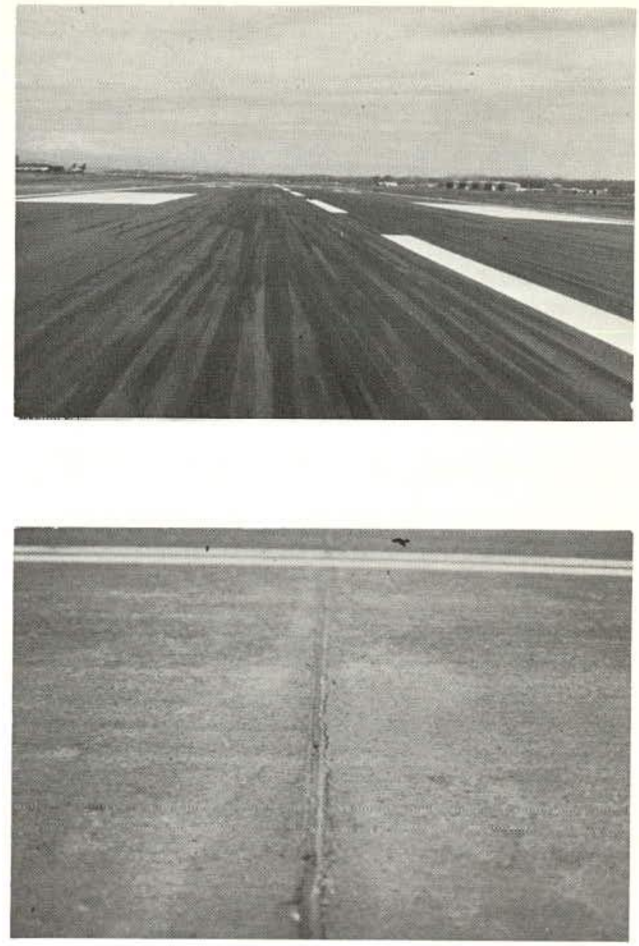


Figure 47. *Pavement at Portland International Airport constructed with LFA materials showing (a) a pavement overview and (b) a traverse joint.*

Performance of pavements with LFA materials, as well as other paving materials, in pavements in Lake County, Illinois, were evaluated by Hazarika (*44*). The primary thrust of this investigation is to make an economic analysis of the life-cycle costs of different pavement systems and to evaluate ways of decreasing life-cycle costs through elimination of the distress that causes the loss in serviceability.

In the course of his investigation, Hazarika (*44*) opened several pavements with LFA materials to determine the cause of the distress. Figures 48, 49, and 50 show the type and extent of distress at three of these locations. A few observations on the type of distress at these locations are instructive.

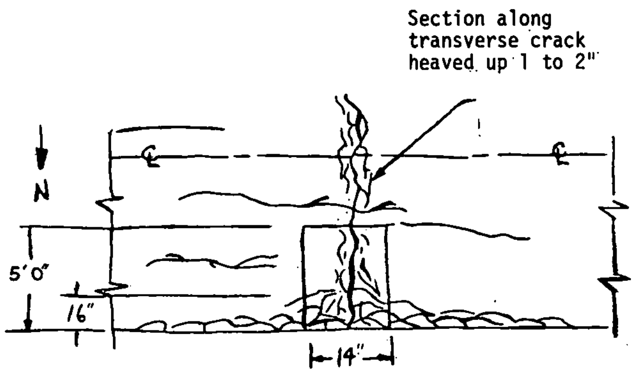
At the Everett Road location (Fig. 48), the distress consisted of alligator cracking along the pavement edge and along either side of a transverse crack. The material along the transverse crack had heaved or tented to a significant degree.

A section of the pavement surface was removed along the transverse crack as shown in Figure 48. Density readings were taken with a nuclear gauge near the pavement edge and at approximately the center of the wheelpath [25 ft (7.6 m) from edge]. The measured densities are shown in Figure 48. Note that there is a significant difference between the density near the pavement edge and that in the wheelpath.

The distress on Winchester Road (Fig. 49) is similar to that on Everett Road except that there is no heave and no alligator cracking along the transverse crack. Density readings for this investigation were taken at three locations: near the pavement edge, in the wheelpath, and between the wheelpaths. Cores were also taken at these locations where the condition of the LFA material permitted. Core strength and density readings are shown in Figure 49.

As with the Everett Road analysis, there is a significant difference between the density of the LFA material near

Sketch of Test Site with Distress:



Plan View

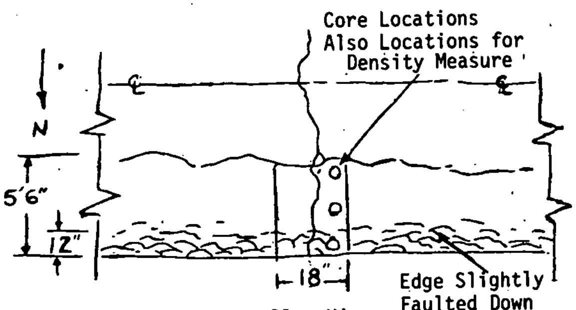
Location	EVERETT ROAD
Year Constructed	1962
Age When Investigated	10 years
ADT (average daily traffic)	
Initial	300
Intermediate	450
Final	600

Location	Moisture Content (%)	Dry Density (pcf)
Pvmt Edge	8.4	113.8
Wheelpath	7.3	134.4
Lane Ctr.	---	---

Notes: pavement badly heaved across width along transverse crack - possibly due to expansion of AC layer.

Figure 48. Distress pattern on Everett Road.

Sketch of Test Site with Distress:



Plan View

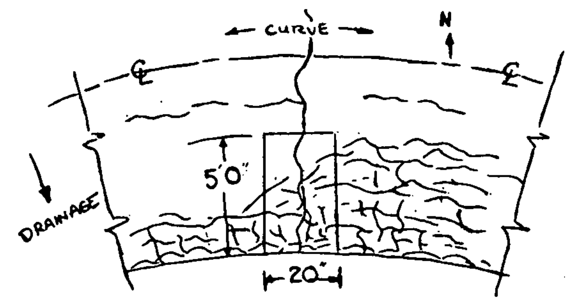
Location	WINCHESTER ROAD
Year Constructed	1963
Age When Investigated	9 years
ADT (average daily traffic)	
Initial	400
Intermediate	2300
Final	1080

Location	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psi)
Pvmt Edge	7.1	123.6	
Wheelpath		127.9	< 2000
Lane Ctr.	6.8	136.4	4500

Notes: alligator cracking along both pavement edges, north edge slightly faulted down, LFA base very wet.

Figure 49. Distress pattern on Winchester Road.

Sketch of Test Site with Distress:



Location	YORKHOUSE ROAD
Year Constructed	1963
Age When Investigated	9 years
ADT (average daily traffic)	
Initial	400
Intermediate	800
Final	650

Location	Moisture Content (%)	Dry Density (pcf)
Pvmt Edge	8.2	111.0
Wheelpath	7.7	131.9
Lane Ctr.	7.6	131.9

Notes: Curve superelevated - pavement badly cracked along lower edge. Upper lane very little cracking - at time of opening test section, there was a very high percentage of semi-truck traffic.

Figure 50. Distress pattern on Yorkhouse Road.

the pavement edge and that found further in from the pavement edge. This difference in density is reflected in the strength of the cores removed from the pavement. Near the edge where density was very low, no cores could be taken because the LFA was badly disintegrated. At the locations in and between wheelpaths, the core strengths reflect the measured densities of the material at these locations, with the higher strength cores also having the higher densities.

The investigation on Yorkhouse Road (Fig. 50) was on a superelevated curve. At this location, all of the distress was along the lower edge of the pavement and consisted of alligator cracking that extended to approximately the midpoint of the lower lane. Density readings for the three locations are shown in Figure 50.

It is noted that in all three locations, the density along the edge was much less than in the wheelpath or between the wheelpaths. Data on the densities from these three locations (plus four additional locations) are summarized in Table 14. These data clearly show the effect of inadequate compaction of the LFA material along the edge of these pavements.

The types of distress observed during this investigation suggest that three factors are involved in the pavement distress. The distress is due primarily to deterioration of the LFA material. This deterioration is, in turn, the direct result of excess moisture in the base material and inadequate density of the LFA, especially along the pavement edge. As illustrated by the data from Winchester Road, when adequate density is achieved, LFA materials develop and maintain a high level of strength. Conversely, it can be shown that reductions in the compacted density result in significantly lower strength and sharply reduced durability for these materials.

The pavement edge is usually inadequately compacted and, therefore, is of low density because of (*a*) the lack of side restraint during rolling or (*b*) the lack of adequate rolling along the edges. The different methods of compaction and ways to achieve uniform densities to the pavement edge are discussed further in Chapter Six, Construction Procedures.

Causes for the difference in densities of the LFA material between wheelpaths compared to the material in the wheelpath are less obvious. Speculation on this phenomenon leads to two possible causes: (*a*) the pattern of the rollers and other compaction equipment during construction, and (*b*) the rolling of construction equipment on the layers of LFA material prior to its setting up. The patterns of the compaction equipment used in the construction of the pavements investigated are not known; thus no further conclusions can be reached concerning this potential cause. Construction traffic on this type of construction does not generally follow the final wheelpaths of the pavement. It is likely that such traffic produces added densification in areas other than the final pavement wheelpaths.

Use of the compacted LFA layers by the construction equipment is normal for the type of construction procedures used with LFA materials. Although this practice has been successful and economical, it does involve certain inherent dangers. Two types of distress have been observed from these operations. One type of distress occurs when the LFA material becomes saturated before setting up. If the material does not have adequate internal stability, the unset LFA begins to shove and rut like a poorly graded aggregate. This distress is immediately observable and can be corrected by reshaping and recompacting the pavement layer and allowing it to cure before additional use is permitted. A more subtle and perhaps more harmful type of distress can occur when heavy construction equipment runs on LFA materials that are partially set up. This damage is particularly harmful because it occurs after the initial set, and even with the autogenous healing, the damage significantly reduces the ultimate strength of the material. This type of damage is also more harmful because it is usually not apparent on the layer surface during construction, especially if the LFA layer is sealed and surfaced immediately. This second type of damage occurs when the material is placed over soft supporting soils and heavily loaded trucks, or other heavy pieces of construction equipment, are permitted on the partially set up layers.

The discussion on performance of pavements with LFA materials has been directed primarily at their use as base and subbase materials. These materials have also been used as the base layer in pavement shoulders for both rigid and flexible pavements. The performance of LFA materials as the base layer in shoulders is somewhat spotty; some installations show good performance whereas in other installations the performance is far less favorable.

A review of the installations in which the LFA material did not give good performance as a shoulder material reveals that the same factors that cause poor performance in the pavement section proper also affect its performance in the shoulder. The biggest single cause for distress in the shoulder is durability of the LFA material. This, in turn, is greatly affected by such factors as improper compaction adjacent to the pavement edge and segregation of the LFA adjacent to the pavement drainage edge. This lack of durability near the pavement edge produces a surface distress in the shoulder near the pavement edge, which in turn allows excessive water to infiltrate the shoulder near the pavement edge. This water, often saturated with salt, causes an ad-

TABLE 14

DENSITY OF LFA MATERIAL AT THREE PAVEMENT LOCATIONS

LOCATION OF TEST	DRY DENSITY (PCF) AVE.	DRY DENSITY (PCF) STD. DEV.	DENSITY AS PERCENT OF WHEELPATH DENSITY
Near pavement edge	122.42	9.1	91.0
Centerline of wheelpath	134.8	6.6	100.0
Between wheelpaths	130.0	7.5	96.0

vanced rate of disintegration in the LFA shoulder base layer (*46, 55, 56*).

Although the potential for distress appears greater when using LFA materials for shoulder base construction, the successful use of this material in some locations indicates that it has the potential for this type of application. During shoulder construction, high-quality construction techniques should be used to ensure uniform density throughout (especially near the pavement edge) and to prevent segregation of the LFA material due to overhandling. It is also important to design shoulders in a manner that permits adequate drainage of infiltrating water.

The performance of pavements with LFA materials is affected by the same factors that affect the performance of pavements with other stabilized materials. Thus, the primary factors affecting the performance of pavements with LFA materials are durability, proper thickness design consistent with the expected loading, and material strength development. Pavements with LFA materials are no more sensitive to these factors or to construction anomalies than are pavements with other types of paving materials. It can be concluded, therefore, that with proper attention to formulation of these mixes, design of appropriate pavement sections, and proper control during construction, LFA materials have the potential to provide pavements with excellent performance records.

CHAPTER FIVE

SELECTION OF MIXTURE PROPORTIONS

Proportions of lime, fly ash, and aggregate (soil) should be selected so that a mixture can be used for a designated purpose. In most instances, quality requirements for the mixture components are specified as indicated in Chapter Two of this synthesis.

The mixture proportions selected must (*a*) possess adequate strength and durability for the designated use (generally as a base or subbase layer), (*b*) be easily placed and compacted, and (*c*) be economical. For a given set of materials, several proportions can provide lime-fly ash–aggregate (LFA) mixtures of satisfactory quality.

APPROACH TO MIXTURE PROPORTIONING

For a given set of materials (lime, fly ash, and aggregate), the factors that can be varied in the mixture selection process are the amount of lime plus fly ash and the ratio of lime to fly ash. The blending of materials creates more suitable gradations that can be stabilized with lime plus fly ash to produce superior quality LFA mixtures (compared to the stabilizing of unblended materials) (*58*). If more suitable and economical LFA mixtures can be obtained by blending procedures, various blended gradations may be appropriately considered in the mixture selection process.

Lime-Plus-Fly Ash Content

It has been found (*57, 58*) that the quality of cementitious stabilized mixtures (strength and durability) is related to the quality of the matrix material. It is possible to achieve a high compacted density in the matrix material, thereby improving its strength and durability, only if there is sufficient matrix material to "float" the coarse aggregate. Lime-plus-fly ash contents in excess of the amount required for maximum dry density produce systems in which the coarse aggregate [plus No. 4 (>4.75 mm)] is floating in a matrix of lime plus fly ash plus aggregate fines.

Many agencies use compacted dry density (ASTM C 593) to determine the approximate amount of lime plus fly ash to incorporate into the mixture. Moisture-density curves are developed for several mixtures with varying lime-plus-fly ash contents. The lime-to-fly ash ratio is constant at some predetermined value. A plot is then prepared showing the relationship between maximum dry density and lime-plus-fly ash content. The peak of the density–lime-plus-fly ash curve corresponds to the condition in which the voids in the aggregate are filled with lime plus fly ash plus aggregate fines [minus No. 4 (<4.75 mm) fraction].

It is desirable and conservative to have lime-plus-fly ash contents in excess of the amount required to develop maximum dry density. LFA mixture quality may drop off rapidly when lime-plus-fly ash contents are less than the amount required for maximum dry density.

Figure 51 shows a typical maximum dry density–lime-plus-fly ash content response for an A-3(0) material. Materials that are well graded in the range of ±1-in. (25-mm) maximum size through the No. 200 (75-μm) sieve do not require large quantities of lime plus fly ash in order to achieve maximum dry density. Figure 52 indicates that for a well-graded crushed stone and a pit-run gravel, lime-plus-fly ash contents in excess of 10 percent decrease maximum dry density. It is apparent that the original gradations of the material to be stabilized significantly influence the lime-

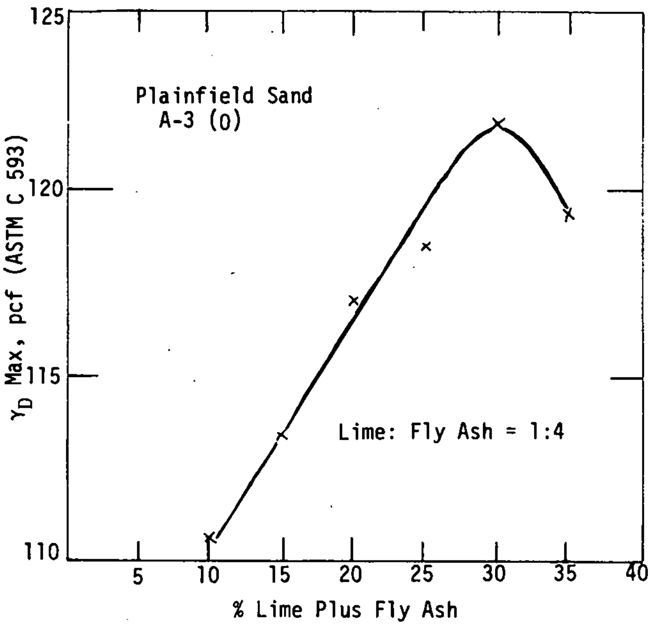


Figure 51. Effect of lime-plus-fly ash content on maximum compacted dry density (Plainfield sand).

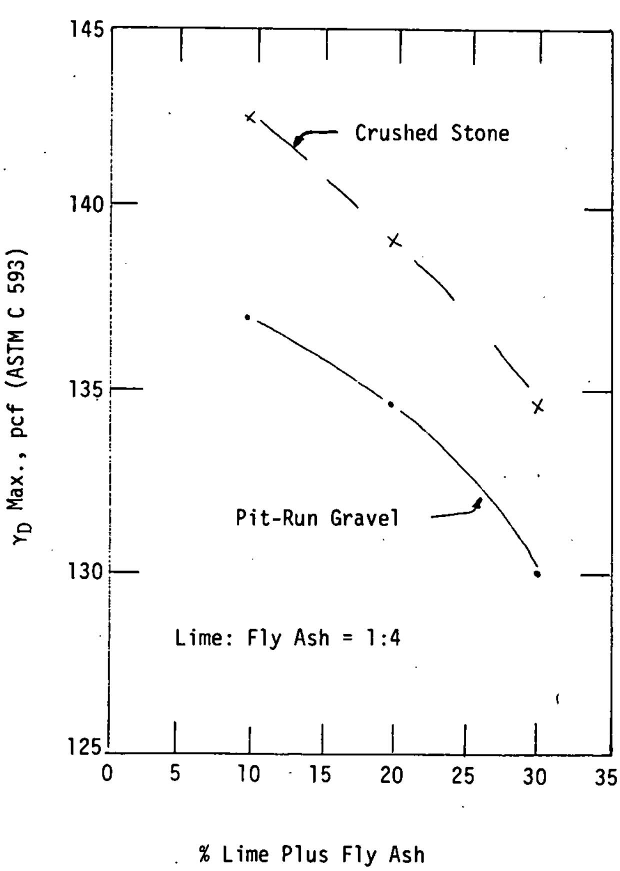


Figure 52. Effect of lime-plus-fly ash content on maximum compacted dry density (crushed stone and pit-run gravel).

plus-fly ash content requirement for developing maximum dry density. Poorly graded materials with smaller maximum sizes require substantially more lime plus fly ash than better graded materials with larger maximum size.

Lime-to-Fly Ash Ratio

The proper lime-to-fly ash ratio must be determined based on laboratory data. This ratio does not necessarily remain constant for a given lime and fly ash. Characteristics of the material to be stabilized, such as fines content and plasticity, also affect the lime requirement. Increased fines content and plasticity require more lime for the same fly ash content.

Once an approximate lime-plus-fly ash content has been established, a series of mixtures with constant lime-plus-fly ash content and varying lime-to-fly ash ratio can be prepared. Mixture compressive strength following a designated curing period [perhaps 7 days at 100 F (38 C) for the ASTM C 593 procedure] can be used to determine the appropriate lime-to-fly ash ratio.

Mixture Designation

After the approximate lime-plus-fly ash content and lime-to-fly ash ratio have been determined, it is necessary to further evaluate mixture quality. Such factors as cured compressive strength, durability, and rate of strength development (time-temperature-strength relation) may be of interest. If the preliminary mixture is not satisfactory, other mixtures with different lime-plus-fly ash contents or lime-to-fly ash ratios should be evaluated.

After the lime and fly ash content requirements have been established for a mixture based on the laboratory testing data, the final lime content is designated. A 0.5 percent increase in lime content is generally sufficient to offset the construction variability (lime content variation) associated with typical mixture production procedures.

In some instances, a less structured approach to selection of mixture proportions may be used. Ranges of lime-plus-fly ash and lime-to-fly ash ratio mixtures are considered and mixture proportions established based on an analysis of strength and durability data. This procedure is usually less efficient.

LABORATORY TEST PROCEDURES

Several different laboratory test procedures are used in developing lime-fly ash–aggregate (LFA) mixture proportions and evaluating LFA quality. Some of the more widely used procedures are considered in the following sections.

Moisture-Density Test Procedures

Moisture-density tests are conducted in the usual manner with the exception that compactive effort varies. Table 15 gives the compactive efforts specified by various procedures and agencies. The Proctor mold, 4.0 in. (100 mm) in diameter by 4.6 in. (117 mm) in length, is used in all of the procedures.

It is important to note that compacted density has a very substantial effect on the cured strength and durability of LFA mixtures. Quality criteria developed for one type of compaction should not be indiscriminately applied to LFA mixtures prepared in accordance with a different compaction procedure.

Compressive Strength

Compressive strengths of cured LFA mixtures are used to evaluate mixture quality and to characterize engineering behavior. Factors of interest in compressive strength testing are sample size, compaction procedure, and curing conditions (time-temperature).

Standard-size specimens [4.0 in. (100 mm) in diameter by 4.6 in. (117 mm) in length] are most widely used. Aggregate particles larger than 0.75 in. (19 mm) are scalped and discarded. For fine-grained soils and sandy materials, 2-in.-diameter (50-mm) specimens have also been used. The University of Illinois procedure (*36*) uses a 2-in.-diameter, 4-in.-long specimen. The Iowa State procedure (*59*) uses a 2-in.-diameter, 2-in.-long specimen. It is important to note that the length-to-diameter ratios vary for the different procedures.

Direct comparison of the strength data developed from specimens of different sizes is difficult. The use of a correction factor based on length-to-diameter ratio (for example per ASTM C 42) should be considered in making such comparisons.

Standard-size compressive strength specimens are prepared using the same compactive effort specified in the moisture-density testing procedure and thus vary, depending on agency requirements. In the University of Illinois procedure (*36*), the mixture is compacted in three layers using a 4-lb (1.8-kg) hammer (full-face compactor) with a 12-in. (300-mm) drop. The number of blows per layer is varied to produce the desired compacted density (normally per ASTM C 593).

General correlations (*59*) for the Iowa State specimen indicate that for one lift, double-ended compaction, 10 blows (5 on each end) of a 5-lb (2.3-kg) hammer with a 12-in. (300-mm) drop approximates standard Proctor density, and 20 blows (10 on each end) of a 10-lb (4.5-kg) hammer with a 12-in. drop approximates modified Proctor density.

It is essential to maintain closely controlled curing conditions for LFA mixtures. Both time and temperature significantly influence LFA mixture strength development. Curing conditions should be indicated when strength data are presented.

Many agencies (Illinois DOT, Pennsylvania DOT, FAA) use ASTM C 593 curing conditions [7 days at 100 F (38 C) in a sealed container]. In several of the Iowa State studies, curing conditions were 70 to 74 F (21 to 23 C) for various time periods ranging from 7 to 28 days.

It is possible to predict the strength development of a lime-fly ash–aggregate (LFA) mixture under field curing conditions. Extensive field temperature studies (preferably with a theoretical heat flow model), such as those described by Thompson and Dempsey (*60*) and MacMurdo and Barenberg (*20*), are required to characterize field curing conditions. The Pennsylvania Department of Transportation (*61*) has used extensive field temperature data to develop a procedure for predicting field curing temperatures from air temperature data.

TABLE 15

MOISTURE-DENSITY TEST PROCEDURES

Agency	Procedure	Compactive Effort *
——	ASTM C 593	10/18/3/25
Federal Aviation Administration	FAA T611	10/18/5/25
Illinois DOT	——	10/18/3/25
Ohio DOT	ASTM C 593	10/18/3/25
Pennsylvania DOT	PTM 106	5.5/12/3/25

*hammer weight, lbs / hammer drop, inches / number of layers / blows per layer

In order to correlate field curing to LFA strength development, it is necessary to establish a strength–degree-day relationship for the mixture. Temperature ranges used to establish the strength–degree-day relationship should be similar to those expected in the field environment. The importance of the curing temperature on strength development is obvious from Figure 29.

Field curing conditions are quite variable. This variability must be considered in developing cure time recommendations for field conditions. Industry-sponsored studies at the University of Illinois indicate that for all curing conditions in Illinois (Chicago, Springfield), the standard deviation for accumulated curing degree days (based on an analysis period from October 15 to November 30) is approximately 75 degree days, 40 F base (42 degree days, 4 C base).

Durability Tests

Three procedures have been predominantly used for evaluating the freeze-thaw durability of LFA mixtures.

The freeze-thaw brushing procedure, formerly included in ASTM C 593, is basically modeled after the soil-cement procedure (AASHTO T 136). Thompson and Dempsey (*60*) indicate that the temperature conditions used in the ASTM C 593 procedure are unrealistic and do not simulate field conditions. The "weight loss" factor determined in the ASTM procedure has no physical significance in terms of basic engineering properties (strength, stiffness, etc.).

Dempsey and Thompson (*36*) developed automatic

TABLE 16

QUALITY CRITERIA

Agency	Minimum Compressive Strength psi	(kPa)	Maximum Weight Loss, % *
ASTM C 593	400	(2800)	14
Illinois DOT	400	(2800)	10
Ohio DOT	400	(2800)	10
FAA	400	(2800)	14
Pennsylvania DOT	Not specified		14

*12 cycles of freeze-thaw

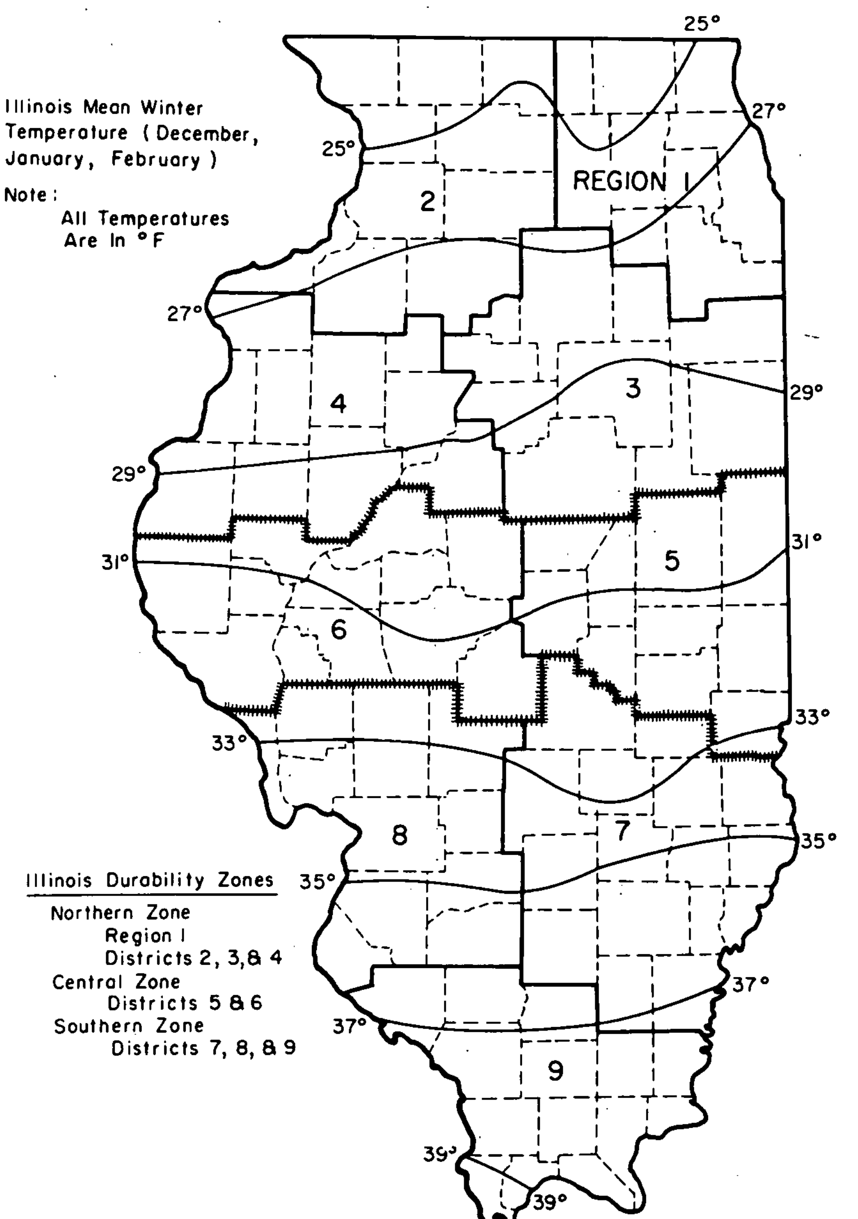


Figure 53. Mean winter temperature data and freeze-thaw durability zones for Illinois.

freeze-thaw testing equipment that accurately simulates field conditions. Compressive strength after freeze-thaw cycling (5 to 10 cycles) is used to characterize LFA mixture durability.

The vacuum saturation test procedure proposed by Dempsey and Thompson (*37*) is a rapid technique (approximately one hour). The vacuum saturation procedure produces an excellent correlation (Figs. 16 and 17) between the compressive strengths of vacuum saturation specimens and freeze-thaw (Dempsey-Thompson technique) specimens. ASTM C 593 has been revised to use the vacuum saturation procedure for durability evaluation purposes.

QUALITY CRITERIA

The acceptability of LFA mixtures is determined by applying selected quality criteria. Most mixture proportion procedures include both strength and durability criteria.

Minimum cured compressive strength and maximum weight loss criteria are specified by ASTM C 593-69, Illinois DOT, and the Federal Aviation Administration, as given in Table 16. The Pennsylvania Department of Transportation has a durability requirement, but no strength criteria. The vacuum saturation strength requirement recently incorporated into ASTM C 593 specifies a minimum vacuum saturation strength of 400 psi (2800 kPa). The Illinois DOT is also currently considering a vacuum saturation strength requirement.

Thompson and Dempsey (*32*) advocate the use of the residual strength approach for establishing freeze-thaw durability criteria. The approach emphasizes that a sliding scale of quality should be specified depending on the field service conditions anticipated for the mixture. For example, little freeze-thaw action occurs in an LFA mixture layer course in southern Illinois, but many freeze-thaw cycles occur in a base course constructed in Chicago. In fact, it has been proposed that Illinois be divided into three separate zones, as shown in Figure 53, for the purpose of establishing stabilized mixture durability criteria.

CHAPTER SIX

CONSTRUCTION PROCEDURES

Among the advantages for use of LFA mixtures in pavement construction are the ease of construction and the fact that conventional construction equipment can be used to mix and place the materials. The major requirements for the effective use of LFA materials are that the ingredients be thoroughly mixed, that the mixture be spread uniformly to the proper thickness with a minimum of manipulation, and that it be compacted to a high relative density. This can be accomplished with construction equipment normally found on a pavement construction site (i.e., spreader box, grader, rollers, water truck, etc.). Although the required construction procedures are well known to pavement contractors, it is emphasized that poor construction techniques can result in reduced pavement performance and a final product with low reliability.

The blending of lime-fly ash–aggregate (LFA) materials can be done either in place on the roadbed using rotary mixers and similar equipment, or in a central plant. Plant blending is recommended where economically feasible because of the greater control over the quantity of ingredients added and the production of a more uniform mix.

CENTRAL PLANT OPERATIONS

Blending of Components

Figure 54 shows a schematic layout of a typical plant used in the blending of LFA mixtures. The main components of these plants are:

1. Aggregate hopper with belt feeders.
2. Fly ash hopper with a belt feeder and controls.
3. Lime storage tank with an intermediate feed hopper and a feed control device.
4. Water storage tank with a calibrated pump.
5. Continuous or batch-type pugmill for blending the components.
6. Surge hopper for temporary storage of blended LFA.

Figures 55 and 56 show various plants set up to produce LFA materials. These plants vary in capacity from small portable units, capable of being transported over the highway system and set up without special equipment, to large permanent plants. The portable plants are designed to pro-

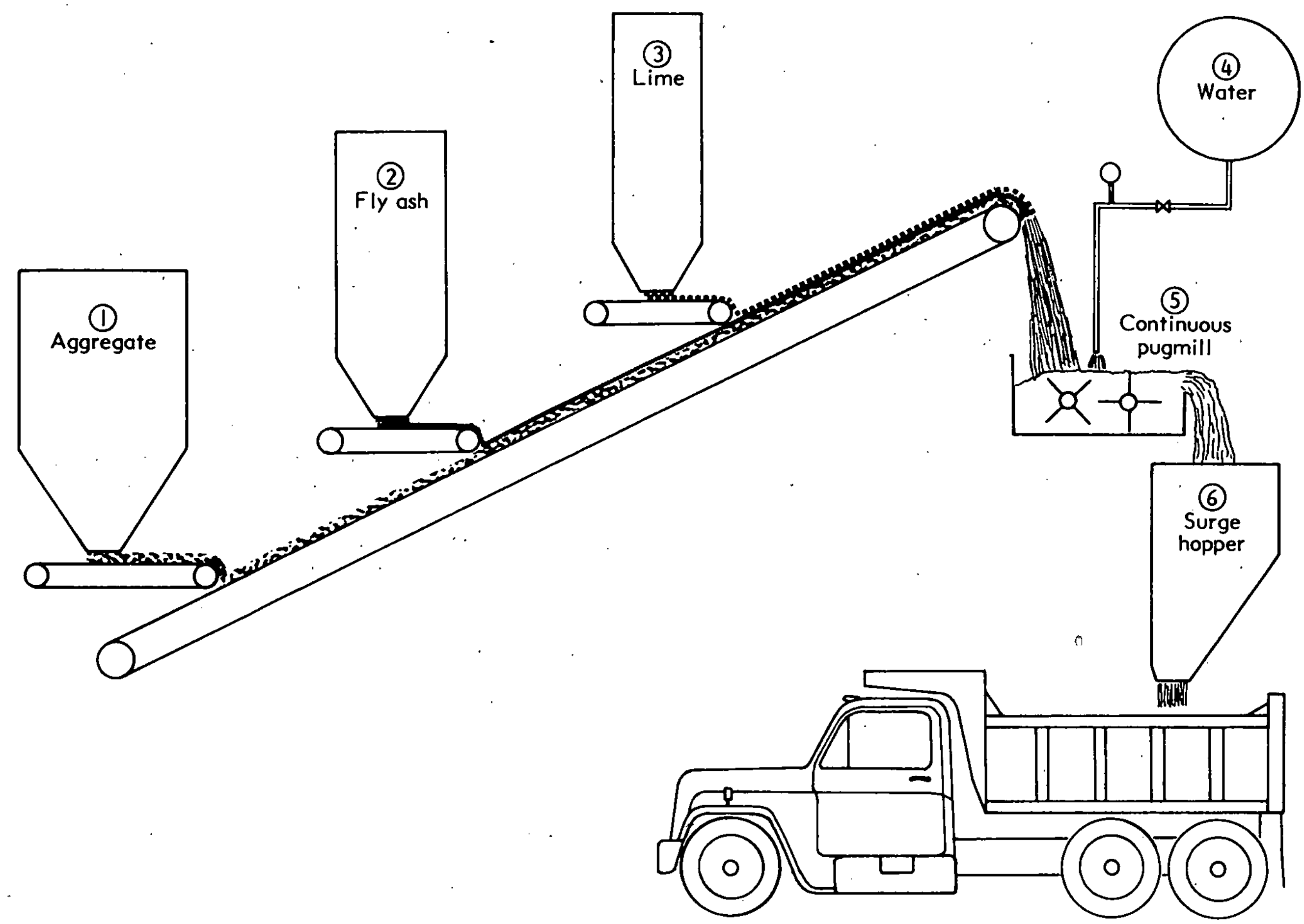


Figure 54. Schematic diagram of typical plant layout for LFA mixes.

duce about 100 tons (90 000 kg) of LFA mix per hour whereas the larger, permanent plants are capable of producing up to 600 tons (540 000 kg) per hour.

In addition to the equipment described, the operator needs front-end loaders, tractors, and similar equipment to charge the aggregate and fly ash hoppers and to maintain an orderly plant site and plant operation.

Lime is stored in silo tanks as shown in the figures. It is delivered in pneumatic transports that are also used to charge the silos with the lime as shown in Figure 57. The storage capacity for lime needed at each plant depends on the plant production capacity, the reliability of the delivery schedules, and time lag between ordering and delivery of the lime.

Fly ash is normally stored in open stockpiles as shown in Figure 58. Fly ash stored in this manner must be conditioned with sufficient water to prevent dusting (usually 15 to 20 percent residual moisture content). During dry weather, the stockpile surfaces must be kept moist, or the stockpile must be covered to prevent surface dusting. The conditioned fly ash is charged into the feeder hopper with a front-end loader or other arrangement. Some fly ashes set up in the stockpile. These must be recrushed before use in LFA mixes. Hammer mills and roll mills have been used effectively to crush the set-up fly ash.

The pugmill mixing plants described in the preceding chapters are normally the type used to blend LFA materials. However, central-mix concrete plants have been used successfully for this purpose. Adequate mixing time must be provided in the central-mix plant to ensure thorough blending of the constituents.

Hauling

LFA mixtures blended in a central plant can be hauled to the road site in conventional, open-bed dump trucks. If haul distances are long, or if drying of the material enroute poses a problem, provisions should be made to cover the trucks with tarpaulins or other suitable cover to prevent loss of moisture or the scattering of dust along haul routes. Sufficient trucks should be made available so that all equipment, such as the mixing plant spreaders, rollers, etc., can operate at a steady, continuous pace rather than on a stop-and-go basis.

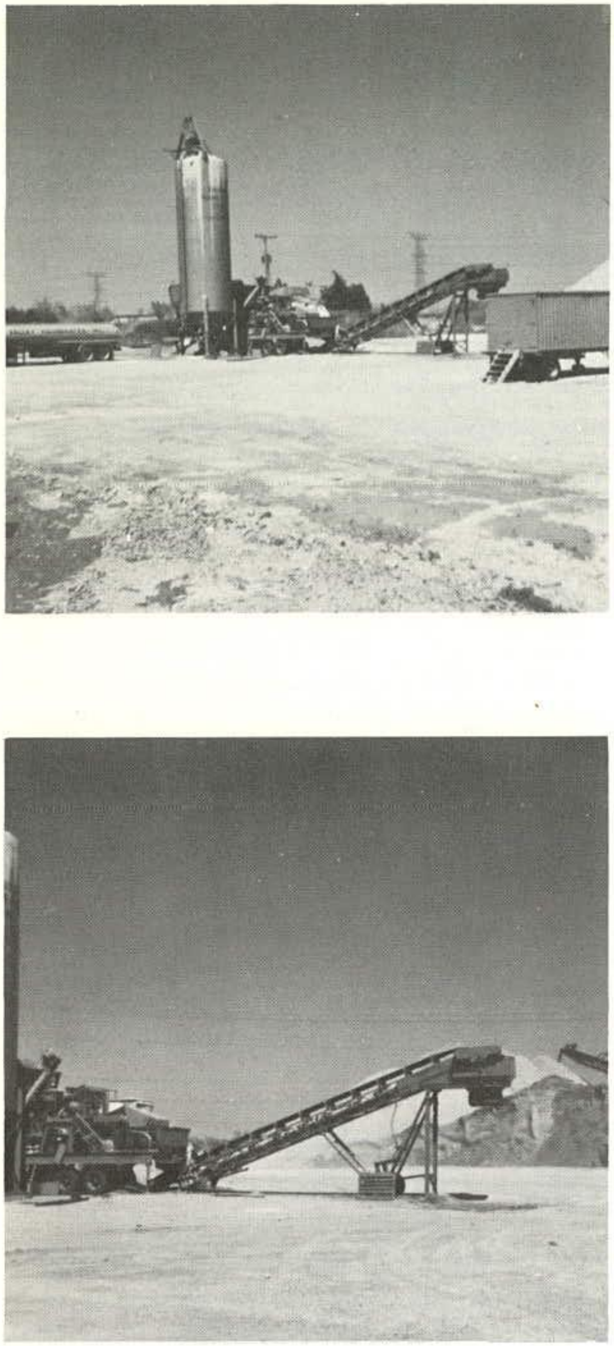


Figure 55. Portable plant used to produce LFA materials.

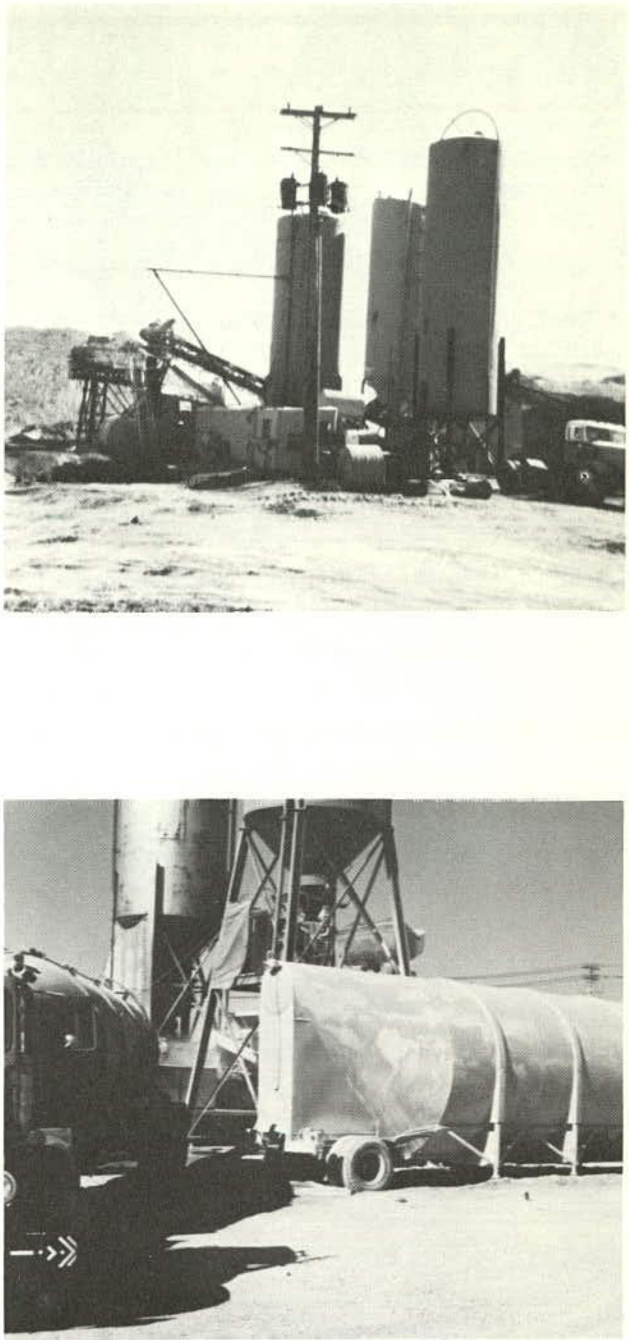


Figure 56. Permanent plant setup used to produce LFA materials.

Spreading

Plant-blended lime-fly ash–aggregate mixtures should be delivered to the prepared subgrade and spread as uniformly as possible with a minimum of manipulation. Figures 59 through 62 show various types of operations and equipment for spreading the blended LFA mixture. Spreader boxes, asphalt laydown machines, or other equipment with automated grade control is recommended because such mechanized equipment generally gives better uniformity of depth with a minimum of manipulation and segregation. The materials can be placed in windrows from the trucks and spread with graders, but this method is not recommended. With the windrow-type of operation, the material can be overmanipulated causing drying and segregation.

Figure 57. Lime being delivered to a plant by a truck having a pneumatic discharge system.

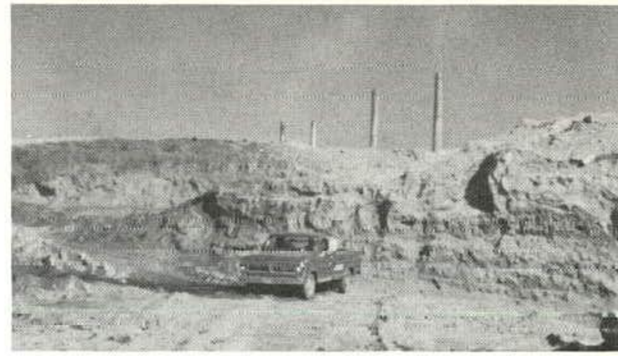


Figure 58. Fly ash stored in open stockpiles near a utility plant.

Layers of LFA mix are normally spread to a thickness between 15 and 30 percent greater than the desired final thickness to allow for compaction. The amount of excess thickness is a function of the aggregate type and source as well as the method of spreading. Some types of spreading operations provide a degree of initial consolidation; therefore, some experimentation is necessary to determine the proper spread thickness for each operation.

The maximum recommended thickness for a single layer of LFA after compaction is 8 to 10 in. (200 to 250 mm), although some agencies specify a lesser maximum thickness. If thicknesses of LFA layers greater than the specified maximum are needed to develop an adequate pavement system, the material should be spread and compacted in lifts. If the material is placed in lifts, the time between lifts

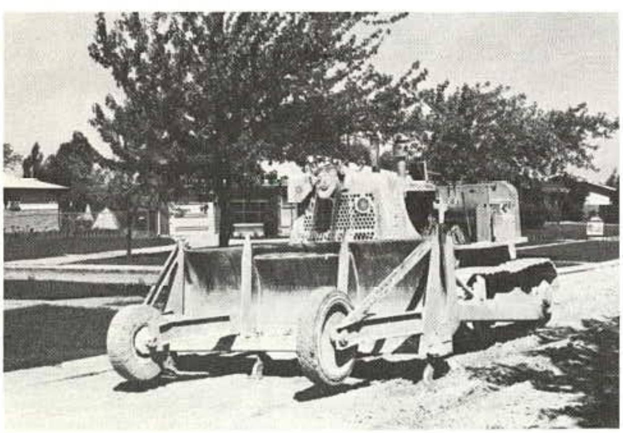


Figure 59. Tractor-mounted spreaders used to distribute LFA mixes over the roadbed.

Figure 60. Tractor-mounted spreader distributing LFA mix from a semi-type dump truck.

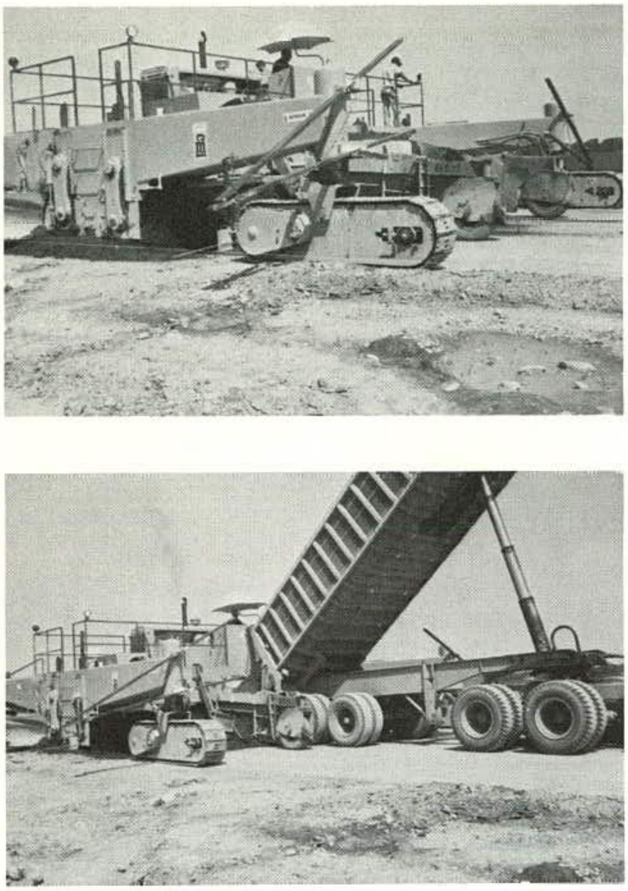


Figure 61. Subgraders used to distribute LFA mix.

Figure 62. Graders for final subgrading of LFA mixes.

should be kept as short as possible so that the lower layer has not set up before the next layer is placed. If the LFA material in the lower layer is fresh and the surface free of loose debris, dirt, or sand, the next layer can be spread without scarifying the lower layer. Subsequent layers should be placed the same day; however with multiple layered pavements, such as airport and marine terminal pavements, this is not always possible. If the LFA mix in the lower layer has taken on an initial set, steps should be taken to ensure the development of a bond between the two layers. Specifically, there should be no loose material on the lower layer; the surface should be moist before placing the LFA material for the subsequent layer.

Compaction

A critical step in the construction of pavement with LFA mixes is compaction. Achieving a high relative density in these materials in place is the key to good performance. Figure 63 shows the final compaction of LFA mixtures with a steel-wheel roller. Steel-wheel, pneumatic, vibratory pan, and vibratory wheel rollers have all been used effectively for compacting LFA mixes. Because the material is basically granular in nature, with little or no cohesion at the time of compaction, pneumatic tire rollers, vibratory rollers, and vibratory pans are usually most effective in providing initial densification of the mixes.

An important factor in achieving good density is an adequate working platform. LFA mixes placed and rolled on a soft subgrade tend to shove rather than densify. This leads to poor quality LFA material and high deflections of the pavements in service. An adequate support for the placement and compaction of LFA mixes is important even if it requires treatment of the existing soil. Treatment of soft subgrades usually results in reduced construction costs and increased pavement performance.

Steel-wheel rollers are generally used only for producing a true and smooth final surface after initial compaction with the other types of compactors. The final surface is usually brought to grade with a grader or string-line subgrader prior to final rolling with steel-wheel rollers.

An advantage of LFA mixes over some stabilized materials is that they can be effectively compacted for an extended period of time after mixing. Compaction within four hours after mixing is strongly recommended; however with some mixes, compaction can be achieved over a longer

Figure 63. Final rolling of a layer of LFA mix. (Preliminary compaction is usually done with vibratory and rubber-tired compactors.)

time span. The length of time that can elapse between mixing and final compaction is a function of the initial reactivity of the mixture and climatic conditions. Generally, compaction should be completed as rapidly as possible to prevent loss of moisture and difficulty in the compaction due to initial set. Most specifications require the material to be compacted within four hours of mixing, and always on the same day on which it is mixed and spread. With some of the fast-setting fly ashes produced from subbituminous coals, it may be desirable to consider using retarders to increase compaction time. Not all retarders are effective, however, and each retarder should be checked with the mix in which it is to be used for effectiveness and possible side effects. With the faster setting fly ashes, the time between mixing and final compaction should be as short as possible, consistent with sound construction practice.

MIXED-IN-PLACE OPERATIONS

Satisfactory quality lime-fly ash mixes have been produced in mixed-in-place operations. The construction procedure consists of preparing a bed of suitable aggregate material of the approximate width of the roadbed, spreading the required amounts of lime-fly ash and water, and mixing with rotary mixers or other mixing equipment (Figs. 64 and 65). After thorough blending of the components to the desired depth, the LFA mix is spread to the required thickness and compacted to the desired density. Although satisfactory performances have been attained with mixes prepared in this manner, the over-all quality of the mixed-in-place operation is less satisfactory than that of plant mix operations. Some problems and limitations with the mixed-in-place operations are discussed in the following section.

Preparation of the Roadbed

In mixed-in-place operations, aggregates already in place on the roadbed can be incorporated into the mix. Although the cost for the aggregates in the mix is greatly reduced, the quality of aggregates obtained in this manner is also usually reduced. Most roadbed aggregates have some soil intermixed, and these soil fines may significantly interfere with the production of a quality LFA mix.

When using in-place aggregates for LFA mix, all the standard mixture proportion tests should be run to evaluate their suitability. It may be necessary to modify the aggregates to produce a satisfactory LFA mix. Specifically, it may be necessary to "sweeten" the in-place material with additional clean aggregate to achieve a satisfactory gradation. If the fine portions of the in-place aggregates contain excessive silts, this may tend to "choke down" the lime-fly ash reactivity, further lowering the quality of mix. If the fines are predominantly clay minerals, lime may be preblended to the aggregate to break down the clay to make a more worakble mix.

If lime is used to make the in-place soil-aggregate workable, the following construction sequence is recommended:

1. Scarify the in-place soil-aggregate material.
2. Spread enough lime on the scarified roadbed to kill the plasticity of the fines and disc the lime into the soil.
3. Allow the lime and soil-aggregate mixture to mellow (usually for 24 hours).
4. Add aggregates and water as required and blend into the mellowed lime and soil-aggregate mixture.
5. Level and smooth to make a prepared aggregate bed of the desired width and thickness for mixing with the lime and fly ash.
6. Spread the lime and fly ash either as a blend or separately.
7. Thoroughly mix the components adding water as necessary to bring the mix to the desired moisture content.
8. Spread and compact to the desired thickness and density.

For conditions where lime or additional aggregates are not required on the road site, steps 2, 3, and 4 can be deleted as appropriate. Steps 1, 5, 6, 7, and 8 apply to all mixed-in-place operations.

Spreading Lime and Fly Ash

In most instances, lime and fly ash are spread separately on the prepared roadbed in the mixed-in-place operations. It is possible, however, to preblend these two components before spreading as, for example, with the "Master Mix" material available from several suppliers. When lime and fly ash are preblended, it is necessary that they be stored in a dry state. They are normally spread in the dry condition.

Lime Spreading

Lime can be delivered and spread on the aggregate bed in either the dry condition or as a slurry. Most lime used for mixed-in-place operations is delivered and spread dry from pneumatic trucks.

Spreading dry lime has two major problems: (*a*) achieving a uniform lime distribution, and (*b*) controlling the dust associated with the discharge from the pneumatic truck. In populated areas, the dusting problem may be severe and special precaution should be taken with this operation.

Figure 64. Rotary mixers used to blend LFA mixes for mixed-in-place operations.

Figure 65. Two views of a mixed-in-place operation.

The major problems associated with spreading the lime in slurry form are the large quantities of water required and the cost of hauling water long distances. In addition, the water used to slurry the lime may cause an excess of water in the mix. The slurry method of lime spreading is practical only when the in-place aggregate requires significant water to bring the mix to optimum moisture content and when an adequate supply of water is nearby and is inexpensive.

Fly Ash Spreading

Nearly all fly ash is spread in the conditioned state (i.e., moisture content at 15 to 25 percent). It is possible to spread dry fly ash from pneumatic trucks, but dusting with this mode of operation is severe and creates special handling problems, especially near populated areas.

Conditioned fly ash is normally delivered in open dump trucks and is dumped and spread with a grader, spreader box, or other types of spreaders. Uniform distribution of the fly ash compared with the aggregate is the major problem with this type of operation.

Blending

As indicated, blending in place is normally done with rotary mixers and similar equipment. Heavy-duty rotary mixers (such as shown in Figure 64) must be used to make this operation successful. Blending can also be done with graders, but this method of blending is much less effective than with rotary mixers. Improper manipulation with graders can result in segregation of the coarse and fine aggregates in the mix.

Compaction

Compaction of LFA for mixed-in-place operations is the same as for plant mix operations.

SEALING AND SURFACING LFA LAYERS

Compacted layers of LFA material should be sealed as soon as possible to prevent loss of moisture. In many instances, a prime coat consisting of from 0.1 to 0.2 gallons per square yard (0.38 to 0.75 litres per square metre) of cut-back liquid or emulsified asphalt is placed the day of compaction, and never later than the day following placement and compaction. Any pavement surface layers are applied as soon thereafter as can be scheduled in the construction sequence. The only justification for delay in surfacing the LFA mixes occurs when heavy rains saturate the base and subbase, making the compacted roadway unstable and causing it to shove and rut under the surfacing equipment and trucks.

CONSTRUCTION SEASON

Construction season varies with the climatic conditions of any particular site and the manner in which the paved section will be used during the first winter. Early-season construction is limited by the dates during which heavy construction can effectively operate on the site after the normal last freezing date. The late-season cutoff date is determined by such factors as the rate of setting of the LFA mix and the anticipated temperature between the last construction date and the beginning of heavy frost penetration. A typical construction season for northern and central Illinois ranges from the last half of April to about mid-October. In more moderate climates, such as at Newark, New Jersey, where no loads are to be placed on the pavement during the winter months, LFA has been placed up to December 1 without any apparent long-term damage.

Procedures have been determined for the systematic determination of the late-season cutoff date based on historical data from a first-order weather station in the area. A model procedure is shown in Appendix B.

SPECIFICATIONS FOR MIXING AND PLACING LFA MIXES

Typical specifications for mixing and placing LFA materials are given in Appendix C. It is emphasized that these are only model specifications and that each agency or user must develop specifications that meet its particular needs. Specifications currently used by Illinois are included in Appendix D.

CHAPTER SEVEN

APPLICATIONS AND LIMITATIONS

As with all paving materials and pavement systems, LFA materials are most effective when used under proper conditions and within specified limitations. Although these materials have wide applicability in pavement construction, there are conditions, of which the proposed user should be aware, involving risks. Some of the conditions and limitations for use of LFA materials for pavement construction are as follows:

- LFA materials can be used for a wide range of pavement systems from low-volume roads to heavy-duty pavements. Appropriate mixture proportion procedures and criteria are available for the entire range of pavement systems.
- LFA materials can be used as either a base or subbase material in flexible pavement systems or as a base material in rigid pavement systems. A riding surface is required for the flexible pavement systems. This can vary from a seal coat for low-volume roads to 4 to 6 in. (100 to 150 mm) of asphaltic concrete for heavy-duty airfield pavements. Use of the seal coat should be limited to very-low-volume roads, preferably with low-speed vehicles.
- The key to good performance with LFA materials is to select a mixture proportion having adequate quantities of lime and fly ash and to employ sound construction techniques. Thorough blending of the components and high relative densities in place will result in good pavement performance with these materials.
- Durability is the single most important property in the performance of LFA materials, especially in areas of cyclic freezing and thawing and where use of deicing salt is heavy. No standard criteria for durability can be given because the level of durability should relate to the *in situ* environmental conditions for the proposed pavements. Durability also varies with the amount of cure the material experiences before it is exposed to detrimental environmental conditions.
- Procedures have been developed for establishing cutoff dates for late-season construction with LFA materials. The procedures outlined in Appendix B are based on the expected curing conditions, the number of freeze-thaw cycles, and the traffic conditions expected at the site. There have been a number of instances in which LFA materials have been placed after the last expected curing weather has passed, allowed to be undisturbed over the winter, and trafficked the following spring and summer without apparent damage. This procedure, while effective, is not recommended for normal use. Application of traffic during the critical freezing and thawing seasons greatly increases the probability of damage in proportion to the amount of traffic

and magnitude of applied loads. Thus, the user must be aware of the potential hazards of late-season construction, and trafficking of insufficiently cured LFA materials.

• High relative density *in situ* is critical for development of high-strength, highly durable materials. Because high density is achieved primarily through compactive effort, it is important that conditions exist for achieving densification with the application of effort. It is particularly important that a firm support be established as a base for compaction. Attempts to achieve a high relative density in materials supported on soft subgrades result in shoving rather than densification of the materials. There are special problems of compaction near the edge of any pavement layer because, without lateral support, the material is likely to shove rather than to densify under the compactor.

• Excessive moisture combined with freezing and thawing and high concentrations of salt is an extreme environmental condition to be avoided. If high groundwater is present at the site, installation of subsurface drainage facilities can provide substantial improvement in performance. Salt brine is detrimental to LFA materials and should be drained from the pavement system as rapidly as possible.

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APPENDIX A

AVAILABILITY OF LIME AND FLY ASH IN THE UNITED STATES

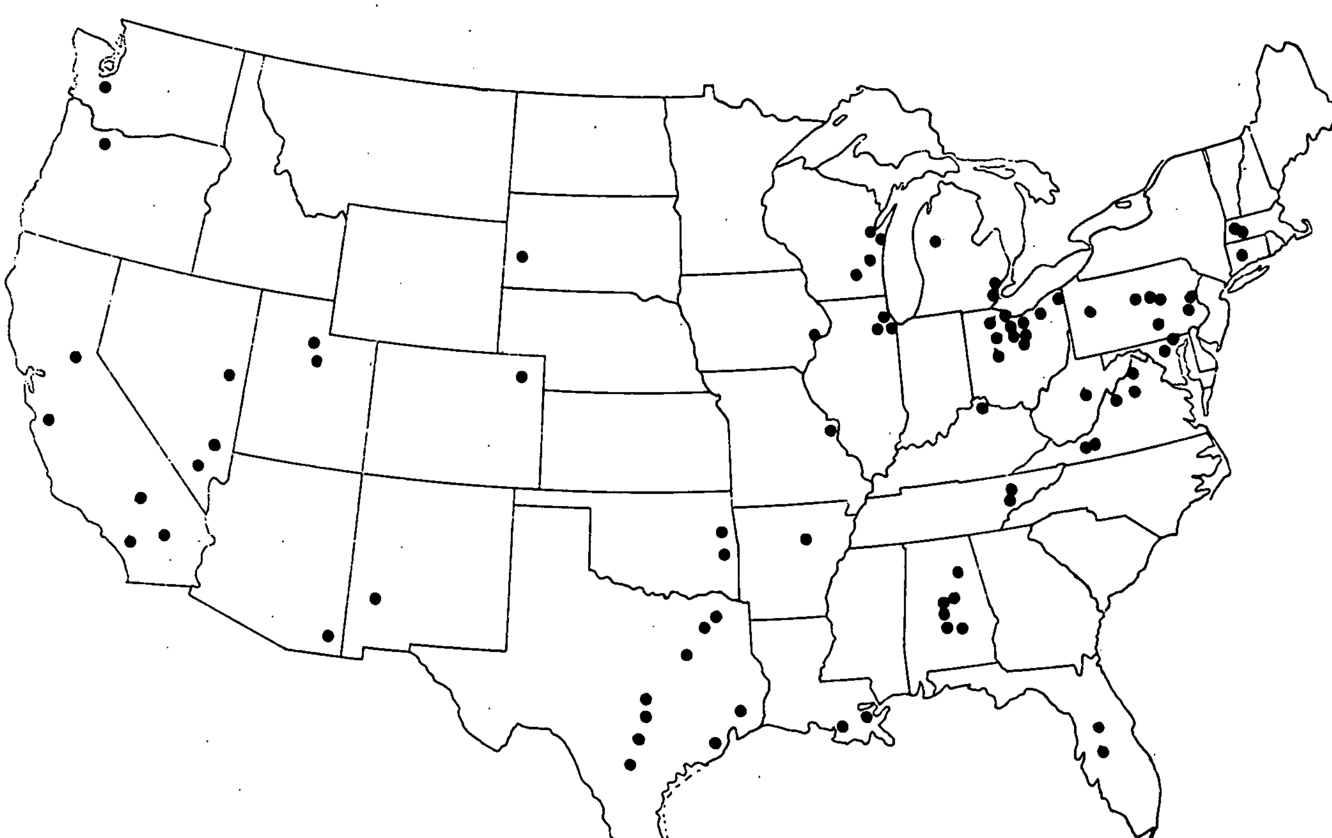

Figure A-1. Commercial lime plants in U.S., 1974.

● - Commercial Plant

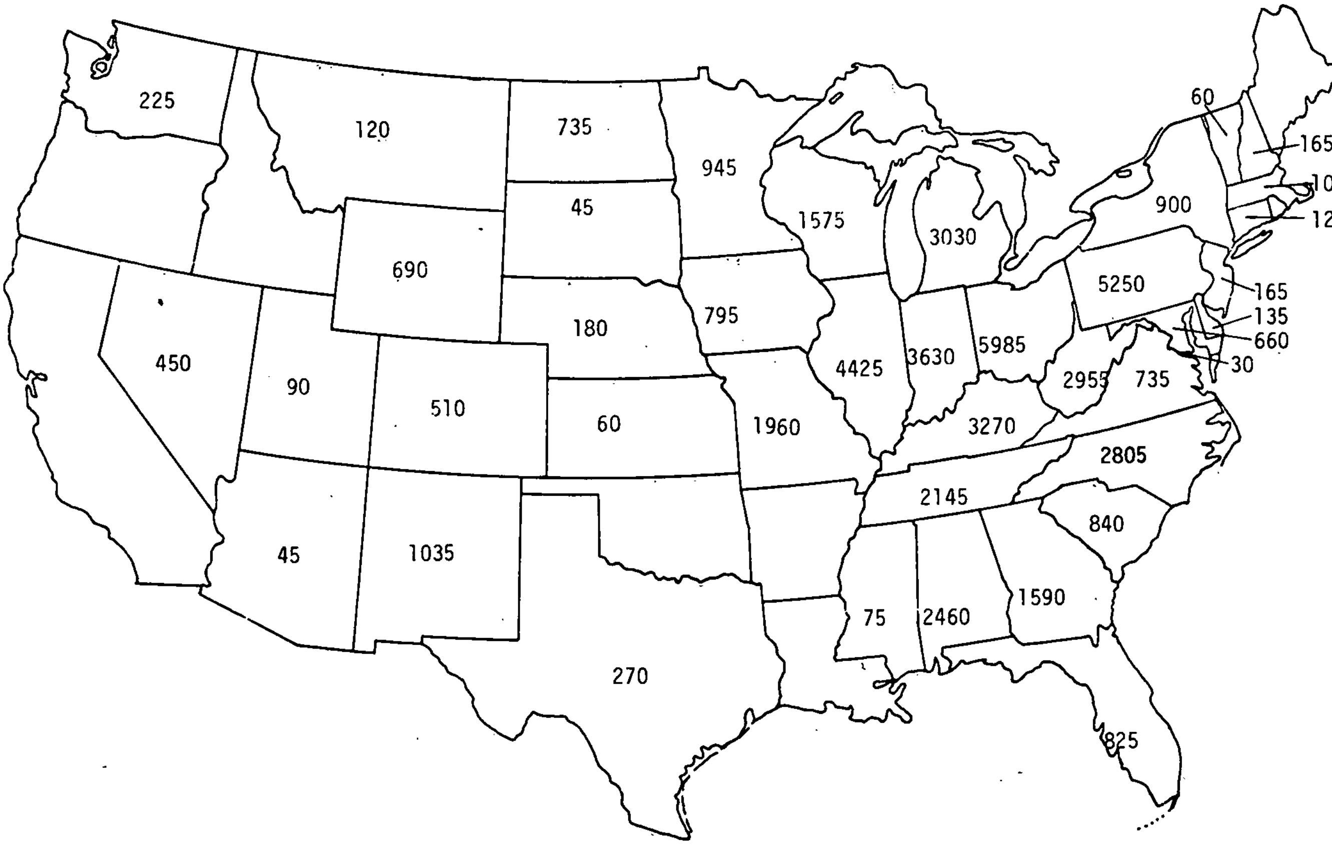

Figure A-2. Approximate ash production (in 1,000s of tons) by major electric utilities, 1973.

APPENDIX B

PROPOSED CUTOFF-DATE PROCEDURES FOR CONTRUCTION WITH LFA MIXES

Strength development of lime-fly ash–aggregate mixes is time- and temperature-dependent. For a particular stabilized fly ash-aggregate mixture, a specified minimum curing, normally expressed in terms of degree days (DD), is required to develop a desired cured strength (CS). For typical conditions in many northern states, little beneficial curing can be achieved on a predictable basis after November 30.

Cyclic freeze-thaw (F-T) action in pavements typically begins in late November or early December. Strength decreases can be caused in the LFA by cyclic F-T action; thus the LFA strength following the completion of the first winter's F-T action (termed the residual strength, RS) is generally less than the CS.

A certain minimum strength called the minimum tolerable strength (MTS) is required for LFA mixes to ensure adequate performance in a pavement system. MTS varies depending on whether the LFA is used as a subbase for a concrete pavement or as a base course in a flexible pavement. Such factors as thickness of asphalt concrete surface course, LFA thickness, subgrade support, traffic, etc., also influence the MTS.

The following procedure determines in a systematic and rational manner, the appropriate cutoff date for construction with LFA mixes with specified cured strength characteristics when used in a specific pavement system. The procedure is based on the residual strength concept (*32*). Figure 18 illustrates the residual strength (RS) and minimum tolerable strength (MTS) concept discussed previously. Procedure:

1. Establish a cured strength-degree day (CS-DD) relationship for the LFA mix. Calculate the DD using a 40 F (4.4 C) base temperature (*20*). Typical data are shown in Figure B-1.
2. Minimum tolerable strength (MTS) requirements for LFA mixes are given in the specifications or can be determined from pavement design criteria.
3. Cured strength (CS) requirements must be consistent with minimum MTS values selected to provide an RS greater than the MTS as illustrated in Figure 18.
4. From Step 1 data, determine the DD required to achieve the CS requirement selected in Step 3 or given in the specifications.
5. From Figure B-2, select the appropriate CUTOFF DATE to provide the accumulation of an adequate number of DD for curing the LFA mix.
6. Adjust the CUTOFF DATE determined in Step 5 for construction and curing variability. The suggested adjustment for the Chicago, Ill., area is to set the CUTOFF DATE seven days earlier than the date obtained from Step 5. The seven days' adjustment is equivalent to approximately 175 DD during mid-October in Chicago.

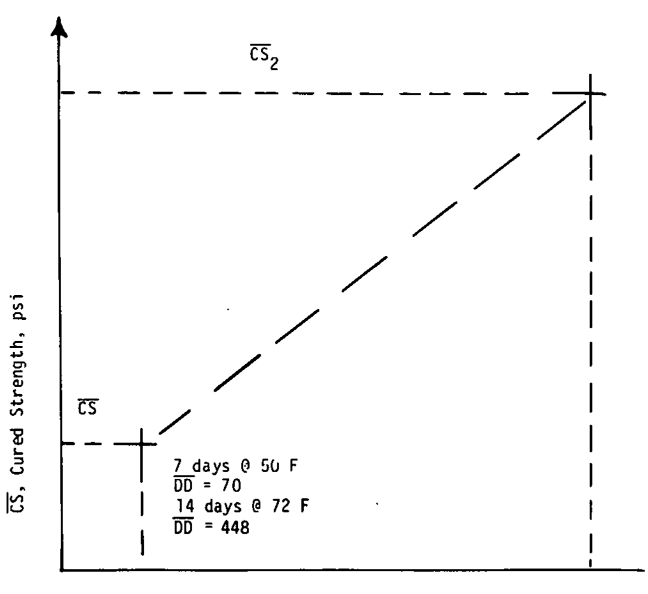

Figure B-1. Typical degree day–cured strength relationship for an LFA mix.

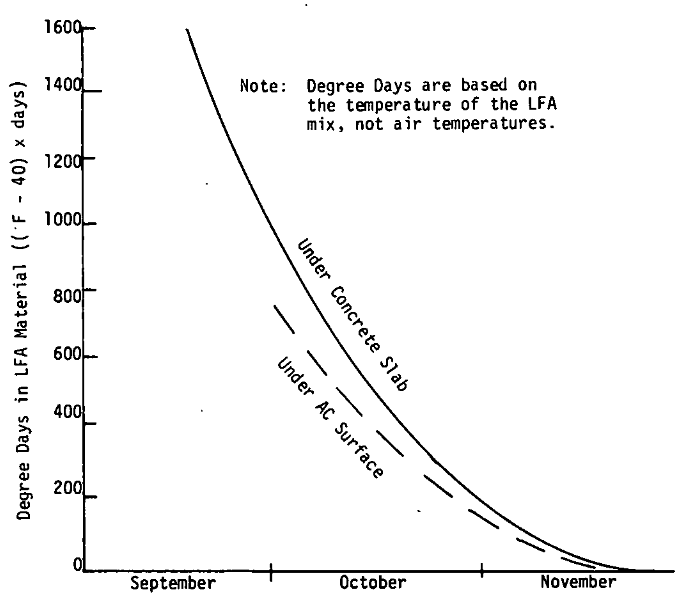

Figure B-2. Typical time–degree day relationship calculated from historical weather data.

APPENDIX C

TYPICAL SPECIFICATIONS FOR LIME-FLY ASH–AGGREGATE BASE/SUBBASE COURSE

1. Description

1.1 This item shall consist of constructing a base course by mixing, spreading, shaping, and compacting mineral aggregate, lime, fly ash, and water. It shall be placed on the prepared underlying course in accordance with the requirements of this specification and shall conform to the dimensions and typical cross sections shown on the plans and to the lines and grades established by the engineer.

2. Materials

2.1 Lime-Fly Ash Cementitious Filler Material.—The lime and fly ash shall be supplied either separately or as a manufactured blend. The lime, fly ash, or blend may contain admixtures such as water-reducing agents, portland cement, or other materials that are known to provide supplementary properties to the final mix. When admixtures are to be included, they are to be used in the laboratory mixture selection.

The lime shall meet ASTM Specification C 207, Type N, sections 2 and 3(a) when sampled and tested in accordance with sections 6 and 7. The fly ash shall meet ASTM Specification C 593, section 3.2, when sampled and tested in accordance with sections 4, 6, and 8. The water-soluble fraction shall not be determined. The preceding requirements may be waived if it is demonstrated that a mix of comparable quality and reliability can be produced with lime and/or fly ash that do not meet these criteria. If portland cement is blended with either lime or fly ash, or both, or added at the mixer, it shall be a standard brand and shall conform to the requirements specified in ASSHTO M 85 for the type specified.

2.2 Water.—The water for the base course shall be clean, clear, and free from injurious amounts of sewage, oil, acid, strong alkalies, or vegetable matter, and it shall be free from clay or silt. If the water is of questionable quality, it shall be tested in accordance with the requirements of AASHTO T 26. Water known to be of potable quality may be used without tests.

2.3 Aggregate.—The aggregate may be either stone, gravel, slag, or sand, crushed or uncrushed, or any combination thereof. In addition to the fine aggregate naturally contained in the coarse material, supplementary fly ash may be used as a mineral filler to provide the desired fines content.

The crushed or uncrushed mass shall consist of hard, durable particles of accepted quality (crushed if necessary to reduce the largest particles to the largest accepted size and free from an excess of flat, elongated, soft, or disintegrated pieces, or dirt or other deleterious materials.

The methods used in processing such as crushing, screening, blending, and so forth, shall be such that the finished product shall be as consistent as practicable. If necessary to meet this requirement or to eliminate an excess of fine particles, the materials shall be screened before and during processing, and all stones, rock, boulders, and other source material of inferior quality shall be wasted.

The aggregate shall show no evidence of general disintegration nor show a total loss of more than 12 percent when subjected to five cycles of the sodium sulfate accelerated soundness test specified in AASHTO T 104; however, if an aggregate source that fails to meet this requirement can show an acceptable performance record in service, it may be accepted.

All material passing the No. 4 (4.75-mm) sieve and produced during crushing or other processing may be incorporated in the base material to the extent permitted by the gradation requirements, unless it is known to contain significant deleterious material.

A wide range of aggregate gradations are permitted with these base materials provided appropriate mixture proportion procedures are followed. If the maximum particle size in the aggregate exceeds 0.75 in. (19 mm), the aggregate shall meet the gradation requirements given in Table C-1 when tested in accordance with AASHTO T 11 and T 27.

The gradation in the table sets limits that shall determine the general suitability of the aggregate from a source of supply. The final gradations selected for use shall be within the limits designated in the table, and shall also be well graded from flue to coarse and shall not vary from high to low limits on subsequent sieves.

In addition to the gradations given in Table C-1, clean sands and sand-sized materials, such as boiler slags, can be used. Also, if the aggregate has a substantial portion (75 percent) passing the No. 4 (4.75-mm) mesh sieve the gradations in Table C-1 can be waived and the aggregate gradation adjusted with the fly ash and fines contents to produce the maximum dry density in the compacted mixture.

The portion of the base material including any blended material passing the No. 40 (425-μm) mesh sieve shall have a liquid limit of less than 25 and a plasticity index of less than 6 when tested in accordance with AASHTO T 89 and T 90.

2.4 Bituminous Material.—The types, grades, controlling specifications, and application temperatures for the bituminous materials used for curing the lime-fly ash-aggregate-

TABLE C-1

REQUIREMENTS FOR GRADATION OF AGGREGATE FOR THE PLANT-MIX BASE COURSE

Sieve designation (square openings)	Percentage by weight passing sieves		
	A	B	C
2 inch (50 mm)	100	-	-
1-1/2 inch (38.1 mm)	-	100	-
1 inch (25 mm)	55-85	70-95	100
3/4 inch (19 mm)	50-80	55-85	70-100
No. 4 (4.75 mm)	40-60	40-60	40-65
No. 40 (425 μm)	10-30	10-30	15-30
No. 200 (75 μm)	5-15	5-15	5-15

TABLE C-2

BITUMINOUS CURING MATERIALS FOR LFA BASES

TYPE AND GRADE	SPECIFICATION	APPLICATION TEMPERATURE, F (C)
Cutback asphalt MC-30	AASHTO M 82	120-150 (49-65)
Emulsified asphalt	Fed. Spec. SS-A-674	75-130 (23-54)

treated base course are given in Table C-2. The engineer shall designate the specific material to be used.

3. Laboratory Tests and Lime-Fly Ash Content

3.1 Lime Content.—The quantity of lime (approximately 2 to 5 percent by weight) to be used with the aggregate, fly ash, and water, shall be determined by tests for the materials submitted by the contractor, at his own expense, and in a manner satisfactory to the engineer.

3.2 Fly Ash Content.—The quantity of fly ash (approximately 9 to 15 percent by weight) to be used with the aggregate, lime, and water shall be determined by tests for the materials submitted by the contractor, at his own expense, and in a manner satisfactory to the engineer.

3.3 Manufactured Blend Content.—The quantity of manufactured blend to be used with the aggregate and water (and any supplemental fly ash) shall be determined by tests for the materials submitted by the contractor, at his own expense, and in a manner satisfactory to the engineer.

3.4 Laboratory Tests.—Specimens of the lime-fly ash–aggregate base course material shall develop a minimum compressive strength of 400 psi (2700 kPa) and demonstrate freeze-thaw resistance of a maximum of 14 percent weight loss as specified in ASTM Specification C 593, section 3.2, when tested in accordance with section 9 of that specification except that all compaction shall be done in accordance with FAA T 611, section 2.2(a) and (b).

4. Construction Methods

4.1 Sources of Supply.—All work involved in clearing and stripping pits, including handling unsuitable material, shall be performed by the contractor. All costs involved in clearing and stripping pits, including labor, equipment, and other incidentals shall be included in the price of the material. The contractor shall notify the engineer sufficiently in advance of the opening of any designated pit to permit staking of boundaries at the site, to take elevations and measurements of the ground surface before any material is produced, to permit the engineer to take samples of the material for tests to determine its quality and gradation, and to prepare a preliminary base mixture proportion. All materials shall be obtained from approved sources.

The pits, as used, shall be opened immediately to expose vertical faces of the various strata of acceptable material and, unless otherwise directed, the material shall be secured in successive vertical cuts extending through all the exposed strata in order to secure a uniform material.

4.2 Equipment.—All methods employed in performing the work and all equipment, tools, other plans and machinery used for handling materials and executing any part of the work shall be subject to the approval of the engineer before the work is started. If unsatisfactory equipment is found, it shall be changed and improved. All equipment, tools, machinery, and plants must be maintained in a satisfactory working condition.

4.3 Preparing Underlying Course.—The underlying course shall be checked and accepted by the engineer before placing and spreading operations are started. Any ruts or soft, yielding places caused by improper drainage conditions, hauling, or any other cause, shall be corrected and rolled to the required compaction before the base course is placed thereon.

Grade control between the edges of the pavement shall be accomplished by grade stakes, steel pins, or forms placed in lanes parallel to the centerline of the runway and at intervals sufficiently close that string lines or check boards may be placed between the stakes, pins, or forms.

To protect the underlying course and to ensure proper drainage, the spreading of the base shall begin along the centerline of the pavement on a crowned section or on the high side of the pavement with one-way slope.

4.4 Mixing.

4.4.1 General requirements.—Lime-fly ash-treated base shall be mixed at a central mixing plant by either batch or continuous mixing. The capacity of the mixing plant should not be less than 50 tons per hour (45 metric tons per hour). The aggregates, lime, and fly ash may be proportioned either by weight or by volume.

In all plants, water shall be proportioned by weight or volume, and there shall be means by which the engineer may readily verify the amount of water per batch or the rate of flow for continuous mixing. The discharge of the water into the mixer shall not be started before part of the aggregates are placed into the mixer. The inside of the mixer shall be kept free from any hardened mix.

In all plants, lime and fly ash (and portland cement when used in the mix) shall be added in such a manner that they are uniformly distributed throughout the aggregates during the mixing operation.

The charge in a batch mixer, or the rate of feed into a continuous mixer shall not exceed that which will permit complete mixing of all the material. Dead areas in the mixer, in which the material does not move or is not sufficiently agitated, shall be corrected either by a reduction in the volume of material or by other adjustments.

4.4.2 Batch Mixing.—In addition to the general requirements as provided in Sec. 4.4.1, batch mixing of the materials shall conform to the following requirements:

—The mixer shall be equipped with a sufficient number of paddles of a type and arrangement to produce a uniformly mixed batch.

—The mixer platform shall be of ample size to provide safe and convenient access to the mixer and other equipment. The mixer and batch-box housing shall be provided with hinged gates of ample size to permit easy sampling of the discharge of aggregate from each of the plant bins and of the mixture from each end of the mixer.

—The mixer shall be equipped with a timing device that will indicate by a definite audible or visual signal the expiration of the mixing period. The device shall be accurate to within two seconds. The plant shall be equipped with an automatic device suitable for counting the number of batches.

—The mixing time of a batch shall begin after all ingredients are in the mixer and shall end when the mixer is half emptied. Mixing shall continue until a homogeneous mixture of uniformly distributed and properly coated aggregates of unchanging appearance is produced. In general, the time of mixing shall be not less than 30 seconds; however, the time may be reduced when tests indicate that the requirements for lime-fly ash content and for compressive strength can be consistently met.

4.4.2.1 Weight Proportioning.—When weight proportioning is used, the discharge gate of the weigh box shall be arranged to blend the different aggregates as they enter the mixer.

4.4.2.2 Volumetric Proportioning.—When volumetric proportioning is used for batch mixing, the volumetric proportioning device for the aggregate shall be equipped with separate bins, adjustable in size, for the various sizes of aggregates. Each bin shall have an accurately controlled gate or other device designed so that each bin shall be completely filled and accurately struck-off in measuring the volume of aggregate to be used in the mix. Means shall be provided for accurately calibrating the amount of material in each measuring bin.

4.4.3 Continuous Mixing.—In addition to the general requirements as provided in Sec. 4.4.1, continuous mixing of the materials shall conform to the following requirements:

—The correct proportions of each aggregate size introduced into the mixer shall be drawn from the storage bins by a continuous feeder, which will supply the correct amount of aggregate in proportion to the lime-fly ash and will be arranged so that the proportion of each material can be separately adjusted. The bins shall be equipped with a vibrating unit, which will effectively vibrate the side walls of the bins and prevent any "hang up" of material while the plant is operating. A positive signal system shall be provided to indicate the level of material in each bin, and as the level of material in any one bin approaches the strike-off capacity of the feed gate, the device shall automatically and instantly close down the plant. The plant shall not be permitted to operate unless this automatic signal is in good working condition.

—The drive shaft on the aggregate feeder shall be equipped with a revolution counter accurate to 1/100 of a revolution and of sufficient capacity to register the total number of revolutions in a day's run.

—The continuous feeder for the aggregate may be mechanically driven or electrically driven. Aggregate feeders that are mechanically driven shall be directly connected with the drive on the lime feeder.

—The pugmill for the continuous mixer shall be equipped with a surge hopper containing sufficient baffles and gates to prevent segregation of material discharged into the truck and to allow for closing of the hopper between trucks without requiring shutdown of the plant.

4.5 Placing, Spreading, and Compacting.—The use of mixers having a chute delivery shall not be permitted except as approved. In all such cases the arrangement of chutes, baffle plates, etc., shall ensure the placing of the lime-fly ash-treated base without segregation.

The prepared underlying course shall be free of all ruts or soft yielding places. The surface, if dry, shall be moistened but not to the extent of precluding a muddy condition at the time the base mixture is placed.

Any dusting or surface ravelling caused by traffic on the sealed base course material shall be the responsibility of the contractor and shall be taken care of as directed by the engineer.

4.6 Construction Joints.—The protection provided for construction joints shall permit the placing, spreading, and compacting of base material without injury to the work previously laid. Care shall be exercised to ensure thorough compaction of the base material immediately adjacent to all construction joints.

4.7 Protection and Curing.—After the base course has been finished as specified herein, it shall be protected against drying until the surface course is applied by the application of bituminous material or other acceptable methods, such as periodic application of water by a pressure water distributor. A double seal shall be used for the small projects where a surface course layer is not required.

The bituminous material specified shall be uniformly applied to the surface of the completed base course at the rate of approximately 0.15 gallons per square yard (0.68 litre per square metre) using approved heating and distributing equipment. The exact rate and temperature of application to give complete coverage without excessive runoff shall be as directed by the engineer. At the time the bituminous material is applied, the surface shall be dense, free of all loose and extraneous material, and shall contain sufficient moisture to prevent penetration of the bituminous material. All surfaces shall be cleaned of all dust and unsound materials to the satisfaction of the engineer. Cleaning shall be done with rotary brooms and/or blowing the surface with compressed air, with the surface reasonably moistened to prevent air pollution. Water shall be applied in sufficient quantity to fill the surface voids immediately before the bituminous curing material is applied.

Should it be necessary for construction equipment or other traffic to use the bituminous-covered surface before the bituminous material has dried sufficiently to prevent pickup, sufficient granular cover shall be applied before such use.

No traffic shall be allowed on the pozzolan base course other than that developing from the operation of essential construction equipment unless otherwise directed by the engineer. Any defects that may develop in the construction of the base course or any other damage caused by the operation of the job equipment is the responsibility of the contractor and shall be immediately repaired or replaced at no expense to the sponsor.

Other curing materials, such as moist straw or hay, may be used upon approval. Upon completion of the curing period, the straw shall be removed and disposed of as directed by the engineer.

Trucks for transporting the mixed base material shall be provided with protective covers. The material shall be spread on the prepared underlying course to such depth that, when thoroughly compacted, it will conform to the grade and dimensions shown on the plans. No time limit is required for placing the base material; however, it is suggested that the base material be placed within several hours to avoid the necessity of replacing moisture that may be lost.

The materials shall be spread by a spreader box, self-propelled spreading machine, or other method approved by the engineer. It shall not be placed in piles or windrows without the approval of the engineer. If spreader boxes or other spreading machines are used that do not spread the material the full width of the lane or the width being placed in one construction operation, care shall be taken to join the previous pass with the last pass of the spreading machine. The machine shall be moved back approximately every 600 ft (180 m) when staggered spreading machines are not used. The first pass shall not be compacted to the edge and, if necessary, the loose material shall be dampened just prior to joining the next pass. When portland cement is used in the mixture, if the temperatures are 70 F (21 C) or more, the materials must be spread within four hours and reworked into the adjacent material. When portland cement is used in the mixture, and the temperatures are less than 70 F (21 C), the materials must be spread within eight hours and worked into the adjacent material. Additional moisture may be required during the reworking operations as directed by the engineer.

The equipment and methods employed in spreading the base material shall ensure accuracy and uniformity of depth and width. If conditions arise where such uniformity in the spreading can not be obtained, the engineer may require additional equipment or modification in the spreading procedure to obtain satisfactory results. Spreading equipment shall be no more than 30 ft (90 m) nor less than 9 ft (2.7 m) in width unless approved by the engineer.

After spreading, the material shall be thoroughly compacted by rolling. The rolling shall progress gradually from one side toward previously placed material by lapping uniformly each preceding rear-wheel track by one-half the width of such track. Rolling shall continue until the entire area of the course has been rolled by the rear wheels. The rolling shall continue until the material is thoroughly compacted, the interstices of the material reduced to a minimum, and until creeping of the material ahead of the roller is no longer visible. Rolling shall continue until the base material has been compacted to not less than 97 percent density, as determined by the compaction-control tests specified in ASTM C 593. Blading and rolling shall be done alternately, as required or directed, to obtain a smooth, even, and uniformly compacted base. Finishing operations shall continue until the surface is true to the specified cross section and until the surface shows no variations of more than 0.38 in. (9.5 mm) from a 16-ft (4.9-m) straightedge laid in any location parallel with, or at right angles to, the longitudinal axis of the pavement.

4.8 Cold Weather Protection.—During cold weather if the air temperature unexpectedly drops below 35 F (1 C) and remains there for a period of several days or more, the completed base course shall be protected from freezing by any approved method if required by the engineer prior to application of the bituminous surface course. Any light surface frost caused by overnight below-freezing temperatures shall be treated by rolling the surface with a light steel-wheel roller as directed by the engineer.

4.9 Thickness.—The thickness of the base course shall be determined from measurements of cores drilled from the finished base or from thickness measurements at holes drilled in the base at intervals so that each test shall represent no more than 300 square yards (250 square metres). The average core thickness shall be the thickness shown on the plans, except that if any one thickness shown by the measurements made in one day's construction is not within the tolerance given, the engineer shall evaluate the area and determine if, in his opinion, that section shall be reconstructed at the contractor's expense or the deficiency is to be deducted from the total material in place.

5. Methods of Measurements

5.1 The quantity of one course, lime-fly ash-treated base, to be paid for will be determined by measurement of the number of square yards of base actually constructed and accepted by the engineer as complying with the plans and specifications.

6. Basis of Payment

6.1 Payment shall be made at the contract unit price per square yard for lime-fly ash base course. This price shall be full compensation for furnishing all materials and for all preparation, manipulation, and placing of these materials and for all labor, equipment, tools, and incidentals necessary to complete the item.

7. Testing and Material Requirements

Test and short title:

AASHTO T 26—Water
AASHTO T 96—Abrasion
AASHTO T 104—Soundness
AASHTO T 11 and T 27—Gradation
AASHTO T 89—Liquid Limit
AASHTO T 90—Plastic Limit and Plasticity Index
AASHTO T 136—Freeze-Thaw Compressive Strength

Material and short title:

ASTM C 207—Lime
ASTM C 593—Fly Ash
AASHTO M 85—Portland Cement, ASTM C 150
AASHTO M 134—Air-Entrained Portland Cement, ASTM C 226
AASHTO M 82—Asphalt MC, ASTM D2027
SS-A-674—Asphalt Emulsion

APPENDIX D

State of Illinois

Department of Transportation

SPECIAL PROVISION
FOR

POZZOLANIC BASE COURSE, TYPE A

Effective April 1, 1964
Revised November 1, 1973

DESCRIPTION. This item shall consist of a base course composed of lime, pozzolan, coarse aggregate and water, plant-mixed and constructed on a prepared subgrade, in accordance with the requirements of this special provision and applicable portions of the Standard Specifications for Road and Bridge Construction, adopted July 2, 1973, to the lines, grades, thicknesses and cross sections shown on the plans or established by the Engineer.

MATERIALS. All materials shall meet the requirements of the following Articles of Section 700 - Materials:

Item	Article
(a) Water	702.01 - 702.02
(b) Aggregate (Note 1)	704.05
(c) Lime (Note 2)	
(d) Pozzolan (Note 3)	
(e) Bituminous Material	713.01 - 713.06, 713.10, 713.11
(f) Water Reducing Admixture (Note 4) .	718.13
(g) Sand Cover	703.01(a), 703.01(e)

Note 1. The gradation requirements shall be as follows:

Passing 1 1/2" sieve	100%
Passing 1" sieve	90-100%
Passing 1/2" sieve	60-100%
Passing No. 4 sieve	40- 70%
Passing No. 40 sieve	0 -25%
Passing No. 200 sieve	
(Gravel)	0- 10%
(Crushed stone & slag)	0- 15%

Boiler Slag. In addition to the aggregates permitted in Article 704.05, boiler slag may be used. The slag shall be wet-bottom boiler slag produced as a by-product of a power plant burning pulverized bituminous coal. The slag shall be composed of hard durable particles and shall be free of excessive or harmful amounts of foreign substances. Boiler slag in an oven dry condition shall meet the following gradation requirements:

Passing No. 4 sieve	80-100%
Passing No. 10 sieve	55- 90%
Passing No. 40 sieve	0- 25%
Passing No. 200 sieve	0- 10%

Granulated slag will be permitted only when authorized in writing by the Engineer.

Note 2. Lime. The lime, either high calcium or dolomitic hydrate, shall comply with the requirements of ASTM C 207, Hydrated Lime for Masonry Purposes, Type N, with the following modifications:

(1) Total calcium and magnesium oxides (non-volatile basis) min. percent 90

(2) Calcium oxide in hydrated lime (as received basis) max. percent 5

(3) Magnesium hydroxide (as received basis) max. percent 5

(4) Mechanical Moisture in hydrated lime (as received basis) max. percent 4

(5) Residue. The sieve analysis of the lime residue shall be as follows:

Sieve	Maximum Percent Retained
No. 4	0%
No. 30	2.5%
No. 100	15 %

Note 3. Pozzolan. The pozzolan, prior to dampening thereof to alleviate the dust problem, shall comply with the requirements of ASTM C 593. The maximum loss on ignition determined in accordance with the procedures of ASTM C 311 shall be 10%. The tests prescribed by this specification shall be performed at the option of the Engineer. At the time of mixing the pozzolan shall be, when dry sieved, in a finely divided condition, as follows:

Sieve	Minimum Percent Passing
1/2 inch	100%
3/8 inch	95%
No. 10	75%

The moisture content of dampened pozzolan shall not exceed 35 percent.

Note 4. A water reducing admixture may be used if permitted by the Engineer. No adjustments will be made in the required lime and pozzolan contents for this addition.

SAMPLES. The Contractor, shall at his own expense, submit to the Engineer a minimum of 25 pounds of lime, 50 pounds of fly ash, and 100 pounds of aggregate which he proposes for use in the pozzolanic mixture. The lime, when sampled, shall immediately be placed in a sealed container and shall be kept sealed. Samples shall be furnished at least 60 days prior to the construction of the pozzolanic base course. The samples as submitted will be tested for acceptance of materials and also to determine whether or not they will produce a satisfactory mixture; and will be used to determine preliminary proportions for the mixture composition.

EQUIPMENT. The equipment shall meet the requirements of the following Articles of Section 800 - Equipment:

Item	Article
(a) Three-wheel Roller (Note 1)	801.01
(b) Tandem Roller (Note 1)	801.01
(c) Tamping Roller (Note 2	801.01
(d) Pneumatic-tired Roller	801.01
(e) Trench Roller (Note 3)	

Note 1. Three-wheel rollers and tandem rollers shall weigh from 6 to 12 tons and shall have a compression on the drive wheels of not less than 190 pounds nor more than 400 pounds per inch width of roller. Vibrating rollers or vibrating compactors will be permitted if approved by the Engineer.

Note 2. In addition to the requirements of Article 801.01, the tampers shall be long enough to penetrate within one inch of the prepared subgrade on the initial rolling.

Note 3. Trench rollers shall be self-propelled and shall develop a compression of not less than 300 pounds nor more than 400 pounds per inch of width on the compaction wheel. The width of the compaction roll shall be not less than 20 inches and its diameter shall be not less than 60 inches. Trench rollers shall meet the approval of the Engineer.

GENERAL CONDITIONS. Except in specific cases, when otherwise permitted by the Engineer in writing, the pozzolanic aggregate mixture shall be constructed between April 15 and September 15 and only when the air temperature in the shade is above 40^{0} F. The amount of pozzolanic aggregate mixture constructed shall be limited to that which can be surfaced during the current construction season. No mixture shall be deposited on a frozen or muddy roadbed. In specific cases, the Engineer may order, in writing, waiver of this limitation.

The applicable provisions of the General Requirements for Base Course Section 300 of the Standard Specifications shall apply.

Wherever the Standard Specifications are referred to hereinafter, and the term "aggregate" is used in the Standard Specifications with reference to base course material, it shall be construed to include pozzolanic base course mixture.

COMPOSITION OF POZZOLANIC AGGREGATE MIXTURE. The lime, pozzolan, and aggregate shall be proportioned within the following approximate limits on a dry-weight basis:

	APPROXIMATE PER CENT BY WEIGHT OF TOTAL DRY MIXTURE	
Ingredient	Gravel, Crushed Stone Crushed Slag or Aggregate Blend	Boiler Slag
Lime	2 to 5	2 to 4
Pozzolan	8 to 20	15 to 30
Aggregate	75 to 90	56 to 78

The actual proportions of lime, pozzolan, water, and aggregate will be set by the Engineer before work begins and will be based on tests conducted on mixtures composed of samples of the constituent materials furnished by the Contractor. The right is reserved by the Engineer to make such changes in proportions during the progress of the work as he may consider necessary.

The composition of the mixture shall be such that when molded into cylinders, cured and tested as stated in the following paragraph, the cylinders shall have a minimum average compressive strength of 400 psi and no individual test shall be lower than 300 psi, and such that the loss in weight shall not be more than 10% after 12 cycles of freezing and thawing when tested in accordance with the applicable paragraph of ASTM C 593.

Test cylinders shall be molded at the optimum moisture content and maximum density in accordance with AASHO T 180, Method C, except that the 5 lift requirement is replaced with 3 lifts. The molded specimens shall then be placed in a sealed container to preserve moisture content and cured in an oven with forced air circulation for 7 days at 100 ± 3^{0} F. At the end of 7 days, the test cylinders for compressive strength testing shall be removed from the containers, allowed to cool to room temperature, soaked for 4 hours, capped and broken for compressive strength within one hour of time of removal from water.

MIXING. The constituents of the mixture shall be accurately proportioned and thoroughly mixed in a mechanical mixer at a central mixing plant. The measuring devices for proportioning the mixture, either by volume or by weight shall be of such accuracy that the proportions of the mixture based on total dry weight will be maintained within the following tolerances:

Lime	± 0.3 percent by weight
Pozzolan	± 1.5 percent by weight
Aggregate	± 2.0 percent by weight

The equipment used must be provided with means, meeting with approval of the Engineer, for calibration and check tests of measuring devices. In all plants, the water shall be proportioned by weight or volume and there shall be means by which the Engineer may readily verify the amount of water per batch or the rate of flow for continuous mixing. If a water reducing admixture is used, the automatic dispensing system shall be capable of continuously introducing the desired quantity of admixture within the range of ± 0.03 gallons per minute.

The moisture content at the time of mixing shall be such that the moisture content at the time of compaction will be within 85 to 110 percent of the optimum moisture determined. The contractor shall provide a platform scale and make arrangements for the use of a certified truck scale of sufficient capacity for calibration and periodic check tests of the feeders or measuring devices as needed during production.

Mixing operations shall be continued until all ingredients are distributed evenly throughout the mixture and a uniform mixture, free of segregation, is obtained. The mixer shall be capable of discharging the mixture without undue segregation.

SUBGRADE. The subgrade shall be prepared in accordance with Articles 212.03, 212.04, 212.08 and 212.09. References therein to base course shall be construed to include pozzolanic aggregate mixture.

PLACING AND COMPACTING AND FINISHING POZZOLANIC AGGREGATE MIXTURE. The pozzolanic aggregate mixture shall be constructed in layers not more than 6 inches (compacted) in thickness, except that if tests indicate that the desired results are being obtained, the compacted thickness of any layer may be increased to a maximum of 8 inches. When the thickness specified is more than 6 inches the mixture shall be placed in 2 or more approximately equal layers. Each layer shall be deposited, full width directly on the prepared subgrade or on the preceding layer of compacted mixture with a mechanical spreader or spreader box of a type approved by the Engineer. Where the mixture must be placed in more than one layer, the previous layer shall be maintained in a moistened condition until the succeeding layer is placed. After having been tested for density and approved by the Engineer, the previous layer shall be dampened with water if required by the Engineer and roughened immediately prior to placing the succeeding layer so that the layers are knit together. The second layer must be placed the same day as the first layer. When placed, the pozzolanic aggregate mixture shall be free from segregation and shall require minimum blading and manipulation.

The pozzolanic base course shall be compacted to at least 97% of maximum density except that if more than one layer is required, the first layer shall be compacted to 97% of maximum density and succeeding layers shall be compacted to 100% of maximum density. The maximum density shall be determined in the same manner as that herein described for test cylinder preparation.

The density of each layer of the compacted base course shall be determined by the Engineer at regular intervals in accordance with AASHO T 191 or by other methods approved by the Engineer, for compliance with these specifications. If these tests indicate that the layer does not comply with the density requirements, the conditions shall be corrected or the material replaced to meet these specifications.

All pozzolanic mixture shall be placed and compacted the same day it is mixed. The entire base course within an increment of work shall be completed within a single working day.

In constructing the top layer, the grade shall be kept at sufficient height so that the top surface, when compacted, will be at or slightly above grade, rather than below grade. Finish grading shall be accomplished by removing excess material followed by recompaction by rolling. In the event that low areas occur, they shall be loosened to the full depth of the lift, dampened with water immediately before placing additional mixture, and then rolled to the satisfaction of the Engineer.

If any subgrade material is worked into the pozzolanic aggregate mixture during the compacting or finishing operations, all pozzolanic mixture within the affected area shall be removed and replaced with new material.

The Engineer may restrict hauling over partially completed work after inclement weather or at any time when the subgrade is soft and there is a tendency for the subgrade material to work into the pozzolanic aggregate mixture.

If for any reason construction operations are delayed or suspended and the Engineer orders any loose or uncompacted material removed and disposed of, the Contractor shall perform this work at his own expense. No pozzolanic aggregate mixture may be salvaged.

CURING. After the pozzolanic aggregate mixture has been constructed as specified herein, the moisture content of the surface material shall be maintained at or slightly below its optimum moisture content until the curing coat is applied. At the time the curing coat is applied, the surface shall be tightly knit and free of all loose or extraneous material. The bituminous curing coat shall be applied the day following final compaction of the mixture unless it should be delayed in the judgement of the Engineer. The bituminous curing coat used shall be that designated by the Engineer and applied at the rate of approximately 0.20 gallons per square yard. It shall be applied uniformly to the surface of the pozzolanic aggregate mixture by a pressure distributor, meeting the requirements of Article 802.05, to produce complete coverage without excessive runoff. The exact rate of application and temperature shall be specified by the Engineer. Should it be necessary for construction equipment to use the base course before the curing coat has cured enough to prevent pick-up, sufficient sand cover shall be applied to prevent pick-up.

The equipment used for wetting the finished pozzolanic aggregate mixture with water and to apply the bituminous material shall be of such limited weight that its use will not cause marring or rutting of the surface.

At least one day shall elapse after the curing coat is applied before the pavement is constructed.

CONSTRUCTION JOINTS AND MAINTENANCE. At the end of each day's construction, a straight transverse construction joint shall be formed by cutting back into the completed work to form a vertical face. Damage to completed work shall be avoided. The pozzolanic aggregate mixture shall be constructed and finished full width each day without longitudinal joints.

The Contractor shall maintain, at his own expense, the entire base course in a manner satisfactory to the Engineer until the pavement has been completed. Maintenance shall include immediate repairs of any defective or damaged portions of the base course. Repairs or replacements shall be made in such a manner as to insure restoration of a uniform surface and durability of the portion repaired or replaced. The Contractor shall also remove and replace at his own expense any pozzolanic aggregate mixture which is unsatisfactory due to its being placed over excessively wet or otherwise unstable subgrade; damaged by rain, freezing or other climatic conditions; damaged by traffic; or which is unsatisfactory due to failure to comply with the requirements specified herein.

No traffic other than essential construction equipment shall be allowed on the finished base until a wearing course has been constructed. At least five days shall elapse after the base course is completed before the wearing course is constructed.

COMPENSATION

TOLERANCE IN THICKNESS. It is the intent that the base course shall be constructed to the nominal thickness shown on the plans. Thickness determinations shall be made at such points as the Engineer may select. When the constructed thickness is less than 90 percent of the nominal thickness, it shall be brought to nominal thickness by the addition of the applicable mixture or by removal and replacement with new mixture. However, the surface elevation of the completed base course shall not exceed by more than 1/4 inch the surface elevation shown on the plans or authorized by the Engineer.

METHOD OF MEASUREMENT. The work will be measured for payment as follows: When work is constructed essentially to the lines, grades or dimensions shown on the plans and the Contractor and the Engineer have agreed in writing that the plan quantities are accurate, no further measurement will be required and payment will be made for the quantities shown in the contract for the various items involved except that if errors are discovered after work has been started, appropriate adjustments will be made.

When the plans have been altered or when disagreement exists between the Contractor and the Engineer as to the accuracy of the plan quantities, either party shall, before any work is started which would affect the measurement, have the right to request in writing and thereby cause the quantities involved to be measured as hereinafter specified.

Stabilized base course of the thickness specified shall be measured in place and the area computed in square yards completed in accordance with this specification. The width for measurement shall be from outside to outside of the top of the final layer of the completed work as shown on the plans or as directed by the Engineer.

The liquid asphalt for the curing coat for the pozzolanic aggregate mixture, and any sand cover required will not be measured for payment, but shall be considered as incidental to the contract.

BASIS OF PAYMENT. This work will be paid for at the contract unit price per square yard for POZZOLANIC BASE COURSE, TYPE A of the thickness specified.

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