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FLY ASH AS A CONSTRUCTION MATERIAL FOR HIGHWAYS

A MANUAL

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and Bernadette S. Kapples

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16. Abstract The pozzolanic properties, light unit weight, and good shear strength of fly ash have led to its utilization as a highway construction material in several areas of this country and in several European countries, particularly Great Britain. Fly ash can be substituted for many conventional materials which are dwindling in supply or escalating in cost. Utilization of this by-product also eliminates the cost and environmental problems associated with its disposal.

The pozzolanic properties of fly ash, which enable it to react with lime to form cementitious products, have made fly ash a good quality base or subbase course material when used with lime or cement to stabilize aggregates and soils, or when used alone with lime or cement. Strength and durability criteria have been established for this application, and appropriate testing procedures have been developed. Construction procedures utilize standard equipment and techniques for central mixing or mix in-place operations.

Fly ash is used as embankment or structural backfill material over weak or compressible soils because of the reduced surcharge that results from its light unit weight. In addition, its good shear strength properties result in low compressibility and good stability characteristics. Economies can be realized in the design of retaining structures backfilled with lightweight fly ash.

Fly ash improves the flow properties and strength characteristics of grouts. It can be used alone for void-filling, or used in conjunction with Portland cement, lime, clay, sand, and gravel to develop grouts for applications related to highway structures.

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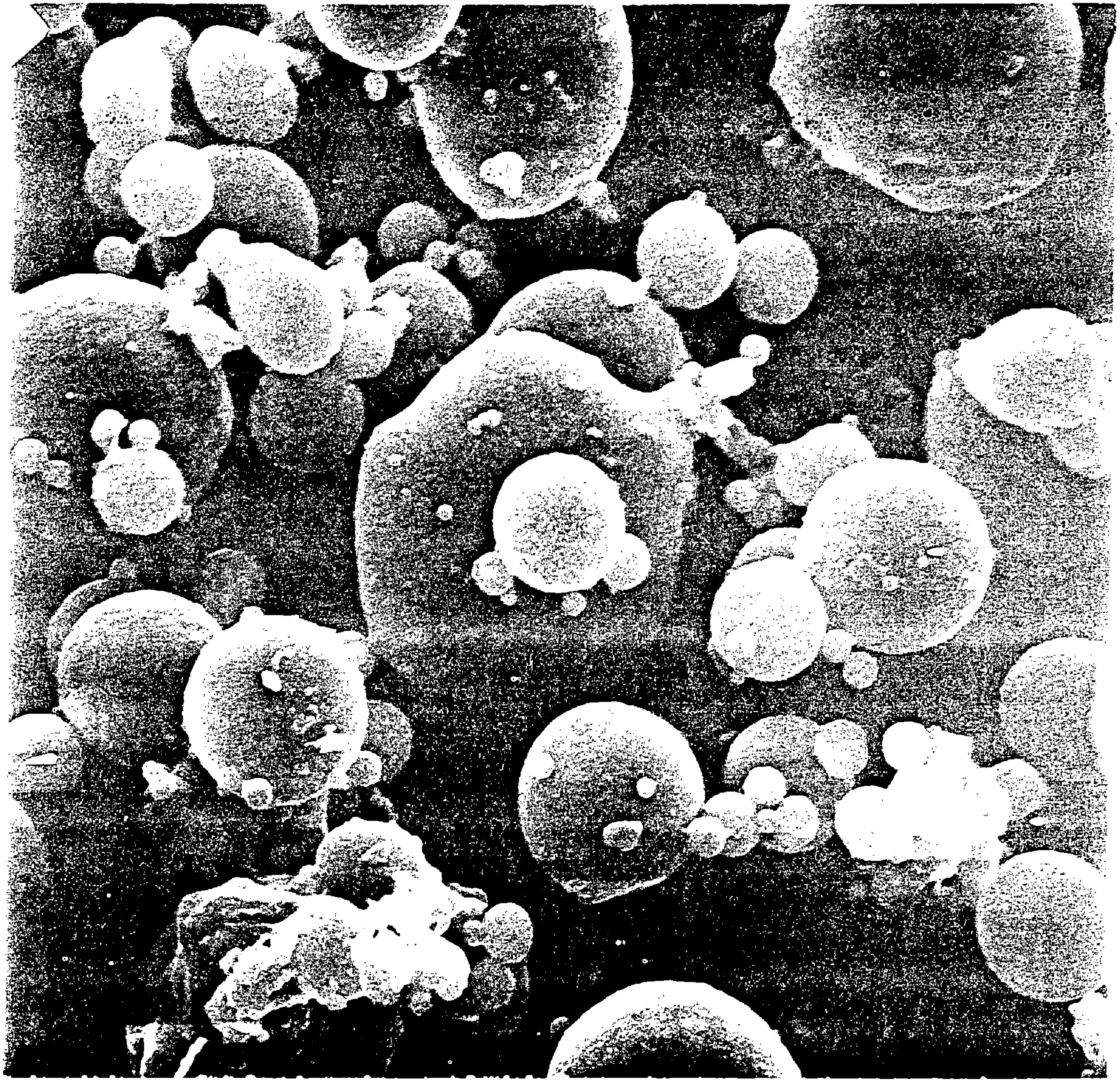

FLY ASH--PEPCO, DICKERSON, MD. Magnification-6,000X

Cover photo by Federal Highway Administration, Offices of Research and Development using a scanning electron microscope. Magnification-20,000X.

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SYNOPSIS

The purpose of this manual is to encourage the utilization of fly ash by providing the technical information necessary to incorporate fly ash into various highway applications, specifically pavements, embankments, backfills, and grouts. The need for large-tonnage utilization of fly ash stems from the large cost and environmental problems associated with its disposal, and the need to find economic substitutes for conventional construction materials which are becoming increasingly scarce and/or expensive.

Fly ash is a powdery by-product of the coal combustion process which is recovered from flue gases, and is usually associated with electric power generating plants. Fly ash is somewhat variable in composition, but is composed primarily of silica, alumina, and various other oxides and alkalies. The chemistry of fly ash is such that it exhibits pozzolanic properties in the presence of lime and moisture, i.e., cementitious products are produced which result in a material of increased strength and durability. Some fly ashes contain sufficient amounts of free lime to self-harden without the addition of a stabilizer such as lime or cement. Fly ash also has a relatively low unit weight which makes it a desirable fill material, particularly over weak soils or behind retaining structures.

Fly ash has been used in conjunction with lime and aggregate to produce a high quality base course in flexible pavement systems and subbase course in rigid pavement systems. Portland cement can be added as a strength accelerator if desired. The mixtures are designed to meet strength and durability criteria. A seven-day unconfined compressive strength of 400 psi (2760 kPa) is often specified in conjunction with durability criteria based on allowable weight-loss after freeze-thaw brushing tests (10-14 percent) or residual strength after a specified number of freeze-thaw cycles or vacuum saturation [400 psi (2760 kPa)]. The amount of fly ash plus stabilizer (lime and/or cement) required is slightly in excess of that required for maximum density of the stabilizer-fly ash-aggregate mixture. The ratio of stabilizer to fly ash is then determined from economic considerations and trial mix strength and durability tests. Thickness design procedures are usually based on either the equivalency method, as developed from the AASHTO Road Test; or the structural design method, using either the Westergaard slab theory, the elastic layered system theory, Meyerhof's utlimate load theory, or a combination of these. A nomograph based on the ultimate load theory is available for layer thickness determinations. Construction procedures are divided into central plant and mix-in-place operations, and utilize standard construction equipment and techniques.

Fly ash alone can be stabilized with either cement or lime and used as base or subbase course material. Construction mixes are developed for either a single source of fly ash or a combination of sources to take into account the variability of fly ash which may be secured for a given project. Strength criteria which have been used for cement-stabilized fly ash are (1) seven-day unconfined compressive strengths of 400-450 psi (2760-3100 kPa), and (2) increasing strength with time (usually measured at 28 days). Recommended criteria for slower-curing lime-stabilized fly ash are (1) 28-day strengths of 550-600 psi (3790-4135 kPa), and (2) increasing strength with time (usually measured at 45 days). Durability testing is recommended for severe service conditions. Proper proportions of the stabilizer and fly ash construction mix are determined from trial mixes which are selected in accordance with general guidelines and tested for strength (also durability, if necessary). The thickness design procedure is based on the Portland Cement Association procedure for soil-cement and involves the determination of a fatigue factor and the subgrade modulus. Recommended construction procedures are based on central mixing techniques, although mix-in-place methods can be used if necessary.

Fly ash can be used in conjunction with lime, and, in some cases, alone, to stabilize soils for base or subbase courses, and to modify subgrade soils to provide additional support for the pavement or to expedite construction. Lime fly ash mixtures are able to reduce plasticity, improve drainage, and reduce shrinkage of many soils, as well as produce a cementitious matrix which further increases the soil's strength and durability. For base and subbase course applications, strength and durability criteria similar to those for lime-fly ash-aggregate mixtures are recommended. General guidelines have been developed for selecting the proportions of each component necessary to produce a satisfactory mixture, although trial mixes must be tested for strength and durability to permit final selection on the basis of quality and economy. For subgrade modification purposes, no specific criteria has been established. Various trial mixes are evaluated primarily on their success in modifying the soil properties in question. Some fly ashes, particularly those which contain large amounts of free lime, have been successful in stabilizing or modifying certain types of soils without the addition of lime. Mix-in-place construction procedures are most commonly used for stabilizing or modifying soils with lime and fly ash, although strict quality control is necessary.

The light weight and good shear strength characteristics make fly ash an excellent embankment material, particularly over weak or compressible soils. Most fly ashes tested have been found to have shear strength characteristics which would make them ideal for light load-bearing fills, such as highway embankments. Fly ashes which are stabilized with lime or cement, or which have inherent self-hardening properties, have shear strength parameters which increase with time. The compressibility of most fly ashes is similar to that for cohesive soils, although stabilized or self-hardening fly ashes are less compressible, particularly if allowed to cure prior to loading. Fly ashes should be evaluated for frost susceptibility, and either stabilized within frost zones or kept below frost zones if found to be frost susceptible. Capillary rise in fly ash is controlled by providing a granular drainage blanket in the vicinity of the water table. Erosion portection can be provided by vegetative cover or with various short-term methods of surface protection. A well-compacted, properly drained embankment with adequate surface treatment represents no danger to surrounding ground and surface waters. Construction procedures for fly ash embankments are very similar to those for conventional soils.

Because of the previously mentioned qualities of light weight and good shear strength, fly ash has been successfully used as structural backfill. Certain economies in design of retaining structures can be realized due to reduced lateral pressures which result from these considerations. Fly ash backfill behind bridge abutments can significantly reduce the differential settlement which often occurs in these locations. In design procedures, fly ash is generally assumed to behave as a granular backfill, although field measurements of lateral pressures on retaining structures backfilled with fly ash have had little success in either verifying or disproving this assumption. Fly ash backfills can be easily compacted with light equipment.

Fly ash is used in grouts because of the advantageous effects its particle size, low unit weight, gradation, and pozzolanic activity have on the flow properties and strength of various grouts. In addition to being used alone for road filling, fly ash has been used successfully in cement, lime, cement-clay, and cement-sand grouts.

I. INTRODUCTION

A. Purpose and Scope

The purpose of this manual is to encourage the utilization of fly ash in highway construction by acquainting highway engineers and contractors with the unique properties of this material and to provide the technical information necessary to safely and economically incorporate this by-product resource into highway design and construction.

The following utilizations for fly ash are covered in this manual:

- base courses and subbase courses
- subgrade modification
- embankments
- structural backfill
- grouting

The use of fly ash in cement or Portland cement concrete for construction of pavements or highway structures is not discussed.

General information is presented about the production, handling, and physical and chemical properties of fly ash which influence its behavior in highway applications. Each application is dealt with individually, with discussions pertaining to:

- factors affecting utilization
- case histories
- design criteria
- testing procedures
- construction procedures

The manual is directed to the engineer familiar with the geotechnical principles associated with highway design and to the contractor familiar with standard highway construction techniques. Suggested additional references are recommended where possible for the reader who wishes to obtain more detailed information on a particular subject.

The information contained in this manual was obtained from an extensive review of published literature from the United States and abroad, as well as correspondence, interviews, and site visits in both this country and Great Britain. The manual thus represents the State-of-the-Art of fly ash utilization in highway construction.

Every attempt has been made to make the design, testing, and construction recommendations as general as possible. Therefore, it will be necessary for the engineer or contractor to take into consideration local economic factors, climatic conditions, and other local considerations when adopting the procedures and recommendations contained herein.

B. Background

The need to recycle waste products has long been recognized in this country. Positive action in this direction is often delayed, however, until disposal of the waste products represents an environmental hazard or great economic cost, or natural materials become so scarce that attention must be directed to waste products as potential replacements. Such is the case with fly ash, a by-product of the coal combustion process. Although production and disposal of fly ash has occurred for most of this century, only the last decade has seen any serious effort directed to its utilization.

The development of this manual has thus evolved from a two-fold need:

1. To find large tonnage uses for fly ash, a by-product resource, in order to reduce the costs and environmental problems associated with its disposal;

2. To find reliable and economic substitutes for conventional highway materials which are dwindling in supply and escalating in cost, which require large quantities of energy in their production, and which might be better used in more critical applications.

The need to utilize fly ash becomes readily apparent when the volume of fly ash produced annually is considered. Preliminary figures for 1975 indicate the production of 42.3

million tons (38,000 Gg) of fly ash during that year.(5)* Of that quantity, only 4.5 million tons (4,100 Gg), or 11 percent, were utilized. The remainder had to be disposed of in ponds or stockpiles. The cost of disposing of this fly ash varies anywhere from $0.25 to $2.00 per ton (1970 figures),(2) a cost which is normally passed on to the consumer of electric power. Figure 1 indicates the rates of fly ash production and utilization during the past ten years. Figure 2 illustrates the geographic distribution of fly ash production on a state-by-state basis.

In view of the present conditions in the petroleum industry, it is anticipated that coal will continue to be the primary source of fuel for power generating plants for the rest of this century. Thus, fly ash production is expected to continue to increase over the next two decades. Since many power plants are close to urban or industrial areas, fly ash disposal could create serious land use problems in the near future. In addition, environmental regulations placed on the disposal areas have forced the adoption of more stringent methods for placement, drainage, and surface protection. These factors are all contributing to escalating disposal costs.

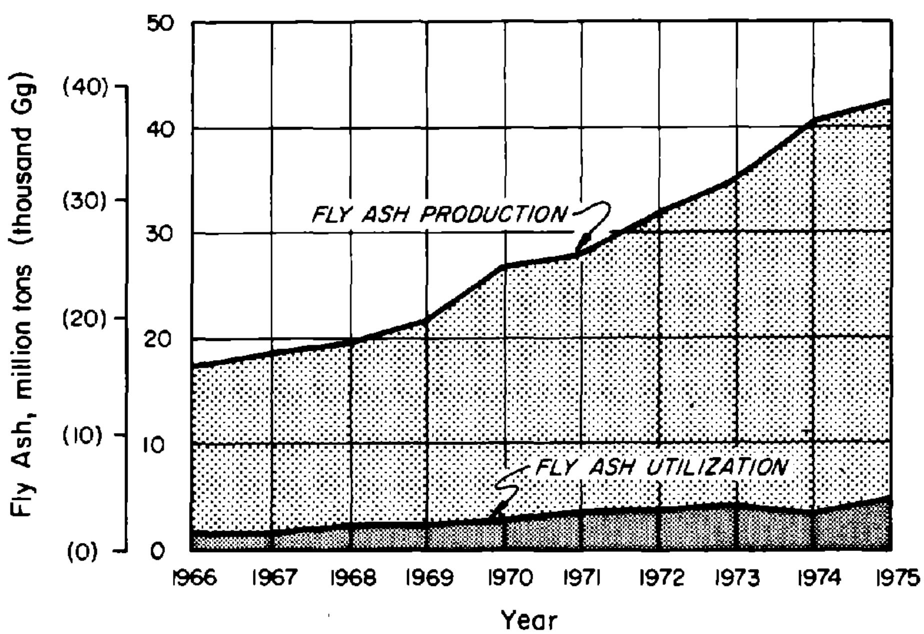

Figure 1. Fly ash production and utilization during the past ten years.(5)

*References can be found at the end of each section.

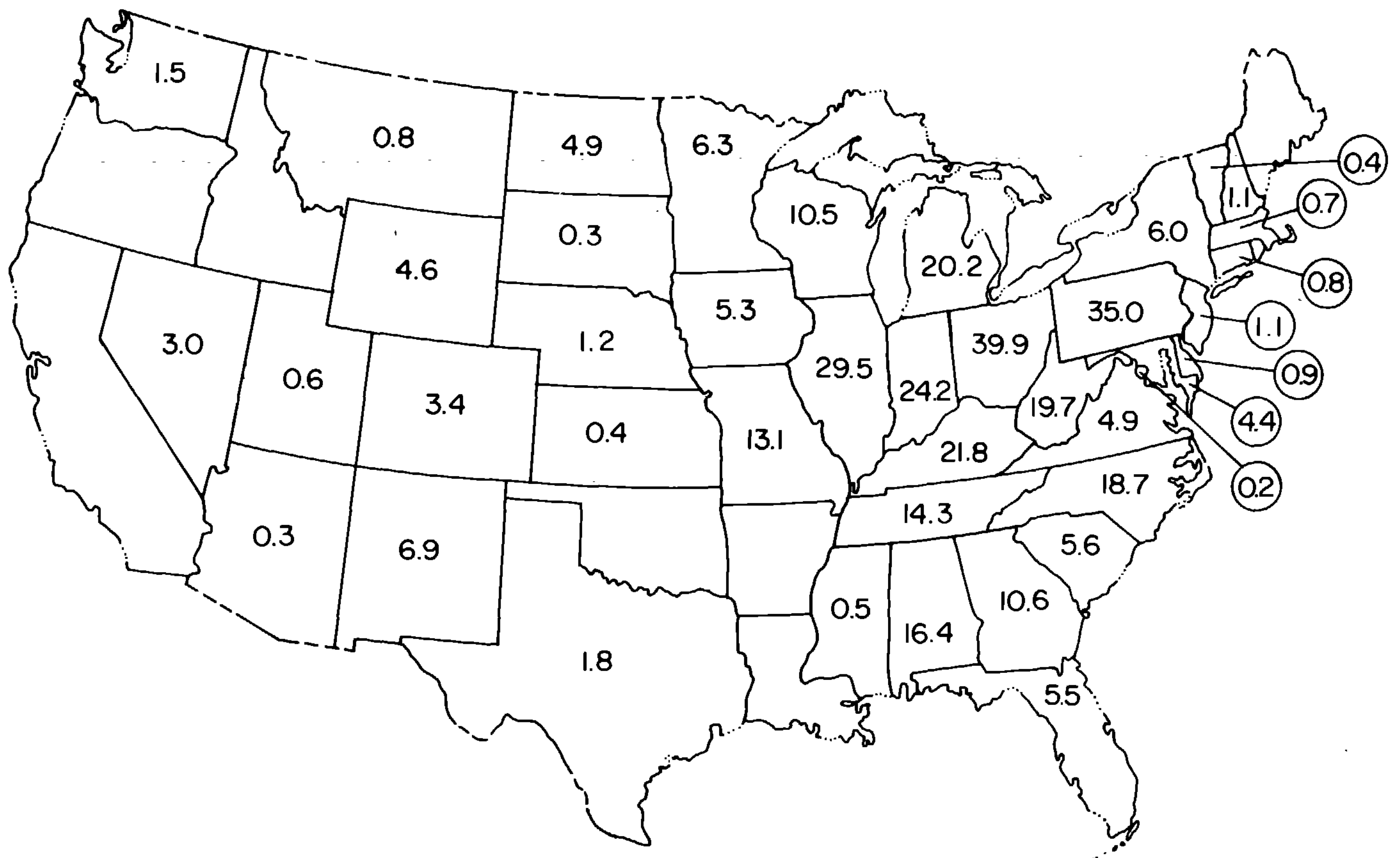

Figure 2. Fly ash production on a state-by-state basis - 1973 figures (in millions of tons, 1 ton = 907 kg).(6)

Two overriding concerns have materialized in the past three to five years which have prompted the search for new highway construction materials. The first is a shortage of conventional aggregates. The highway construction industry consumes over half of the annual production of aggregates.[3] However, there are certain geographic areas of the country where aggregates are in short supply. The same condition exists in most metropolitan areas. The reasons include lack of the raw materials, environmental and zoning regulations which prohibit mining and production of aggregates, and land use patterns which make aggregate deposits inaccessible. These factors, and others, combine to produce an escalation of aggregate costs, with a resultant increase in highway construction and maintenance material costs. There is thus a great need to find more economical replacements for conventional aggregates, thereby reducing highway costs and freeing aggregates for more critical uses where economic substitution of another material is not possible. It is natural, therefore, that attention is being focused on utilization of waste materials and by-products--in this case, fly ash. As a substitute for aggregates or aggregate products in pavements, fills, and grouts, fly ash has the advantage of low cost and assured availability in regions where coal-fired power plants are located.

Secondly, the recent "petroleum crisis" ushered in an era of energy consciousness which has precipitated a reevaluation of current methods and patterns of consuming energy. It is the opinion of many that energy demands should be taken into account, as well as economic costs, when an analysis of a project is performed. In terms of highway construction materials, the trend in the future will be toward the use of materials which require less energy input in their production, handling and placement. A recent energy study[1] revealed that the energy requirements for producing the materials for various asphaltic, crushed stone, and Portland cement concrete pavements ranged from 30 to 96 percent of the total energy required for production, handling, and placement of the various pavements. Since fly ash is a by-product of another process and must be collected and stored regardless of its potential for utilization, it is obvious that fly ash has a considerable energy advantage over these conventional materials. The energy required for its production is negligible in comparison to conventional materials and, since construction techniques for fly ash are identical in some cases and similar in others to those for conventional materials, the total energy requirement for fly ash applications could be expected to be less than that of other materials.

The necessity of encouraging large-tonnage utilization of fly ash becomes greater with each passing year. Utilization of fly ash has occurred in the cement, concrete and structural products industries. The successful utilization of fly ash in highway construction will require the following:

- o adoption by highway agencies of specifications for fly ash which are based on performance;
- o unified promotional efforts directed to various Federal, state, county and local agencies which are potential users of fly ash in highway construction;
- o construction of demonstration projects as visible examples of proper utilization of fly ash and as opportunities for further experimentation with construction methods and design techniques; and
- o continued research into testing and design procedures for fly ash and the development of new applications in the area of highway construction.

The fact remains that fly ash has been shown, in case after case, to be an economical and reliable construction material for various highway applications. It is hoped that the information presented in this manual will lead to the increased use of fly ash, and that successful experiences will ultimately dispel the "new material neurosis"(4) which so often accompanies attempts to introduce a heretofore non-conventional material to the construction industry.

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II. PRODUCTION, PROPERTIES, AND UTILIZATON OF FLY ASH

A. Production, Handling, and Storage

Fly ash is a by-product of the combustion of coal and is usually associated with power plants which burn fossil fuels. Fly ash is a very fine, light dust which is carried off in the stack gases from a boiler unit and collected by mechanical or electrostatic methods. It is derived primarily from rock detritus which collects in the fissures of coal seams,(9) and constitutes 8 to 14 percent of the weight of the coal.(10) Fly ash should not be confused with bottom ash, a granular by-product which drops to the bottom of the boiler unit and is occasionally mixed and stored with fly ash.

The quantity and quality of fly ash produced are a function of several factors. Coal source and method of production are perhaps most influential on the nature of the final product. Most data published on the nature of fly ash relate to fly ash produced from bituminous coals. Anthracitic coal ashes tend to be somewhat higher in carbon content,(2) whereas lignite and western coal ashes have considerably higher calcium oxide contents, all of which can greatly affect the utilization potential of a fly ash. The specifics of the chemical and physical characteristics of fly ash are discussed later in this section.

There are three main categories of boiler units in which fly ash is produced: stoker-fired units, cyclone furnace units, and pulverized coal-fired units.(10) Stoker-fired units usually produce varying amounts of coarse fly ash, depending on whether they are underfed and travelling grate stokers or spreader stokers. In cyclone units, only a small portion of the fly ash produced is released into the flue gases and collected. Most of the fly ash melts and becomes a slag at the bottom of the furnace In pulverized coal-fired units, finely pulverized coal is burned in suspension, and most of the fly ash produced enters the stream of flue gases and is removed by mechanical collectors or electrostatic precipitators. Pulverized coal-fired units are widely used in the utility industry.(10)

Several pollution control processes not yet in widespread use could change the nature of fly ash production in the future. Among these are coal fractionation and desulfurization processes.(10) The effect on the fly ash produced varies considerably, depending upon the specific method being used. Limestone and dolomite injection processes are under intense investigation, and widespread adoption of these processes will result in the production of a modified fly ash with characteristics significantly different from the bituminous fly ashes presently being produced.

After production, the nature of a fly ash can be further modified by the handling and storage techniques used. These techniques tend to vary within the utility industry and are a function of power plant design. Handling and storage techniques can be divided into two broad categories of dry and wet methods. Dry methods usually entail short-term storage of freshly produced fly ash in hoppers or extended storage in larger silos. Dry fly ash can be recovered from hoppers and silos by discharging directly through gates or doors into transport vehicles. Wet methods entail the addition of some quantity of water to the dry fly ash after collection, and subsequent disposal by usually one of two methods. Large amounts of water can be added to the fly ash to transport it in slurry form to settling ponds or lagoons, where the fly ash settles to the bottom and the excess water is then carried away. Very often, the fly ash is stored in the same lagoon with coarser bottom ash which has been removed from the bottom of the boiler unit. Dry fly ash can also be conditioned with smaller amounts of water to prevent it from dusting, then hauled by vehicles to a dry disposal area and stockpiled in large quantities. Stockpiled and ponded ash can be recovered by excavation. A single power plant may utilize more than one particular storage method.

B. Chemical and Physical Characteristics of Fly Ash

The chemical and physical composition of a fly ash is a function of several variables:[12]

1. coal source;

2. degree of coal pulverization;

3. design of boiler unit;

4. loading and firing conditions; and

5. handling and storage methods.

Thus, it is not surprising that a high degree of variability can occur in fly ashes, not only between power plants but within a single power plant. A change in any of the above factors can result in detectable changes in the fly ash produced. The degree to which any change affects the utilization potential of the fly ash is a function of the nature and degree of the change, and the particular application for which the fly ash might be utilized.

Fly ash is comprised of very fine particles, the majority of which are glassy spheres, and the remainder of which are crystalline matter and carbon.(4) Fly ash from bituminous coals contains large quantities of silica (SiO_2), alumina (Al_2O_3), and ferric oxide (Fe_2O_3), with smaller quantities of various oxides and alkalies. Carbon is also usually present in varying amounts. The range of chemical constituents for bituminous fly ashes is indicated. in Figure 3.

The chemical compositions of fly ashes from lignite coals, western coals, and fly ashes produced from limestone and dolomite injection processes are sometimes significantly different from those of bituminous fly ashes. Chemical compositions of typical lignite and modified fly ashes are compared in Table 1 to that for a typical bituminous fly ash.

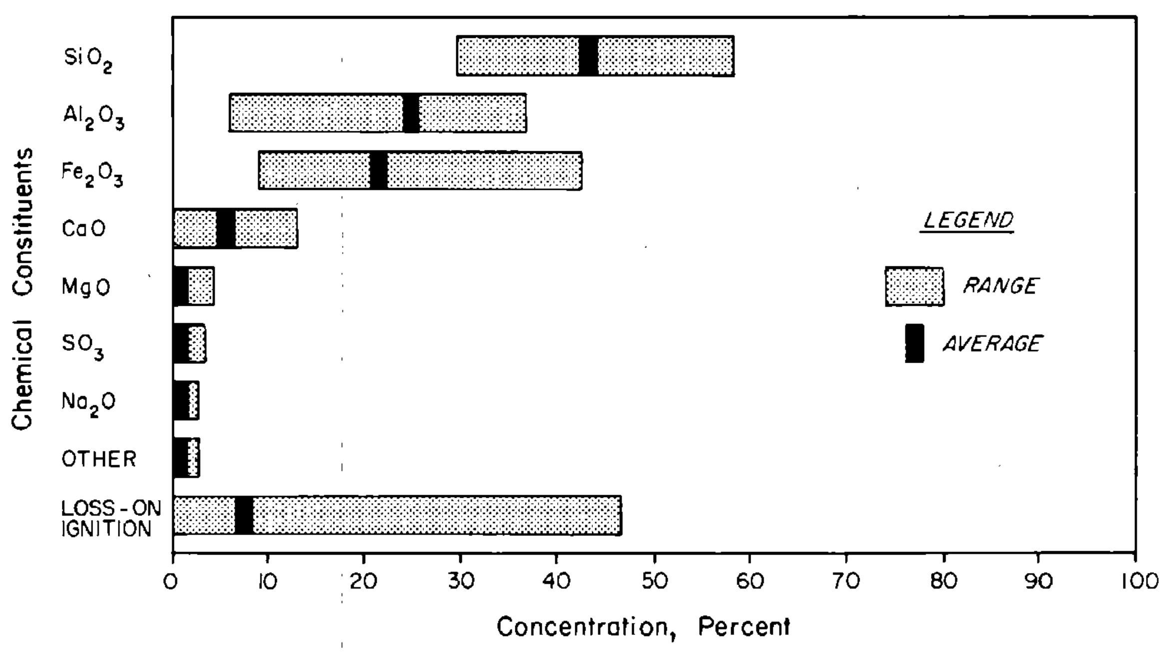

Figure 3. Range of chemical constituents in bituminous fly ashes.(8)

Table 1. Comparison of chemical compositions of typical bituminous, lignite, and modified fly ashes.[5]

CONSTITUENT	PERCENT BY WEIGHT			
	BITUMINOUS ASH	LIME MODIFIED ASH	DOLOMITE MODIFIED ASH	LIGNITE ASH
SiO_2	49.10	30.85	30.81	32.60
Al_2O_3	16.25	13.70	12.54	10.70
Fe_2O_3	22.31	11.59	10.72	10.00
TiO_2	1.09	0.68	0.42	0.56
CaO	4.48	33.58	17.90	18.00
MgO	1.00	1.49	14.77	7.31
Na_2O	0.05	1.12	0.72	0.87
K_2O	1.42	0.71	0.99	0.68
SO_3	0.73	2.20	8.09	2.60
C	2.21	1.12	1.76	0.11
L.O.I.*	2.55	1.03	1.95	0.62
H_2O soluble	2.51	22.11	20.39	8.55

*Loss-on-Ignition

The amount of calcium oxide (CaO), it can be seen, is very much influenced by the coal source and the use of limestone and dolomite injection processes. The carbon content, on the other hand, is more a function of the efficiency of the particular boiler unit and the fineness of the pulverized coal. Older units tend to produce higher carbon fly ash than newer, more efficient units.[4] The carbon content is usually measured as the percent of weight loss-on-ignition at high temperatures, usually 750°±50° C.

The chemical composition of a fly ash influences its color to a large degree. Fly ashes range in color from cream to dark brown or gray.[4] The cream color is usually produced by a high calcium oxide content, and gray to black by increasing quantities of carbon.

The specific gravity of most fly ashes falls within the range of 2.1 to 2.6.[15] The particle size of fly ash ranges from 1μ to 100μ in diameter for the glassy spheres, with an average of 7μ, and from 10μ to 300μ in diameter for the more angular carbon particles.[10] In terms of a typical soil grain-size analysis, most fly ash particles fall within the silt range, with small percentages in the fine sand and clay sizes.[2] Fly ash from mechanical collectors is normally coarser than fly ash collected by electrostatic precipitators. Since fly ash is often stored with coarser bottom ash in lagoons, ash recovered from lagoons is often of coarser gradation than hopper or silo fly ash. The range of typical grain-size distributions for fly ash is shown in Figure 4.

A more meaningful measure used to indicate the fineness of a fly ash is the Blaine Fineness. This usually ranges from 1700 cm^2/gm in fly ashes from mechanical collectors to 6400 cm^2/gm in fly ashes from electrostatic precipitators.[10,15] Another measure of fineness is the surface area, which is the product of the Blaine Fineness and specific gravity and is usually specified in terms of cm^2/cm^3. It is likely that the gradation and fineness are most influenced by the degree of pulverization of the coal.[13]

Water soluble content of bituminous fly ash ranges from 1 to 7 percent.[2,3,9,14] Lignite fly ash has a slightly higher water soluble content. Modified fly ashes from desulfurization processes, however, may have water soluble contents on the order of 20 percent or greater.[5] The leachate from fly ash is usually alkaline with a pH ranging from 6.2 to 11.5.[10] The leachate contains principally calcium and sulphate ions, with smaller quantities of magnesium, sodium, potassium, and silicate ions often present.[2] Part of the soluble calcium is accounted for by free lime. In bituminous fly ashes, the quantity of free lime varies from 0.03 to 0.73 percent, averaging about 0.3 percent.[2] The free lime content of modified fly ashes and fly ashes from some western and lignite coals is considerably higher.

The compacted dry densities of fly ash are commonly found to be in the range of 70 to 95 pcf (1120-1520 kg/m^3) when determined in accordance with AASHTO T 99-74.[8] Lower densities are very often associated with high carbon content. The moisture-density relationship for fly ash is similar to that for cohesive soils. That is, for a given compactive effort the dry density increases with increasing moisture content to a point of maximum dry density. As the moisture content continues to increase beyond an optimum value, dry density decreases. Hopper and silo fly ashes tend to have sharp, well-defined points of maximum dry density and optimum moisture content, with rapid decline in density values on either side of optimum moisture content. Fly ashes which have been exposed to large quantities of moisture, such as

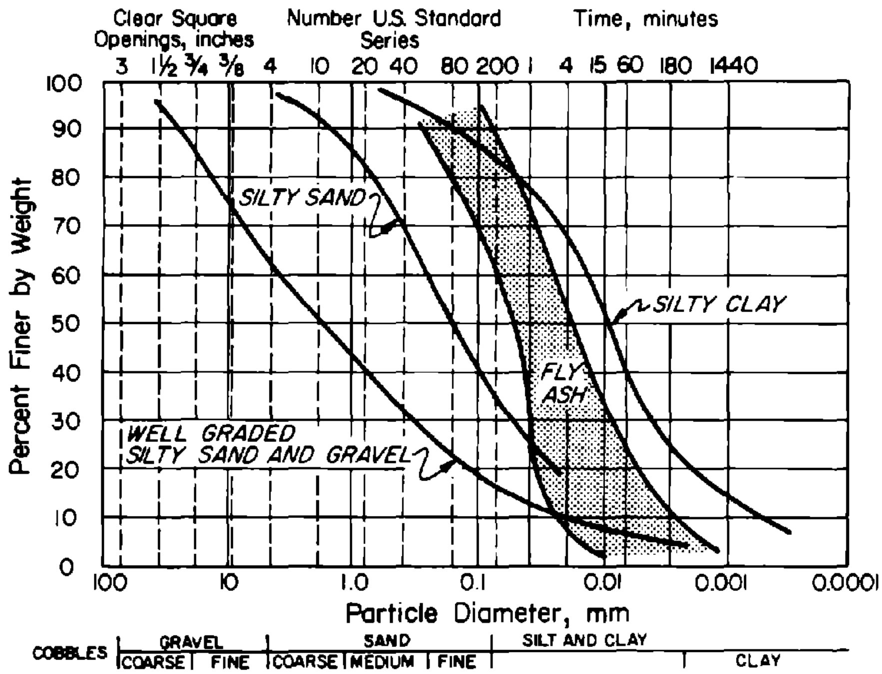

Figure 4. Range of typical fly ash grain sizes.(7)

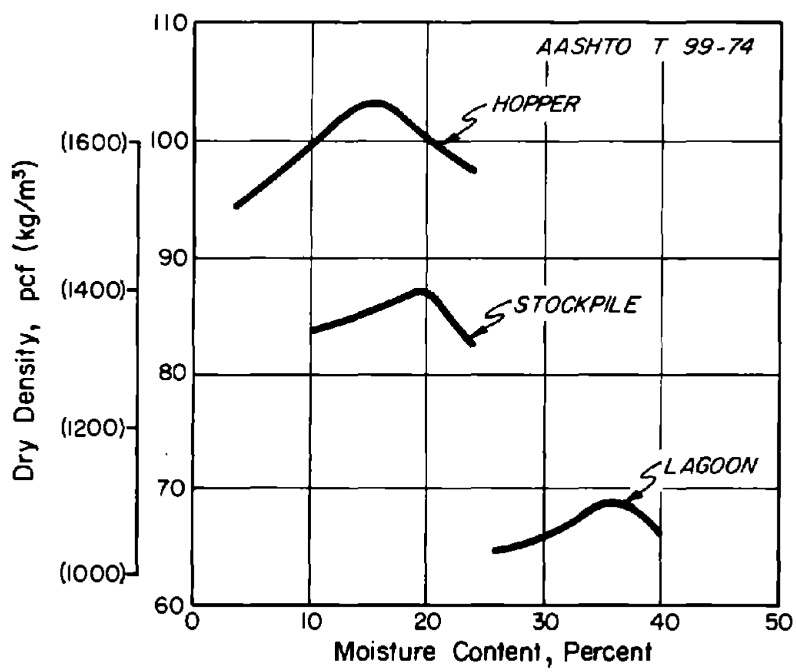

Figure 5. Moisture-density curves for some typical fly ashes.

lagoon ashes, tend to have flatter moisture-density curves, with little change in dry density occurring over a broad range of moisture contents. Maximum dry densitites of lagoon ashes tend to be lower and occur at higher moisture contents than those for hopper and silo fly ashes. Stockpiled fly ashes tend to have intermediate values of maximum dry density. Figure 5 illustrates the moisture-density curves of several different fly ashes.

C. Properties Affecting Utilization in Highway Construction

Although fly ash has been considered a waste product of the power-generating process, it has many properties which render it usable in a number of applications. For years, it has been used in the manufacture of cement, concrete, brick, and other structural products. Its potential for utilization in highway construction has been recognized by some but has not yet received widespread attention in this country. Recognition of the unique properties of fly ash which make it an ideal solution for certain engineering and construction problems as well as a logical substitute for more expensive natural materials which are dwindling in supply and escalating in cost will enhance the utilization of fly ash in highway applications.

The outstanding property which makes fly ash a viable engineering material is its pozzolanic nature. A pozzolan is a siliceous or alumino-siliceous material which is not cementitious in itself, but which, in finely divided form and in the presence of moisture, reacts with alkali and alkaline earth products to produce cementitious products.(1) The pozzolanic reaction between fly ash and lime can result in a material of substantial strength. Natural pozzolans, such as volcanic ash, have been highly regarded construction materials since ancient times. The ancient Roman buildings, for instance, were built from pozzolanic material, and many of the ruins still stand today. The fact that fly ash is a by-product of another manufacturing process and is readily available in many areas of the country enhances its potential for economic utilization as a modern day pozzolan.

The pozzolanic reaction between lime and fly ash can be produced in a number of ways. In lime-stabilization, lime is added directly to the fly ash, moisture is introduced, and the mixture is then compacted to facilitate the pozzolanic reaction. In cement-stabilization, cement is added to the fly ash in lieu of lime; the cement hydrates upon contact with moisture producing its own cementitious compounds as well as releasing certain amounts of lime which then react with the fly ash in a pozzolanic manner.[2] Certain types of fly ash also contain substantial quantities of free lime which react with other compounds in the fly ash upon addition of moisture to produce cementitious compounds without the addition of lime or cement. This third process is known as self-hardening of the fly ash.

The pozzolanic properties of fly ash have been employed in various phases of highway construction in this country and abroad. Fly ash stabilized with lime and/or cement has been used in base and subbase courses for roadway pavements. Lime- and/or cement-fly ash stabilization of soils and aggregates has been used for producing base and subbase course material for roadways and for subgrade improvement. Stabilized and unstabilized fly ash has been used for structural backfill and embankment material. Fly ash has also been incorporated into grout mixtures for void-filling and injection grouting below highway structures and pavements. Although not within the scope of this manual, fly ash has also been incorporated into Portland cement concrete pavements.

The pozzolanic reaction, although still not completely understood, involves the silica and alumina compounds in the fly ash, as well as any free lime which may be present. The extent and rate of the reaction is a function of several factors:[2,16,17]

1. quantity of stabilizer (free lime or cement);
2. total amount of silica (SiO_2) and alumina (Al_2O_3) in the fly ash;
3. presence of adequate moisture;
4. compacted density;
5. presence of carbon in the fly ash;
6. fineness of the fly ash;
7. temperature; and
8. age.

In the case of Items 1, 2, 4, 6, 7, and 8, it appears that the greater the quantity, value, or degree, the greater the pozzolonic reaction. Items, 1, 2, 4, and 8 appear to affect the ultimate degree of pozzolanic reaction, as measured by some convenient method, such as unconfined compressive strength; whereas Items 6 and 7 tend more to influence the rate at which the chemical reaction occurs rather than its magnitude. In the case of Item 3, an extreme in either direction adversely affects the reaction. In Item 5, quantities beyond a certain limit inhibit the reaction.

Based on the above factors, some general statements about the pozzolanic reactivity of various fly ashes are possible:

a. fly ashes with large amounts of free lime (as indicated by, although not equal to, the CaO content) tend to be very reactive and probably exhibit some degree of self-hardening. However, exposure to large amounts of moisture, as may occur in a lagoon or stockpile, could effectively reduce the free lime content since the free lime is water soluble and can be leached out of the fly ash over some period of time.

b. High carbon content can effectively inhibit any pozzolanic activity to the point where it may not be possible to economically utilize the fly ash in applications which depend upon the development of strength or durability through the pozzolanic reaction. A carbon content of 7 to 10 percent can be considered a reasonable upper limit for such applications.[6,11]

c. Fine fly ashes tend to be more reactive than coarser fly ashes.

d. Lagoon fly ashes may be less reactive than stockpiled fly ashes, and stockpiled fly ashes less reactive than hopper and silo fly ashes, if significant quantities of free lime have been leached out.

e. Fly ashes from less efficient boiler units, which tend to have high carbon contents, may be less reactive than fly ashes from more efficient units.

In addition to the pozzolanic nature of the fly ash, the relatively light unit weight of fly ash has made it a particularly suitable fill material over areas of weak subgrade where heavier fill materials could cause excessive settlement or failure in the weak soils. In addition, any self-hardening properties which a fly ash may possess tend to increase the fly ash's shear strength with time, rendering the fly ash a very stable material capable, in many cases, of sustaining substantial loads. The self-hardening phenomenon has also contributed to the negligible settlement which has occurred in many fly ash structural backfills behind bridge abutments, a location particularly susceptible to differential settlement problems.

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III. LIME- OR CEMENT-FLY ASH-AGGREGATE PAVEMENTS

A. Factors Affecting Utilization

Lime-fly ash-aggregate (LFA) mixtures are blends of mineral aggregate, lime, fly ash and water, combined in proper proportions which, when compacted, produce a dense mass. When compacted to a high relative density, and with reasonable curing conditions, these mixtures gradually harden to produce high quality paving materials with many unique and desirable properties. Cores with strengths greater than 3,000 psi (20,680 kPa) in compression have been obtained from a number of sites, but strengths ranging from 500 to 1,000 psi (3450-6890 kPa) after approximately a year in service are more typical.[2,4,8,9,13,19,21,22,27,28]

When used in pavement systems, LFA mixtures are placed in layers commonly referred to as base or subbase courses. A wearing surface must be applied to an LFA base course to protect the material from the abrasive effects of traffic, from weathering, and from water infiltration. LFA mixtures have provided satisfactory performance when used under either concrete or asphalt surfaces.[2,4,5,6,9]

The term LFA refers to a lime-fly ash-aggregate mixture. Portland cement, however, may be either partially or wholly substituted for the lime to accelerate the rate of strength gain of the mixtures, especially for late season construction. These mixtures are designated as LCFA. The substitution of cement for lime is not on a one-to-one basis, however, as most mixtures require more cement than lime to achieve the same ultimate results.

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

a) LFA and LCFA materials can be used for a wide range of pavement systems from low-volume roads to heavy-duty pavements. Appropriate design procedures and criteria are available for the entire range of pavement systems.

b) LFA and LCFA materials can be used as either base materials in flexible pavement systems or as subbase materials 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 inches (102-152 mm) of asphalt concrete for heavy-duty airfield pavements. Use of the seal coat should be limited to very low-volume roads, preferably to pavements carrying only low-speed vehicles.

c) The key to good performance with LFA and LCFA materials is good mix design with adequate quantities of lime and fly ash, and sound construction techniques. Thorough blending of the components and high relative in-place densities are also essential to good performance.

d) Durability is the most important single property in the performance of LFA and LCFA 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, as 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.

e) Procedures have been developed for establishing cut-off dates for late season construction with LFA and LCFA materials. The procedures outlined in Appendix A are based on the expected curing conditions, number of freeze-thaw cycles, and traffic conditions expected at the site. There have been a number of instances where LFA and LCFA materials have been placed after the last expected curing weather had passed, allowed to lie 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 materials.

f) High relative in-place density is critical for development of high-strength, highly durable materials. Since 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 results in shoving rather than densification of the materials. There are special problems of compaction near the edge of any pavement layer as, without lateral support, the material is likely to shove rather than densify under the compactor.

g) Excessive moisture combined with freezing and thawing and high concentration of salt are extreme environmental conditions to be avoided. If high ground water is present at the site, installation of subsurface drainage facilities is likely to provide substantial improvement in performance. Salt brine is detrimental to LFA and LCFA materials and should be drained from the pavement system as rapidly as possible.

B. Case Histories

Since the mid-1950's, the use of LFA has increased steadily to the point where well over a million tons of these mixtures were produced annually during the first half of the 1970's. As of 1975, LFA and LCFA materials have been approved for use by such agencies as the Federal Highway Administration and the Federal Aviation Administration, plus a number of state highway departments, including Illinois, Maryland, Michigan, New Jersey, North Dakota, Ohio, Pennsylvania, Virginia, and West Virginia. In addition, a number of other states have shown great interest in these materials as fly ash becomes more generally available in more areas of the country with the more widespread use of coal as an energy source.

There have been hundreds of applications of LFA and LCFA during the past quarter century. Only a few case histories are reported herein, however, to illustrate the magnitude and variety of uses for this material.

Early Construction Projects in Illinois and Pennsylvania

Some excellent examples of construction with LFA mixtures during the late 1950's and 1960's are Howard Street in Chicago, Illinois, constructed in 1958 and shown in Figure 6 and 7, and the O'Hare Field access road, constructed in 1966 and shown in Figures 8 and 9. The Howard Street project

Figure 6. Spreading LFA mixture on Howard Street.

Figure 7. Compacting LFA mixture on Howard Street.

Figure 8. View of completed LFA pavement at O'Hare Field.

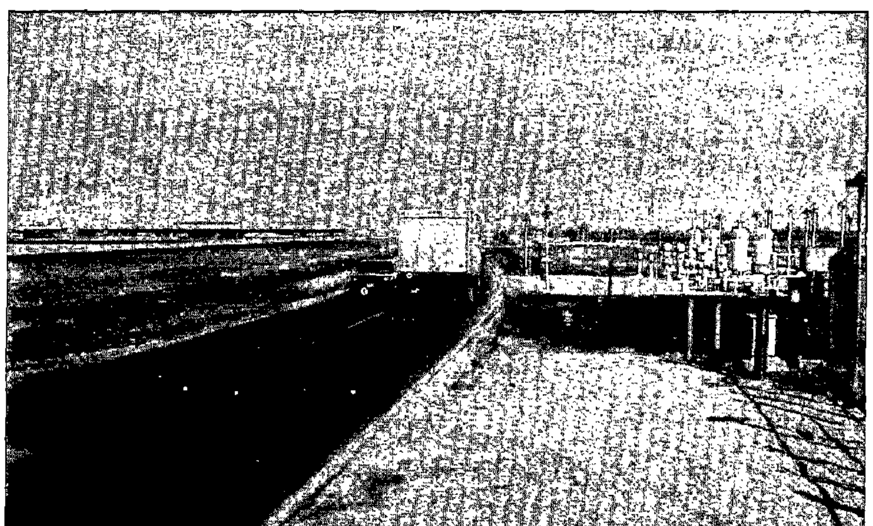

Figure 9. View of completed LFA pavement at O'Hare Field.

was constructed with county day labor, and a minimum of heavy equipment. The O'Hare service road carries primarily heavy trucks to service O'Hare Field. Both pavements have given excellent service.

Other early pavement projects include Hayford Street in Chicago, Illinois; streets and parking areas for the Eddystone Power Station of the Philadelphia Electric Company in Pennsylvania; and the truck entrance roads around the G. and W. H. Corson plant near Plymouth Meeting, Pennsylvania (near Philadelphia). These roads were all constructed in the mid-1950's, and were still giving excellent service 12 years later.[6] Insofar as is known, these pavements are still in service (1976), although maintenance has been performed, and some have been overlaid with asphalt concrete.

Runways, Taxiways and Aprons
Newark Airport, Newark, New Jersey

The largest single project involving the use of fly ash in base and subbase construction was the Newark Airport runways, taxiways, and aprons. In this job, approximately 2 million square yards (1.7 million m^2) of pavement were placed using LCFA ranging from 22 to 30 inches (559-762 mm) in thickness for the base and subbase layers. Typical mix proportions were 2.8 percent lime, 0.7 percent Portland cement, 12 to 14 percent fly ash, and the remainder hydraulic fill sand and coarse aggregate. More than 2 million tons (1.8 million Mg) of fly ash were used on this project.[29,36]

Figures 10 through 13 show the Newark airport in various stages of construction. Much of the pavement was placed in 1969 and 1970, and portions of the airport opened to traffic. These portions open to traffic have provided excellent service with only minor problems associated with other problems, such as subgrade settlement, improper construction techniques, and similar causes, which required only minor maintenance.

C. Constituents and Properties of LFA Mixtures

Lime

The term lime as used herein includes the various chemical and physical forms of quicklime, hydrated lime, and hydraulic lime. The most commonly used forms of lime for LFA mixtures are the monohydrated dolomitic and high calcium types.[2,24,27,34]

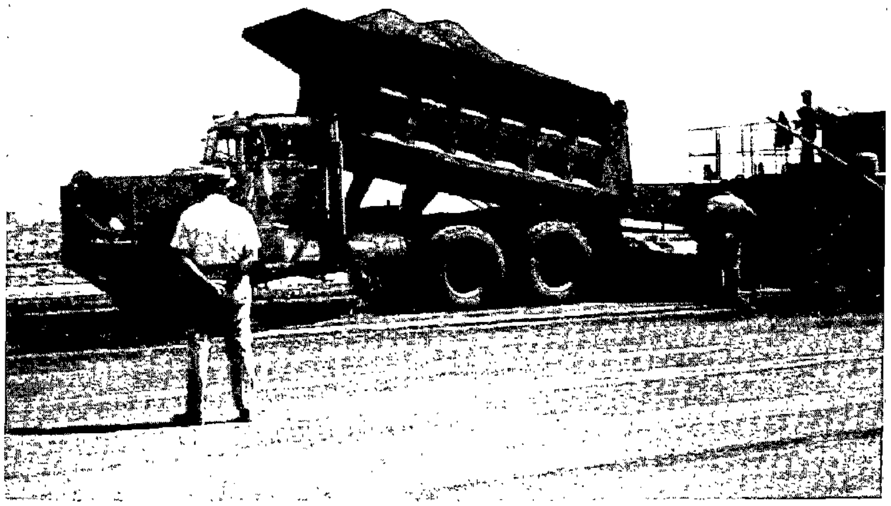

Figure 10. Tailgating LCFA mixture into paving machine at Newark Airport.

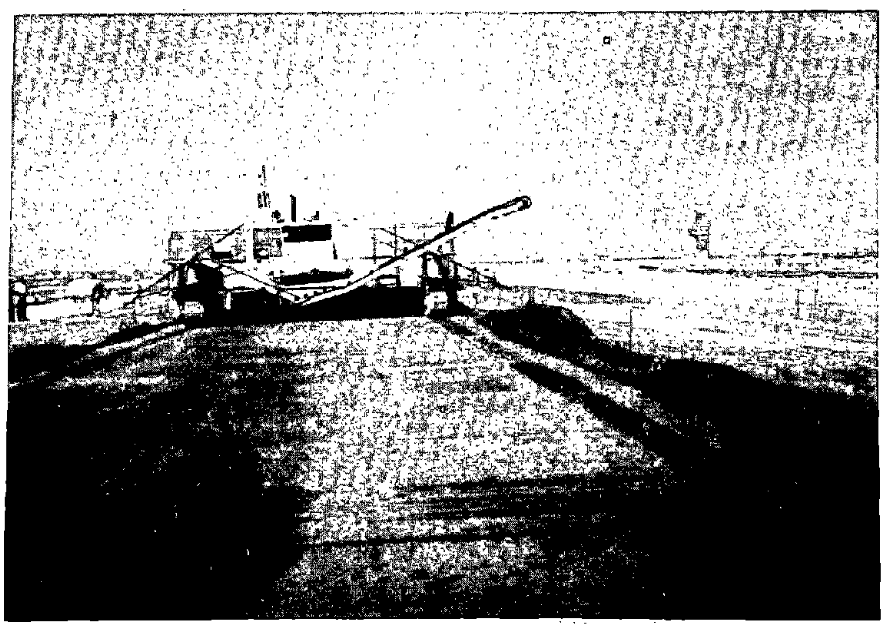

Figure 11. Spreading LCFA base course with paving machine at Newark Airport.

Figure 12. Spreading LCFA base course with motor patrol in confined areas at Newark Airport.

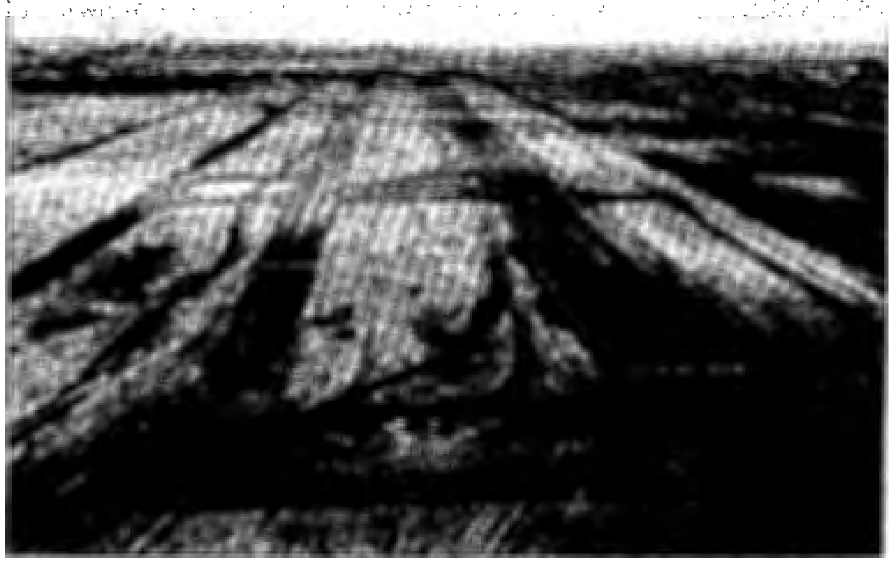

Figure 13. Overview of completed LCFA layers at Newark Airport.

Pozzolans

The most commonly used pozzolan today is fly ash. Not all fly ashes and pozzolans have the same chemical properties. To determine if a fly ash is of satisfactory quality for this application, it should be tested for pozzolanic reactivity in accordance with ASTM C 593.(21,25,26,28,33,35)

Aggregates

Aggregates which have been successfully used in LFA mixtures cover a wide range of types and gradations, including sands, gravels, crushed stones, and several types of slag.[2,4,14,15,16,17,19,23,24,31,36] 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 high density. Further, the aggregate should be free from deleterious organic or chemical substances which may interfere with the desired chemical reaction between the lime, fly ash, and water, and should consist of hard, durable particles, free from soft or disintegrated pieces.

Fine-grained aggregate mixtures have generally produced materials of greater durability than coarser-grained mixtures. However, mixtures with coarser aggregate gradations are generally more mechanically stable and may possess higher strengths at an early age. With time, however, mixtures with fine-grained aggregates may ultimately develop strengths which equal or exceed those obtained with coarser-grained aggregates. The key to the ultimate strength development lies in the lime-fly ash matrix rather than the aggregate.

Proportions

The relative proportions of each constituent used in specific LFA mixtures varies over a range. Effective mixtures have been prepared with lime contents as low as 2 percent, and as high as 8 percent, while fly ash contents vary from a low of 8 percent to a high of 36 percent.[7,9,19,36] Typical proportions are 2-1/2 to 4 percent lime and 10 to 15 percent fly ash. In some instances, small quantities (0.5 to 1.5 percent) of Type I Portland cement have been used to accelerate the initial rate of strength gain in LFA mixes. Mix design procedures have been developed and are discussed later in this section.

Pozzolanic reactions from which LFA mixtures derive their long-term strengths are influenced by many factors, including ingredient materials, proportions, processing, moisture content, field density, and curing conditions. The pozzolanic nature of fly ash and its reaction with lime is discussed in Section II, with details on how characteristics of the fly ash itself affects the reaction.

For an LFA mixture to develop its maximum possible strength, the ingredients must be thoroughly mixed. The time required to achieve a uniformly blended product depends upon the type and efficiency of the available mixing equipment, mixture proportions, and to some extent, on the ingredients themselves.

Curing conditions have a profound influence on the properties of LFA mixtures. Both curing time and temperature greatly affect the strength and durability of "hardened" mixtures.

Because of the combined effects of time and temperature on the strength development of the LFA mixtures, it is difficult to specify combinations of curing conditions which simulate field conditions. One method of taking into account the combined effects of temperature and time is to combine the two variables into a single variable called a degree-day. The degree-day concept is detailed in Reference 20 and in Appendix A.

While low curing temperatures retard the reaction process of LFA mixtures and almost entirely stop the reaction below 40° F (4° C), reduced temperatures or even freezing of the mixtures have no apparent permanent detrimental effect on the chemical properties of the constituents.[8,9] Although these materials are subjected to a significant number of freeze-thaw cycles in the field during the winter months, increases in strength are again developed with rising temperatures during the subsequent spring and summer months.

Under acceptable curing conditions, chemical reactions in LFA mixtures continue as long as sufficient lime and fly ash are available to react. Cores taken in chronological order from pavements over a 10-year period indicate a continuing development in the strength of the mixture with time. This continuing reaction process can manifest itself in a phenomenon called autogenous healing, which is one of the unique properties of LFA mixtures,[8,9,12] and is discussed later in this section. There are a number of recorded cases where distressed areas caused by improper loading of LFA pavements during early life have actually healed over with time. This can only occur, however, if there are sufficient quantities of lime and fly ash available to provide the necessary reaction components.

Admixtures

In an effort to accelerate development of early strength and improve the short-term durability characteristics of LFA mixtures, and thereby permit extension of the construction period later into the fall, admixtures have been added to accelerate or complement the lime-fly ash reactions. Most of the work in this area has been with chemicals in liquid suspension or in powdered form.

Portland cement is an effective admixture for use in LFA mixtures. The early strength development associated with hydration of Portland cement complements the slower strength development associated with some lime-fly ash reactions.[29,30,36,37]

Certain other admixtures (e.g., water reducing agents) may also give beneficial results. However, the use of many admixtures may not be feasible due to handling problems and prohibitive costs.

Compressive Strength

Properly designed mixtures compacted to a high relative density and properly cured may ultimately develop compressive strengths well in excess of 3,000 psi (20,680 kPa). Materials cured for seven days at 100° F (38° C) normally develop compressive strengths in the range of 500 to 1,000 psi (3450-6890 kPa). These same materials are likely to develop compressive strengths in excess of 1,500 psi (10,340 kPa) after one or two years of service.

Flexural Strength

LFA mixtures are significantly stronger in compression than in tension. Thus, the tensile strength is also an indicator of its quality. However, pure tensile strength is difficult to measure in these mixtures. An effective alternate method of evaluating the composite tensile and compressive capacity is through a determination of the flexural strength. Although it can be determined directly from tests, most agencies estimate the flexural strength by taking a ratio of the material's compressive strength. The ratio of flexural to compressive strength for most LFA mixtures is between 0.18 and 0.25. An average value of 20 percent of the compressive strength is a good, conservative, engineering estimate of the flexural strength of LFA mixtures.[2]

Durability

Durability is a measure of a material's ability to perform in an unfavorable environment. Properly designed LFA mixtures can be produced to meet durability criteria for high quality base materials. Several methods for evaluating the durability of LFA mixtures have been developed.[2,4,6,13,14,31,32]

Bending Resistance

The stiffness of LFA mixtures is usually expressed in terms of their moduli of elasticity (E). Typical E values for LFA mixtures range from 0.5×10^6 to 2.5×10^6 psi (3.4×10^6 - 17.2×10^6 kPa). Specific values depend on whether a tangent modulus or secant modulus is used. The expected range of E values for a specific LFA mixture is a function of several factors, in particular, aggregate characteristics (particle hardness and gradation), degree of compaction, and extent of curing of the mixture.[2,5,6,16,18]

Autogenous Healing

A unique characteristic of LFA mixtures is their inherent ability to heal or recement across cracks by a self-generating mechanism. This phenomenon is known as autogenous healing. The degree to which autogenous healing occurs is dependent upon many factors, including:

- the age at which the mixture cracks;
- the degree of contact of the fractured surfaces;
- the curing conditions;
- the availability of reaction products (lime and fly ash); and
- moisture conditions.

Because of the autogenous healing property, LFA mixtures are less susceptible to deterioration under repeated loading and are more resilient to attacks by the elements than other materials which do not possess this property.[2,3,12,14,31,32]

Fatigue

Lime-fly ash-aggregate mixtures, like all paving materials, can fail under repeated loading at stress levels considerably less than the stress required to cause failure of the material when loaded to failure in a single load application. Because of autogenous healing characteristics, however, LFA mixtures are less susceptible to failure by fatigue than most other paving materials. This is due to the healing process which provides a greater curing effect than the damage being caused by the repeated loads.[1,2] Unless fatigue failure occurs during the first few days of loading, it is not normally a factor in the performance of these pavements.

Poisson's Ratio

The Poisson's ratio of a material usually varies somewhat with the intensity of the applied stress. For LFA mixtures, however, this ratio usually remains relatively constant at a value of about 0.08 at stress levels below approximately 60 percent of ultimate and then increases at an increasing rate with the stress level to a value of about 0.3 at failure.[2,9] For most calculations, Poisson's ratio for LFA mixtures can be taken as between 0.10 and 0.15 without appreciable error.

Coefficient of Thermal Expansion

Hardened LFA materials, like all stabilized paving materials, are subject to dimensional changes with changing temperatures. The coefficient of thermal expansion of LFA mixtures is influenced primarily by the aggregates and the moisture content of the material. Typical values for the coefficient are about the same as for concrete at the same moisture content (approximately 6 x 10^{-6} inches per inch per degree Fahrenheit).[2,8,9,23]

D. Mix Design Concepts

Strength and Durability Criteria

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

Minimum cured compressive strength and maximum weight loss criteria are specified by the Illinois Department of Transportation and the Federal Aviation Administration as shown in Table 2. The Pennsylvania Department of Transportation has a durability requirement, but not a strength requirement. ASTM C 593 specifies a minimum cured compressive strength, and the vacuum saturation strength durability requirement to be incorporated into ASTM C 593 specifies a minimum vacuum saturation strength of 400 psi (2760 kPa) and replaces the maximum weight loss criteria previously specified. The Illinois Department of Transportation is also currently considering a vacuum saturation strength requirement.

Table 2. Specified design criteria for LFA and LCFA mixtures.

AGENCY	MINIMUM COMPRESSIVE STRENGTH psi (kPa)	MAXIMUM WEIGHT LOSS[1] %
ASTM C 593	400 (2760)[2]	-
Illinois Department of Transportation	400 (2760)	10
Ohio Department of Transportation	400 (2760)	10
Pennsylvania Department of Transportation	not specified	14
Federal Aviation Administration	400 (2760)	14

[1]After 12 cycles of freeze thaw.

[2]Minimum compressive strength after vacuum saturation test, and no weight loss. Criteria has been approved by ASTM but had not been published at the time of this report.

Thompson and Dempsey(31) 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 base 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 for the purpose of establishing stabilized mixture durability criteria.

Mix Design Procedures

The objective of the mixture design procedures is to develop the proper proportions of lime (cement), fly ash and aggregate. The design mixture must: 1) possess adequate strength and durability for its designated use, (2) be easily placed and compacted, and (3) be economical.

For a given set of component materials (lime, cement, fly ash, and aggregates) the factors that can be varied are the lime to fly ash ratio and the ratio of lime plus fly ash to the aggregate fraction. If cement is used with the lime, the ratio of lime to cement is also a variable. It is often more economical to blend aggregates from several sources to achieve a blend which gives superior performance than to use just one aggregate source and vary the binder components.[4,36]

The quality of LFA and LCFA mixtures, as measured by their strength and durability, is closely related to the quality of the cementitious matrix in the mixture. This matrix can be defined as the lime plus the fly ash and that portion of the aggregate finer than the number 4 sieve. Only if there is sufficient matrix material to "float" the coarser aggregate fraction is it possible to achieve a high compacted density which is essential to good strength and durability of the mixture.[4] In general, the more uniform the particle-size distribution of the aggregate, the lower the quantity of lime plus fly ash needed to achieve a highly compacted density in the matrix. Care must be taken, however, that the proportion of lime and fly ash in the matrix is sufficient to provide a good chemical reaction.[4] Also, sand aggregates with single-sized particles and sands devoid of minus 200-sized particles may require high fly ash content to serve as filler as well as a pozzolan in the mixture.[4,10,29,36]

Figures 14 and 15 illustrate the variation of density and compressive strength with lime plus fly ash contents for both coarse- and fine-grained aggregates. To achieve a quality mixture, it is necessary that the amount of lime plus fly ash be slightly in excess (2-3 percent) of that required for maximum dry density. As indicated earlier, poorly graded materials, such as the Plainfield sand in Figure 15, require a higher lime plus fly ash content because of the volume of voids to be filled.

The proper proportions of lime to fly ash, or lime to cement to fly ash, must be based on laboratory mix design data. These ratios do not remain constant, but are a function of the aggregate and fly ash properties and the rate of strength development desired in the mixture. Lime to fly ash ratios of from 1:2 to 1:7 have been evaluated and found acceptable,[10] but most mixtures have a ratio of about 1:3 or 1:4 for reasons of economy and quality.

After the lime plus fly ash to aggregate ratio has been determined, the mixture should be evaluated and adjusted for quality by changing the lime to fly ash or lime to cement to fly ash ratios. This is done by preparing trial mixes, curing them for prescribed periods of time at a prescribed

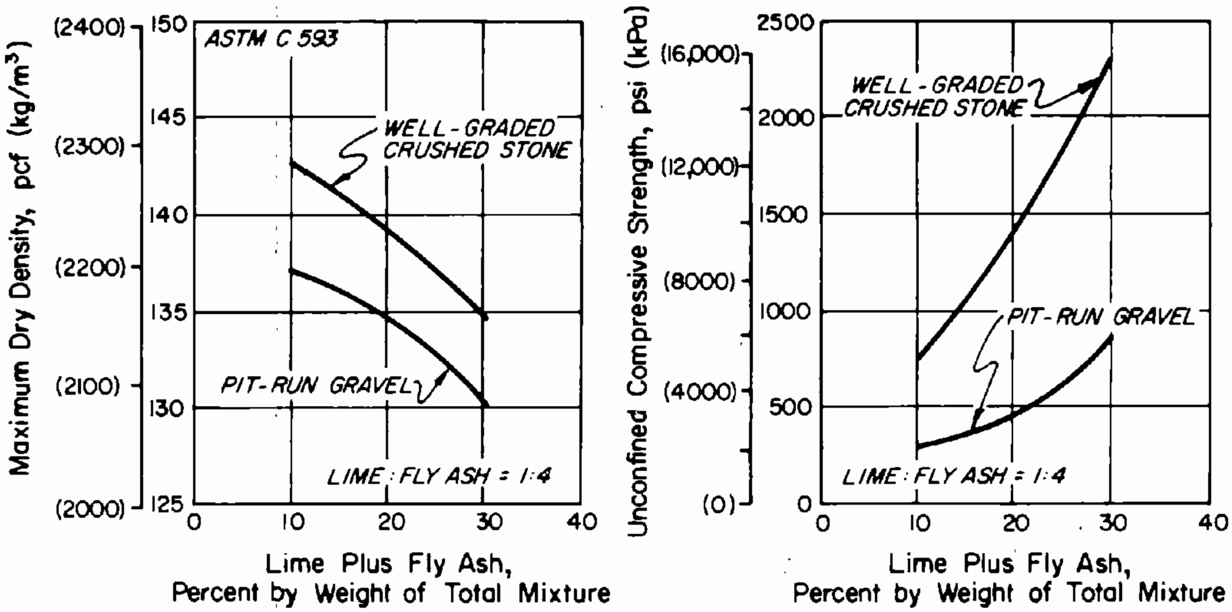

Figure 14. Variation of maximum density and unconfined compressive strength with lime plus fly ash content for coarse-grained aggregates.

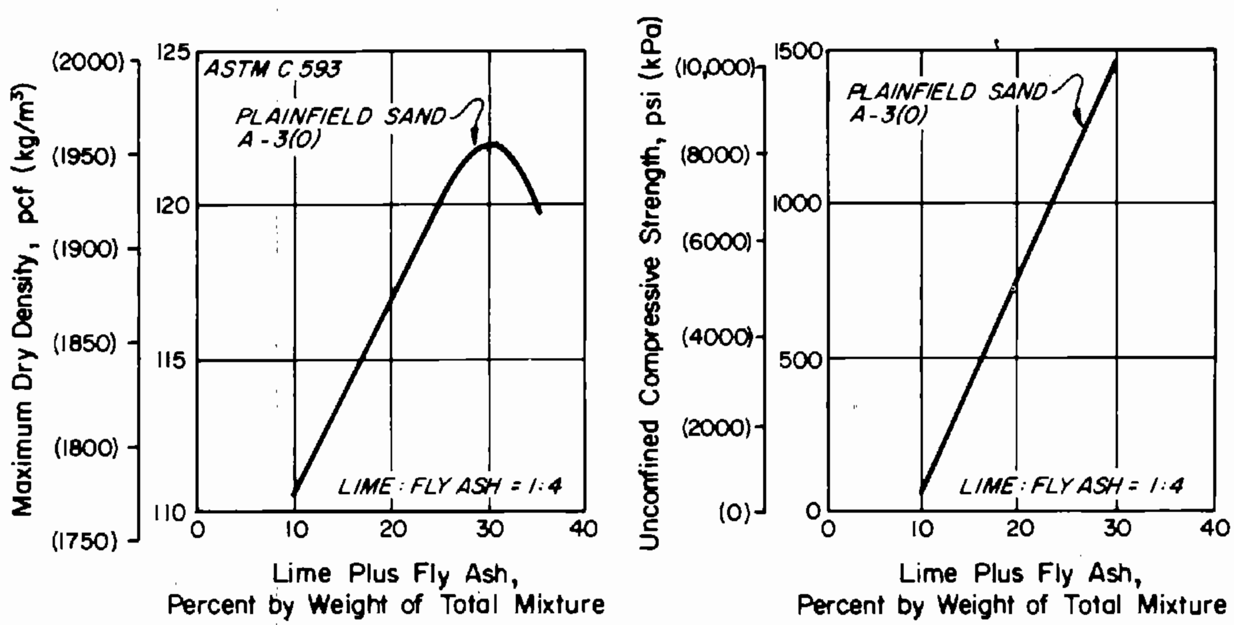

Figure 15. Variation of maximum density and unconfined compressive strength with lime plus fly ash content for fine-grained aggregate.

temperature (ASTM C 593 specifies 7 days at 100° F or 38° C), and testing for strength, durability, and rate of strength development. This latter requires curing at various temperatures for varying time periods. Durability requirements for these materials are given in ASTM C 593, as modified in 1975.

When the lime, cement, and fly ash requirements have been established, the designated mix must be adjusted to compensate for construction variability. The amount of adjustment needed is related to the level of quality control provided by the producer. For typical operations, the lime plus fly ash content should be increased by about two percent, and the lime content by about 1/2 percent.(11)

In some instances, a less structured approach to mix design is used, and typical mixture proportions are evaluated for adequacy and quality. As a guide to selecting appropriate component ratios, the four mixtures shown in Table 3 have provided highly serviceable mixtures for normal construction operations.

Laboratory Testing Program

1. Moisture-Density Relationship

Moisture-density tests are conducted in the usual manner as described in AASHTO T 180-74 with the exception of the compactive effort used. In Table 4, several of the different compactive efforts in common use are summarized. In each case, 4.0-inch (102 mm) diameter by 4.6-inch (117 mm) high, 1/30 cubic foot (9.4×10^{-4} m^3) molds are used.

It is important to note that compacted density has a very significant effect on the cured strength of LFA and LCFA mixtures. Strength or durability criteria based on one compactive effort cannot be applied to mixtures prepared in accordance with procedures using other compactive efforts.

2. Compressive Strength Tests

Standard Proctor-sized [4.0-inch (102 mm) diameter by 4.6-inch (117 mm) high] specimens are the most commonly used to evaluate the compressive strength of cured LFA or LCFA mixtures. Aggregate particles larger than 3/4 inch (19 mm) are normally scalped from the aggregates and discarded. For fine-grained aggregate mixtures, such as those containing fine sand, 2-inch (51 mm) diameter by 4-inch (102 mm) high

Table 3. Typical LFA mixtures.

	MIX AGGREGATE			
	CRUSHED STONE	GRAVEL	SAND	SLAG
MIX COMPONENT	(a) MIX PROPORTIONS - PERCENT BY WEIGHT[1]			
Aggregate	82 - 87-1/2	77 - 87-1/2	65 - 82	60 - 82
Fly Ash	10 - 14	10 - 18	15 - 30	15 - 35
Lime[2]	2-1/2 - 4	2-1/2 - 5	3 - 5	3 - 5
SIEVE SIZE	(b) AGGREGATE GRADATION - PERCENT PASSING			
1"	100	100	100	100
3/4"	90 - 100	90 - 100	100	100
1/2"	60 - 85	60 - 85	100	100
#4	50 - 75	50 - 75	90 - 100	90 - 100
#40	10 - 20	7 - 15	20 - 40	10 - 40
#100	2 - 5	3 - 6	0 - 3	0 - 2

[1]Based on total mix dry weight. [2]Lime or lime plus cement at a 3:1 ratio.

Table 4. Specified compactive efforts for LFA and LCFA mixtures.

AGENCY	PROCEDURE DESIGNATION	COMPACTIVE EFFORT[1]
	ASTM C 593	10/18/3/25
Illinois Department of Transportation	-	10/18/3/25
Ohio Department of Transportation	ASTM C 593	10/18/3/25
Pennsylvania Department of Transportation	PTM 106	5.5/12/3/25
Federal Aviation Administration	FAA T 611	10/18/5/25

[1]Hammer weight (lbs)/height of drop (inch)/no. of layers/blows per layer.

specimens have also been used, but there is difficulty in correlating the results from the two sizes of specimens.

It is essential to maintain a closely controlled environment during the curing of LFA and LCFA mixtures as both time and temperature have a profound effect on the strength and durability of these mixtures. Curing conditions (time in days and curing temperature) should always be specified along with the strength data. The standard curing conditions for these materials are seven days and 100° F (38° C), but for evaluation of the rate of strength development other times and temperatures are specified, such as 28 days at 70° F (21° C), 7 days at 50° F (10° C), 14 days at 72° F (22° C), and 2 days at 130° F (54° C). The method for converting various times and temperatures to equivalent degree-days is explained in Appendix A.

3. Durability Tests

Three procedures have been predominantly utilized for evaluating the freeze-thaw durability of LFA and LCFA mixtures. The freeze-thaw brushing procedure formerly included in ASTM C 593 is basically modeled after the soil-cement procedure (AASHTO T 136-70). Thompson and Dempsey (31) have indicated that the temperature conditions utilized 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 therms of basic engineering properties (strength, stiffness, etc.).

Dempsey and Thompson[14] developed automatic freeze-thaw testing equipment which accurately simulates field conditions. Compressive strength after freeze-thaw cycling (5 or 10 cycles) is used to characterize LFA and LCFA mixture durability. Details of the test procedure are presented in Reference 14.

The vacuum saturation test procedure proposed by Dempsey and Thompson[15] is a rapid technique (approximately one hour). The justification for using the vacuum saturation procedure is the excellent correlation between the compressive strengths of vacuum saturation specimens and freeze-thaw (Dempsey-Thompson technique) specimens. Details of the procedure are presented in Reference 15. The revision of ASTM C 593 currently approved includes the use of the vacuum saturation for durability evaluation purposes.

E. Thickness Design

Thickness design of pavements with LFA and LCFA mixtures has been based on both the equivalency concepts of pavement performance developed from the AASHTO Road Test (R-1) and on structural design considerations.[2,R-2,R-3] For those situations where considerable experience is available on performance of these and similar materials as a function of pavement thickness, the equivalency approach is adequate and valid. For those situations where this experience is not available, the more rigorous approach using theoretical analyses may be required either in lieu of, or in conjunction with, the equivalency approach. A brief description of each approach is given below.

Equivalency Method

As shown in Reference R-1, the structural capacity of a layered pavement system is given by the relationship

$$SN = a_1D_1 + a_2D_2 + a_3D_3$$

where

> SN is the structural number or structural capacity of the pavement system;
>
> D_1, D_2, and D_3 are the thicknesses of the surface, base, and subbase respectively; and
>
> a_1, a_2, and a_3 are the equivalency values or material coefficients often referred to as structural coefficients.

The above equation indicates that, to achieve a specified structural capacity, there is a linear, inverse relationship between the structural coefficient of the material in a layer and the layer thickness. The required structural number for a pavement is a function of the subgrade conditions, expected traffic, environmental conditions, and level of performance required. The structural number of the specific conditions can be determined from nomographs in Reference R-1.

Some typical values of a_2 for LFA or LCFA mixtures used by several states are shown in Table 5. The values shown for a_2 compare with values of 0.13 to 0.14 for crushed stone base and 0.30 to 0.35 for good quality bituminous stabilized base materials (black base).

Ahlberg and Barenberg[2] suggest a range of values for a_2 based on the quality of the material. These are shown in Table 6. LFA or LCFA mixtures with compressive strengths of less than 400 psi (2760 kPa) after curing seven days at 100° F (38° C) are not recommended unless thoroughly evaluated for durability. Some recent field tests indicate that the value of a_2 for the high quality LFA mixtures should be increased to as high as 0.50.[16,18] These higher recommended values have not been verified by long-term performance data, however.

When using the AASHTO equivalency approach to design, it is critical to keep in mind that, in addition to the equivalent thickness values, specified minimum thicknesses must also be adhered to. These minimum values are based on the thicknesses required to support the heaviest anticipated load without structural damage to the LFA or LCFA layer. The recommended minimum thicknesses are listed in Table 7.

Table 5. Typical a_2 values for LFA and LCFA mixtures.

STATE	COEFFICIENT a_2
Illinois	0.28
Pennsylvania	0.30
Ohio	0.28
Michigan	same as black base

Table 6. Recommended a_2 values for LFA and LCFA mixtures.

QUALITY OF LFA	COMPRESSIVE STRENGTH (7 days @ 100° F) psi (kPa)	RECOMMENDED STRUCTURAL COEFFICIENT a_2
High	Greater than 1000 (6900)	0.34
Average	650-1000 (4480-6900)	0.28
Low	400-650 (2760-4480)	0.28

Table 7. Recommended minimum thicknesses for LFA and LCFA base and asphaltic concrete surface.

DESIGNATED PAVEMENT USE	RECOMMENDED MINIMUM THICKNESS	
	ASPHALTIC CONCRETE SURFACE, AC inches (mm)	LFA, LCFA BASE inches (mm)
Parking Facilities:		
Autos and light commercial only	Surface Treatment or 1-1/2 (38)	5 (127)
Passenger cars and medium truck traffic	1-1/2 (38)	5 (152)
Channelized truck[1]	2 (51)	7- 8 (178-203)[2]
Commercial truck	3 (76)	8-10 (203-254)[2]
Residential Streets	1-1/2 (38)	6 (152)
Feeder Streets	2 (51)	7 (178)
Secondary Roads	2 (51)	8 (203)
Primary Roads	3 (76)	10 (254)

[1]Delivery vehicles, etc., within shopping center.

[2]These minimum thicknesses should be validated using the structural analysis procedures described in this Section.

Structural Design Method

For those situations where experience and performance data are not available, structural thickness design should be based on the anticipated strength of the LFA or LCFA mixtures at the time of loading. Because of the continuing strength gain characteristics of the mixtures, fatigue produced by the total number of loads applied over the life of the pavement is not normally a factor.(2,3) The number of load repetitions applied at an early age, however, may well be a design consideration.(2) Particulary for pavements constructed with LFA and LCFA mixtures late in the construction season, the number of load applications during the first winter is likely to be the critical loading condition. Thus, the pavement structure must have sufficient capacity, as determined by the strength and thickness of the LFA or LCFA layers, to carry the anticipated traffic during the first winter season without significant damage to the pavements.

The structural capacity of pavements with LFA and LCFA materials is normally calculated from the layer thicknesses and material properties using the Westergaard slab theory,(R-3) the elastic layered system theory,(R-4) or Meyerhof's Ultimate Load theory.(R-2) The most reliable results are usually achieved when a combination of these theories are applied and the results from the various theories reconciled.

Procedures have not been standardized for using the more theoretical methods of analysis as a basis for pavement thickness design with LFA and LCFA mixtures. The basic approach is to analyze the pavement system using the appropriate theory and to compare the calculated stresses with the anticipated strength of the material using some appropriate factor of safety.

The most formalized of the analytical design procedures is presented in Reference 2 and is based on the ultimate strength theory. The ultimate load capacity of a slab when loaded near a crack or near the pavement edge is, according to Meyerhof:(R-2)

$$P_u = \frac{(\pi + 4)}{6} \cdot \frac{f_y h^2}{(1-\frac{2a}{3L})}$$

where

P_u is the ultimate load;

f_y is the yield strength of the LFA or LCFA;

h is the effective thickness of the LFA or LCFA;

a is the radius of the equivalent circular loaded area; and

L is the radius of relative stiffness of the slab as given by

$$L = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}}$$

where

E is Young's modulus of elasticity for the LFA or LCFA;

μ is Poisson's ratio for the LFA or LCFA; and

k is the modulus of subgrade reaction.

Figure 16 is a nomograph which can be used to determine the appropriate thickness of LFA or LCFA for edge loading conditions. By applying an appropriate factor of safety to the ultimate load, P_u, the design axle load, P_o, can be calculated. The design axle load is used in conjunction with the modulus of rupture, f_b, to determine the thickness of the LFA or LCFA layer required to carry the design axle load.

The most critical feature of this design procedure is the selection of the appropriate factor of safety. Ahlberg and Barenberg(2) recommend that the factor of safety vary from 1.3 to 3.0 and be a function of the expected traffic early in the pavement life. While these values may appear low to those not familiar with these materials, it must be kept in mind that these materials gain strength with additional curing so that a pavement which has a factor of safety of 2.0, for example, when the initial load is applied may well have a factor of safety of 3.0 or greater after 30 days. Barenberg(6) reviewed the performance of 18 pavements after several years of service and concluded that the indicated range for the factor of safety is adequate for providing reliable pavement designs.

In applying the analytical procedures to the design of LFA and LCFA pavements, the effects of the surface layer are usually ignored. The minimum surface thicknesses given in Table 7 are applicable in these cases as well.

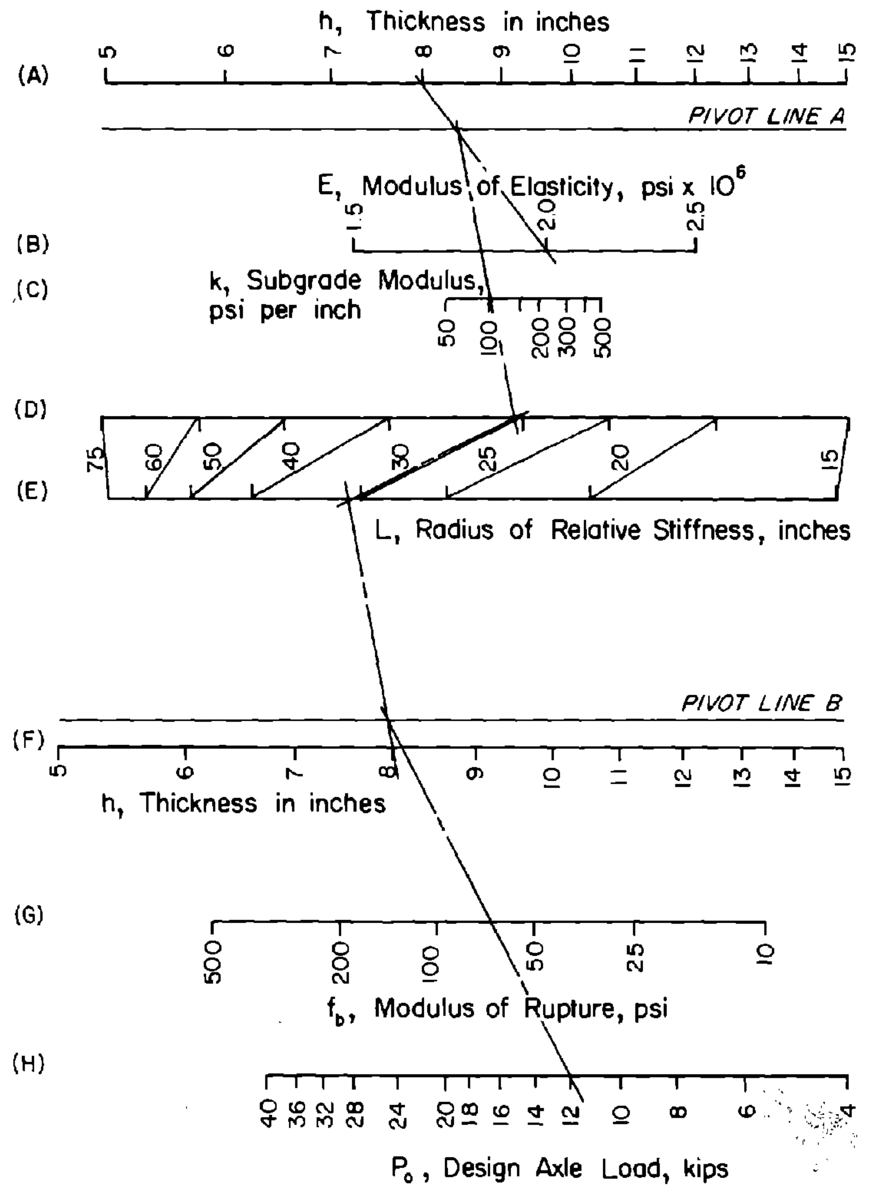

Figure 16. Nomograph for determining required thickness of LFA or LCFA layers.

F. Construction Procedures

Among the advantages of LFA and LCFA mixtures in pavement construction are the ease of construction and the ability to use conventional construction equipment to mix and place the materials. The major requirements of construction are that the ingredients be thoroughly mixed, spread uniformly to the proper thickness with a minimum of manipulation and segregation, and compacted to a high relative density. This can be accomplished with construction equipment normally found on a pavement construction site (i.e., spreader box, motor patrol, rollers, water truck, etc.). While 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 of low reliability. The critical factors are discussed in greater detail in the following paragraphs. These procedures are equally applicable to LFA and LCFA mixtures.

Blending and Placing of Materials

Blending of LFA materials can be done either in-place on the roadbed using pulvimixers and similar equipment or in a central plant. Plant blending gives greater control over the quantity of ingredients added and more uniform mixing. Both central plant and mix-in-place operations are discussed below separately.

1. Central Plant Operations

Mixing - The main components of a central plant 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) Pugmill for mixing of the components.

Most plants use the continuous type pugmill, in which case the components are metered volumetrically onto a conveyor belt which feeds directly into the pugmill. The conveyor belt may be covered to prevent wind erosion of the components from the belt. If the pugmill is the batch type, then each component is usually weighed from individual bins and dumped into the pugmill. A surge bin is needed with both types of pugmills to collect the blended mixtures into truck-sized lots.

Lime and cement are stored in vertical silos and delivered to the plant by gravity and compressed air. For continuous-type plants where the lime and cement are metered volumetrically, these components are usually transferred from the large storage silos to small feed tanks. The feed tanks have compressed air flowing through the lime and cement from the bottom of the tank to prevent consolidation of the material. Care must be taken to calibrate these plants after a period of continuous operation as the lime and cement may consolidate with time during non-operational periods. In the consolidated state, the amount of material fed onto the belt during each revolution of the feeder is much greater than in the unconsolidated state.

Fly ash is normally stored in open stockpiles, but fly ash stored in this manner must be conditioned with sufficient water to prevent dusting (usually 15 to 20 percent residual moisture content). Also, during dry weather, the stockpile surfaces must be kept moist or the stockpile covered to prevent dusting. The conditioned fly ash is charged into the feeder hopper with a front end loader or by other means. Some fly ashes will set up, or self-harden, in the stockpile and must be recrushed before using in LFA mixes. Hammer mills and roll mills have been used effectively to crush the hardened fly ash.

The pugmill-type mixing plants described above are normally the type used to blend LFA materials. However, central mix concrete plants have also been successfully used for this purpose. When using the central concrete mixing plant, care must be taken to assure adequate mixing time is allowed for thorough blending of the constituents.

Hauling - LFA mixtures which are blended in a central plant location can be hauled to the road site in conventional, open-bed dump trucks. If haul distances are long or drying of the material enroute poses a problem, then provisions should be made to cover the trucks with tarpaulins or other suitable covers to prevent loss of moisture or scattering of environmentally objectionable dust along the 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.

Spreading - Plant blended LFA and LCFA mixtures should be delivered to the prepared subgrade and spread as uniformly as possible with a minimum of manipulation. Various types of equipment have been used successfully for spreading the blended LFA mixture. Use of spreader boxes, asphalt laydown machines, or other equipment with automated grade control is recommended, as the more automated equipment generally gives better uniformity of depth with a minimum of manipulation and segregation. An alternate method of spreading sometimes used but not recommended is to place the materials in windrows from the trucks and spread with road graders. With the windrow-type of operation, care must be taken not to overmanipulate the material which causes drying and segregation.

Layers of the LFA mixture 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 experimentation may be necessary to determine the proper spread thickness for each operation, as some types of spreading operations provide a degree of initial consolidation.

The maximum recommended thickness for a single layer of LFA after compaction is 8 to 10 inches, with some agencies specifying 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 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. As a general rule, subsequent layers should be placed the same day, but with multiple layered pavements, such as airport and marine terminal pavements, this is not always possible. If the LFA mixture in the lower layer has taken on an initial set, steps should be taken to insure the development of a bond between the two layers. Specifically, steps should be taken to insure that there is no loose material on the lower layer, and that the surface is moist before placing the material for the subsequent layer.

Compaction - A critical step in the construction of pavements with LFA mixes is compaction. Achieving a high relative in-place density of these materials is the key to good performance. Steel-wheel, pneumatic, and vibratory rollers, and vibratory pans have all been used effectively for compacting LFA mixes. Since the material is basically granular in nature, with little or no cohesion at the time of compaction, pneumatic-tired rollers, vibratory rollers, and the vibratory pans are usually most effective in providing initial densification of the mixes.

An essential 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. Care should be taken to provide an adequate support for the placement and compaction of these materials, even if it requires treatment of the existing soil. Treatment of soft subgrades usually pays off in reduced construction costs and enhanced pavement performance.

Steel-wheel rollers are normally 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 motor patrol or string-line sub-grader prior to final rolling with the 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, but with some mixes compaction can be achieved over a longer 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 as it is mixed and spread. With some of the fast-setting fly ashes, normally 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.

2. Mix-in-Place Operations

Satisfactory quality lime-fly ash-aggregate mixtures have been produced in mix-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. After thorough blending of the components to the desired depth, the LFA mixture is spread to the required thickness and compacted to the desired density. While satisfactory performances have been attained with mixtures prepared in this manner, the overall quality of the mix-in-place operations is significantly less satisfactory than with plant mix operations. Some problems and limitations with the mix-in-place operations are discussed below.

<u>Preparation of the Roadbeds</u> - One of the greatest advantages of mix-in-place operations is that the aggregates already in-place on the roadbed can be incorporated into the mixture. While the cost for

the aggregates in the mixture are greatly reduced, the quality of aggregates obtained in this manner is usually reduced. Most roadbed aggregates have some soil intermixed, and these soil fines may significantly reduce the quality of LFA mixtures.

When using in-place aggregates for LFA mixtures, all the standard mix design tests should be run to evaluate their suitability. It may be necessary to modify the aggregates to produce a satisfactory LFA mixture. 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 aggregates in place contain excessive silts, this may tend to "choke down" the lime-fly ash reactivity, further lowering the quality of the mixture. If the fines are predominantly clay minerals, then lime may be preblended into the aggregate to break down the clay, making it a more workable mix.

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

1. Scarify the in-place soil aggregate material;

2. Spread enough lime on the scarified roadbed to reduce 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-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 mix-in-place operations.

Spreading Lime and Fly Ash - In most instances, lime and fly ash are spread separately on the prepared roadbed in the mix-in-place operations. It is possible, however, to preblend these two components before spreading as is done with the "Master Mix" material supplied by several suppliers. When the lime and fly ash are preblended, it is necessary that they be stored in a dry state. The preblended mix is normally spread in the dry condition.

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

   Spreading dry lime poses two major problems: (1) achieving a uniform lime distribution and (2) controlling the dusting associated with the discharge from the pneumatic truck. In populated areas, the dusting problem may be severe, and special precautions should be taken with this operation.

   The major problems associated with spreading the lime in slurry form are that large quantities of water are required, and the cost of hauling water long distances may be considerable. 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 an adequate supply of water is nearby and inexpensive.

b. Fly Ash: Nearly all fly ash is spread in the conditioned state (i.e., residual 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, dumped, and spread with a motor patrol, 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 - Blending in place is normally done with rotary mixers and similar equipment. Heavy duty rotary mixers must be used to make this operation successful. Blending can also be done with road graders, but this method is much less effective than the rotary mixers, and improper manipulation with road graders can result in segregation of the coarse and fine aggregates in the mix.

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

3. 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 (.0005-.001 m^3/m^2) of cut-back liquid or emulsified asphalt is placed the day following compaction, and any surface layers are usually applied as soon thereafter as can be scheduled in the construction sequence. The only justification for delay in surfacing the LFA mixes is if heavy rains saturate the base and subbase making the compacted roadway unstable, causing it to shove and rut under the surfacing equipment and trucks.

4. Quality Control Testing

The quality of LFA mixtures, as produced and placed, must be monitored on a continuing basis to insure a quality product. The tests most commonly run on these materials are listed below in their order of importance or frequency of test:

1. In-Place Density (AASHTO T 238-73, AASHTO T 205-64, or AASHTO T 191-61);

2. Lime (and Cement) Content (ASTM D 3155-73);

3. Aggregate Gradation (ASTM D 136-71);

4. Moisture Content (ASTM D 2216-71 or AASHTO T 239-73).

In addition, frequent checks should be made on all batch and continuous feeds of mixing plants to insure that the metering of the components is progressing uniformly.

5. Construction Season

Construction season varies with the climatic conditions of any particular site and the manner in which the paved section is used during the first winter. Early season construction is normally limited by the dates in 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 mixture, and the anticipated temperature between the last construction date and beginning of heavy frost penetration. A typical construction season for northern and central Illinois ranges from about April 15-30 to mid-October. However, in Newark, New Jersey, where the climate is more moderate, no loads were to be placed on the pavement during the winter months, so LCFA was mixed and placed through December 1st 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 A.

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IV. STABILIZED FLY ASH PAVEMENTS

A. Factors Affecting Utilization

Stabilized fly ash base and subbase courses represent a unique application of fly ash in that fly ash serves both as a pozzolan and an aggregate. The fly ash and stabilizer function mechanically much the same as a fine-grained soil-cement except that the natural pozzolanic reaction between the fly ash and the stabilizer continues to produce an increase in strength over a long period of time, thereby enhancing the durability of the base or subbase course.

Although many fly ashes possess self-hardening properties (see Section II), the strength developed within a reasonable time period is generally not adequate for pavement application, either in terms of load-bearing capacity or frost resistance, particularly since most base or subbase courses are constructed within the frost zone in most regions of the United States. Hence, the addition of a stabilizer is usually required. Lime and cement are the most commonly used stabilizers.

Certain chemical and physical characteristics influence the degree to which a fly ash can react with a stabilizer. This reactivity determines not only the ultimate strength which a stabilized fly ash achieves, but also the rate at which the strength gain takes place with time. The presence of silica, alumina, and calcium oxide in large quantities enhances the reactivity of a fly ash. In fly ashes where the calcium oxide has been leached out, as in the case of lagoon or stockpiled ashes, reactivity may be somewhat reduced. High carbon contents are detrimental to the pozzolanic reaction; thus, fly ash produced in older, less efficient units may require slightly greater amounts of stabilizer than the low carbon fly ashes found in newer power plants. The gradation of a fly ash, also, can affect the rate of pozzolanic reaction, resulting in higher reactivities in fly ashes from hoppers than from lagoons, where coarser ash may be mixed with the fly ash.

As previously mentioned, stabilized fly ash can be used as either a base or a subbase course. When used as a base course, it is normally covered with a bituminous wearing surface to protect it from water and abrasion.

B. Case Histories

Based on demonstration projects and actual construction projects, lime- or cement-stabilized fly ash base and subbase courses show great promise as large-tonnage applications of fly ash as well as acceptable alternatives to natural aggregate courses. Although relatively new in this country, stabilized fly ash has been in use as a base course material in Great Britain for nearly 15 years. Extensive research and development work in the early 1960's at the British Road Research Laboratory and the Universities of Salford and Glasgow in Great Britain led to a number of field trials using cement-stabilized fly ash base courses. The success of these trials resulted in the adoption of specifications for fly ash-cement base and subbase courses by the Ministry of Transport (later the Department of the Environment). More recently, two ready-mix plants for fly ash-cement base course have been established in Great Britain. Their combined annual output is about 40,000 tons (36,000 Mg) per year.(15)

The French have been using fly ash in pavement construction since 1957.(7) Fly ash-cement mixes have been used as subbase on a number of major highways and have been found to be superior in quality and economy to locally available paving materials.

In the United States, several field trials and demonstration projects have been undertaken in different parts of the country to evaluate the performance of stabilized fly ash pavements. Test sections have included those constructed at Kansas City, Missouri,(8) Haywood, West Virginia,(6) and Charleston, West Virginia.(9) The results of these field trials to date have all been favorable.

Several case histories of construction projects utilizing cement-stabilized fly ash base or subbase courses are presented below.

Access Road, Ince "B" Power Station
Lancaster, England

Pre-mixed cement-stabilized fly ash was used in May of 1972 as a subbase for an access road which was to carry all construction traffic for the Ince "B" Power Station.(14) The 24-foot (7.3 m) wide road was constructed on a fly ash embankment about 6.6 feet (2 m) in height. Approximately 253 tons (230 Mg) of subbase mix made with fly ash from Bold Power Station were placed daily, with a total of 2200 tons (2000 Mg) being used on the project. The subbase mix contained 11 percent cement by weight, and was placed in a single layer and compacted to a final thickness of 4 inches (100 mm). A 6-inch (150 mm) base course of bituminous stabilized material and a 4-inch (100 mm) asphalt surfacing were placed on the subbase.

Access Roads, Royal Agricultural Showground Stoneleigh, England

Cement-stabilized fly ash was selected as the base course for three access roads at the Royal Agricultural Showground at Stoneleigh, in Warwickshire.(5) A total of 2635 lineal feet (803 m) of roadway was constructed utilizing fly ash reclaimed from lagoons at Hams Hall Power Station. A design mix of 10 parts fly ash to 1 part cement was mixed in-place on a lime-stabilized clay subgrade and compacted to a thickness of 8 inches (203 mm). Unconfined compressive strengths of the cement-stabilized fly ash base course achieved in the field are shown in Table 8.

The base course was sealed with a tack coat at the end of each day's work, and a slurry seal 1/8 inch (3 mm) in thickness was applied as a surface treatment.

Access Road and Parking Lot, Harrison Power Station Haywood, West Virginia

In September of 1975, 10,000 square yards (8,400 m^2) of pavement utilizing cement-stabilized fly ash base course were constructed for the access road and parking lot at Harrison Power Station.(6) Fly ash was trucked directly from hoppers of two boiler units at Harrison to a pugmill which had been set up on-site. Cement and fly ash were premixed with water in the pugmill at the rate of 83 pounds (37.5 kg) of fly ash and 10 pounds (4.5 kg) of cement per cubic foot of compacted mix. The base course mix was spread and compacted to an 8-inch (203 mm) thickness and sealed with a bituminous emulsion. A 3-inch (76 mm) bituminous

Table 8. Development of compressive strength in cement-stabilized fly ash base course at Stoneleigh, England.(5)

AGE OF BASE COURSE days	UNCONFINED COMPRESSIVE STRENGTH psi (kPa)
7	400 (2760)
28	760 (5240)
90	1250 (8620)
270	1660 (11,450)

surface was applied a few weeks later. Cores taken at 7 days and 90 days yielded unconfined compressive strengths of 566 psi (3900 kPa) and 869 psi (5990 kPa) respectively. Strengths of cores taken at 180 days indicated that the pavement had experienced no strength loss during the severe winter. Monitoring of the pavement is scheduled to continue for a period of one year after construction. Photographs of the construction are shown in Figures 17 through 20.

C. Mix Design Concepts

Mix design procedures for cement- or lime-stabilized fly ash consist of two parts: development of a design mix for a given sample of fly ash and determination of a construction mix for a given source of fly ash or, in some cases, for an entire power plant. A design mix indicates those proportions of fly ash, stabilizer, and water which, when mixed and compacted in the laboratory to a specified density, satisfy specified strength and/or durability criteria. In reality, samples of fly ash collected from the same source but at different times (in the case of hoppers and silos) or different locations (in the case of lagoons and stockpiles) may differ in chemical and physical characteristics (see Section II). This may result in different reactivities between the samples; thus, one sample of fly ash may require a different quantity of stabilizer than another sample to satisfy the established design criteria. It is reasonable to assume that there may be variations in the fly ash used for a specific construction project, particularly if large quantities of fly ash are used, construction occurs over a long period of time, or more than one source of fly ash must be used.

The concept of a construction mix was developed to account for the variability of fly ash in the mix design procedure. Briefly, a construction mix is the design mix for the least reactive fly ash expected from a given source(s) over some period of time, with some additional water added to compensate for moisture normally lost during the construction process.[6]

The choice between lime or cement stabilization is influenced by a number of factors. Cement produces a more rapid strength gain than lime, although equal strengths are usually achieved within three months.[16] In stabilized fly ash base/subbase construction, the curing period normally allowed before opening the road to traffic is seven days. Because of the slower reaction times with fly ash-lime mixes, either higher curing temperatures or greater

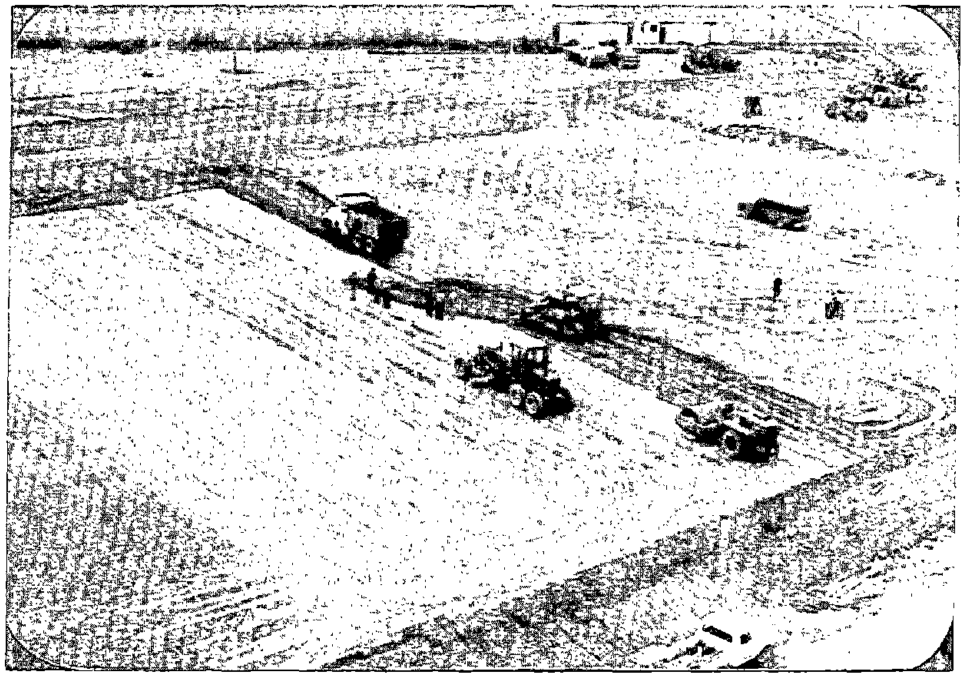

Figure 17. Spreading, compaction, and fine-grading of cement-stabilized fly ash base course at Harrison Power Station parking lot.

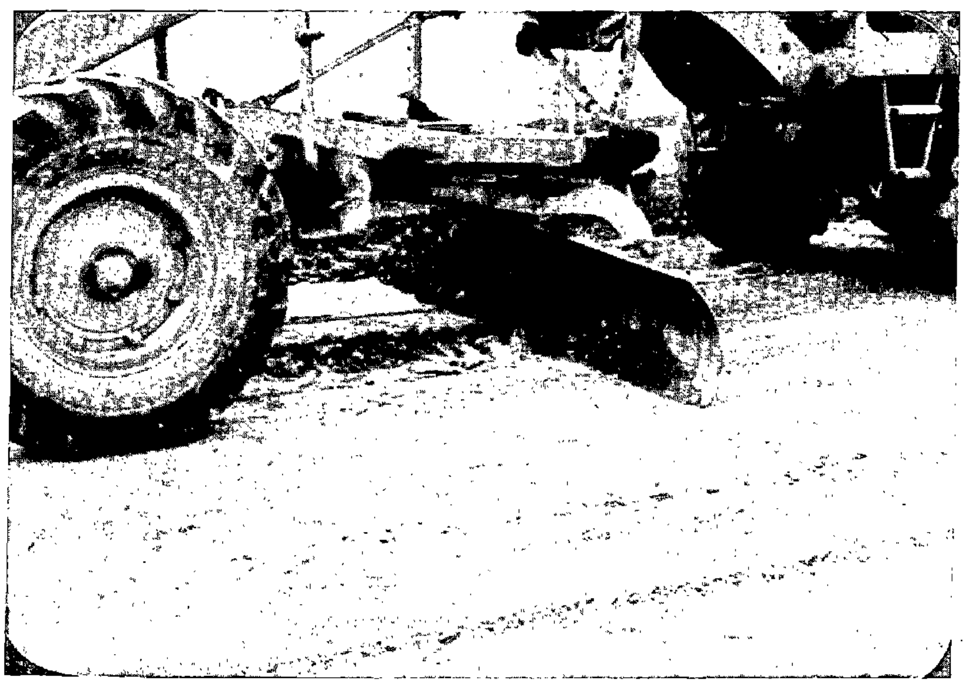

Figure 18. Close-up of fine-grading of cement-stabilized fly ash base course at Harrison Power Station parking lot.

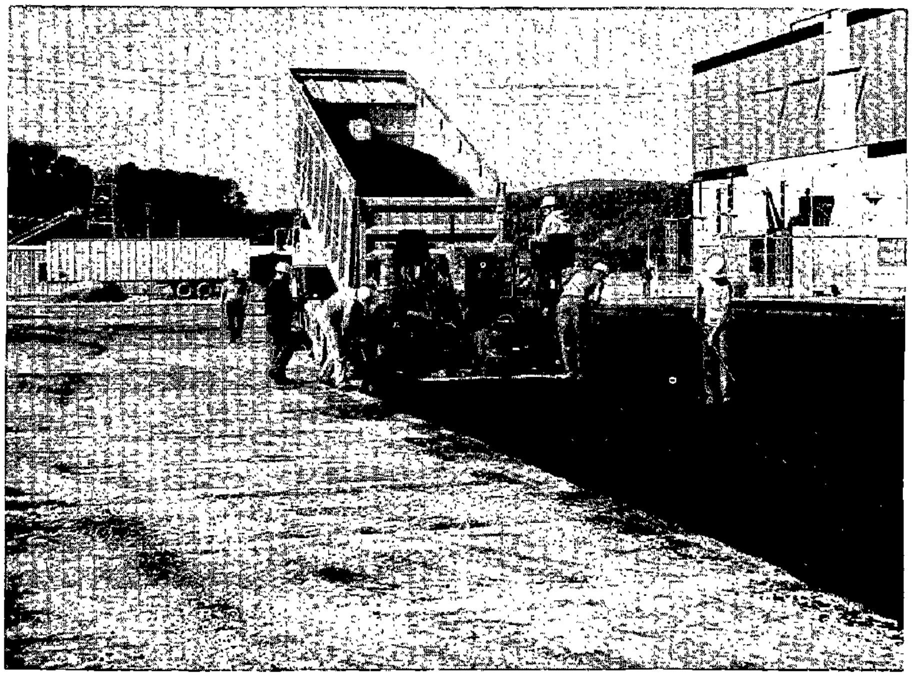

Figure 19. Construction of wearing surface at Harrison Power Station parking lot.

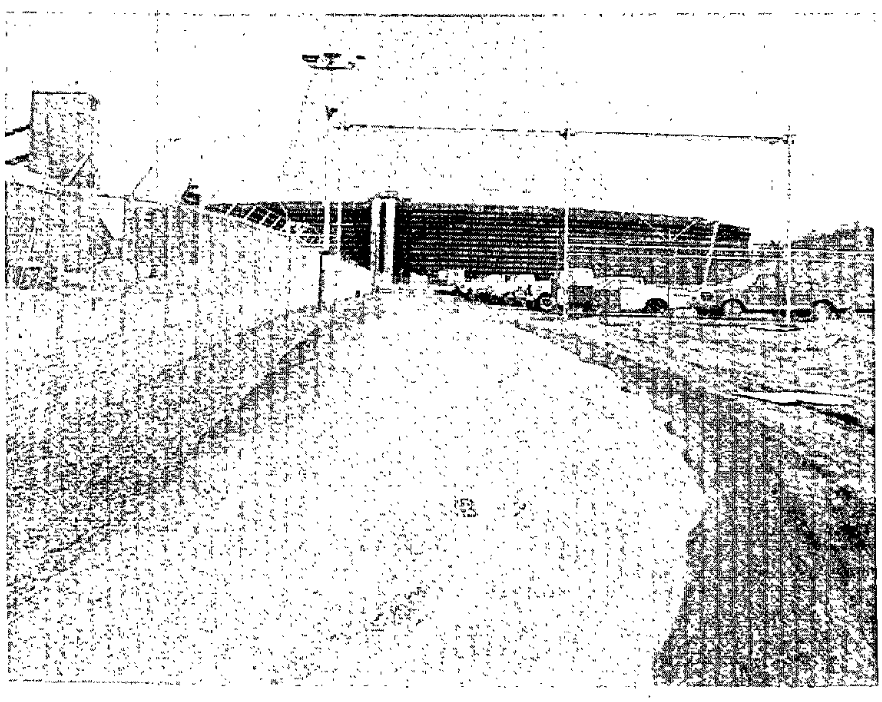

Figure 20. View of access road to Harrison Power Station parking lot after first winter.

quantities of lime may be required to achieve the required strength in seven days, or a longer curing period can be specified. In certain areas of the United States, lime is cheaper than cement. However, the aforementioned conditions must be taken into consideration as well when evaluating the relative economies of lime versus cement.

Because of the difference between cement and lime stabilization, separate design criteria have been developed for each. They are discussed separately below.

Cement-Stabilized Fly Ash Mixes

1. Design Criteria

The thickness design method used for stabilized fly ash base/subbase courses requires only that the mix be durable.[6] Durability can be measured in several ways. However, the most practical method to date is based on unconfined compressive strength. The British mix design criterion is based on compressive strength,[4] as are the criteria developed in the design guide for cement-stabilized fly ash base courses which has been published by the National Ash Association.[6] The basis of both criteria is that a specified compressive strength is an indication of the mix's ability to resist damaging frost action. The following dual mix design criteria have been adopted for cement-stabilized fly ash:

a. The seven-day unconfined compressive strength of the mix, when cured under moist conditions and at 70±3°F (21±2°C), must be 400-450 psi (2760-3100 kPa).

b. The unconfined compressive strength of the mix must increase with time.

A previous study in this county attempted to correlate unconfined compressive strength with durability as determined by the freeze-thaw and wet-dry brushing tests developed for soil-cement mixes (AASHTO T 136-70 and 135-70, respectively).[6] Conclusions reached in that study were that, while the wet-dry tests had negligible effect on the fly ash-cement mixes tested, the freeze-thaw test was unduly abrasive and was very dependent on sample preparation techniques. Thus, it was suggested that these particular tests were not suitable measures of durability for fly ash-cement mixes, and that compressive strength tests could be used alone for ensuring adequate durability.

Although no specific data is available on the applicability of other types of durability tests to fly

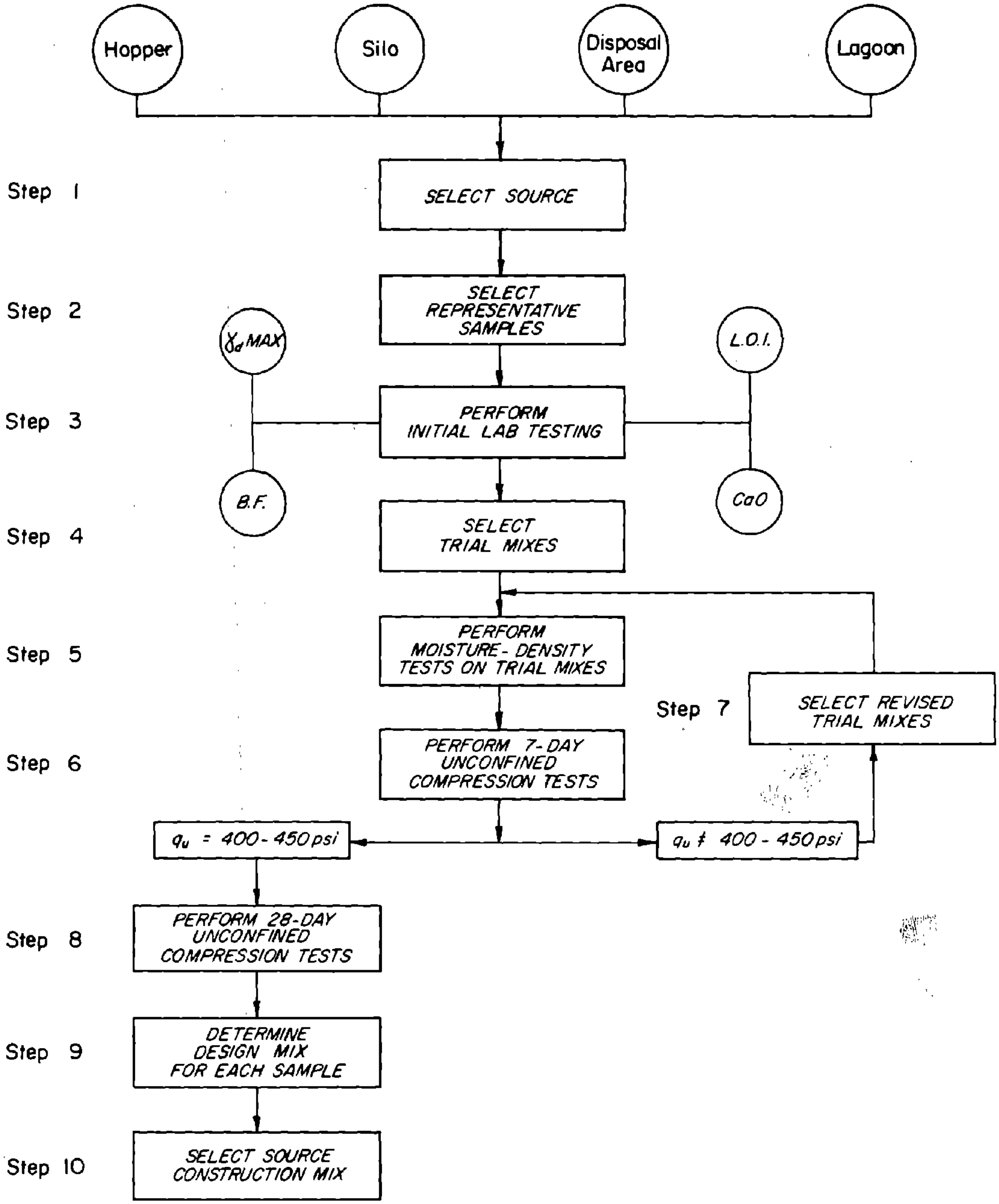

Figure 21. Single source testing program for determining cement-stabilized fly ash construction mix.(6)

ash-cement mixes, cement-stabilized fly ash pavements which will be subjected to extreme service conditions should be tested for durability in some manner. Durability tests based on residual strength after a certain number of freeze-thaw cycles have been developed for lime-fly ash-aggregate and lime-fly ash-soil mixes (see Sections III and V), and these would seem suitable for fly ash-cement mixes. The particular criterion to be met should be determined from the anticipated field conditions to which the pavement will be subjected.

2. Laboratory Testing Program

The generalized laboratory testing procedure for cement-stabilized fly ash is outlined in Figures 21 and 22. Figure 21 represents the procedure to be followed for determining design and construction mixes when fly ash is to be secured from a single source. If more than one source is utilized, fly ash from <u>each</u> source is subjected to the testing program in Figure 21. In addition, the selection procedure for a multiple source construction mix illustrated in Figure 22 is performed. The procedures for the laboratory testing program are discussed in detail in Reference 6.

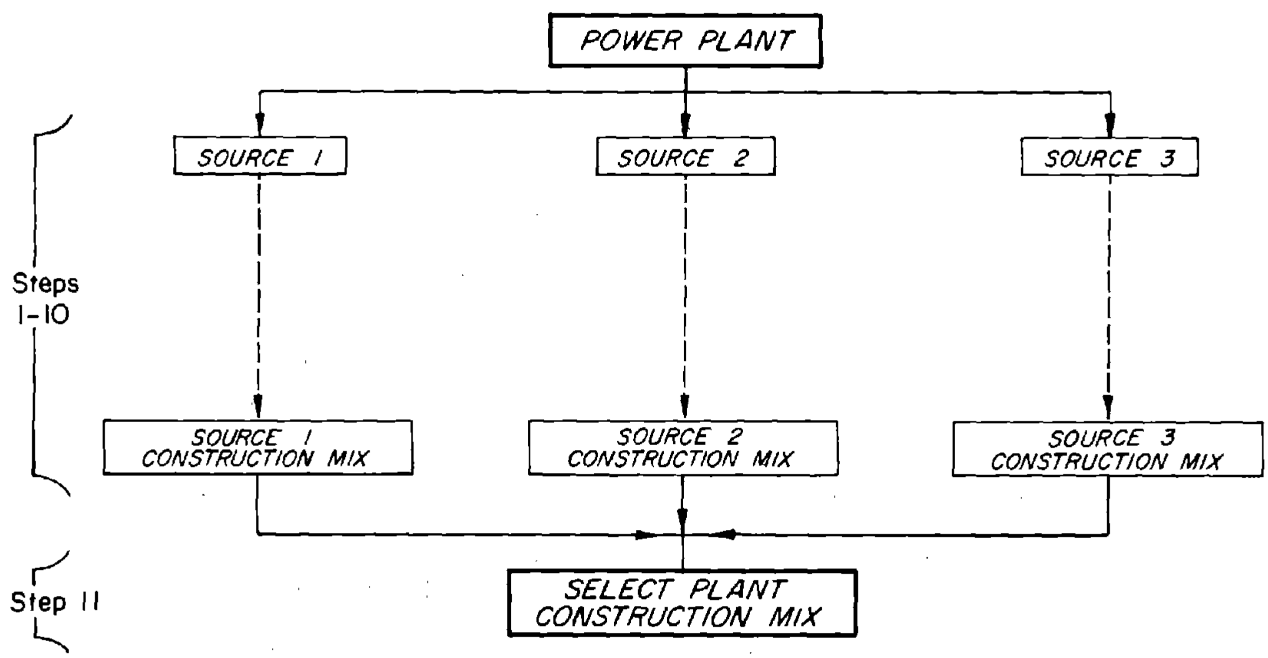

Figure 22. Multiple source testing program for determining cement-stabilized fly ash construction mix.(6)

Step 1. Select Source - The selection of a power plant is determined almost solely by location. Selection of a fly ash source within a power plant is done with the aid of plant personnel or ash marketing advisers.

Step 2. Select Representative Samples - Samples are taken from each source under consideration in such a manner as to obtain as realistic a representation as possible of the variation in the fly ash which can be secured from that source. In the case of hoppers and silos, samples should be taken over a period of time. For stockpiles and lagoons, samples should be taken from different locations. A minimum of three representative samples is recommended for each source.

Step 3. Perform Initial Laboratory Testing - A number of initial laboratory tests can be performed to provide some indication of the reactivity of a fly ash sample. The following tests should be performed on each representative sample collected:

- o Moisture-Density Relationship (AASHTO T 99-74)
- o Blaine Fineness (Specific Surface, ASTM C 311-68)
- o Loss-on-Ignition (ASTM C 311-68)
- o Calcium Oxide (CaO) Content

Step 4. Select Trial Mixes - The results of the initial laboratory testing can be used to determine the cement content of the first trial mix for each sample. The guidelines for selecting the initial cement contents are illustrated in Figure 23. These guidelines are only approximate, but should serve to reduce the total number of trial mixes which must be tested.

Step 5. Perform Moisture-Density Tests on Trial Mixes - Once the initial cement content has been chosen for a trial mix for each representative sample, the moisture-density relationship for each trial mix should be determined in accordance with AASHTO T 134-70. This is necessary since the unconfined compressive strength decreases markedly with a decrease in density; therefore, strength testing is done at maximum dry density and optimum moisture content. Also, the addition of cement affects the compaction characteristics of a fly ash; thus, it is necessary to establish the moisture-density curve for each fly ash-cement trial mix as well as for the fly ash samples.

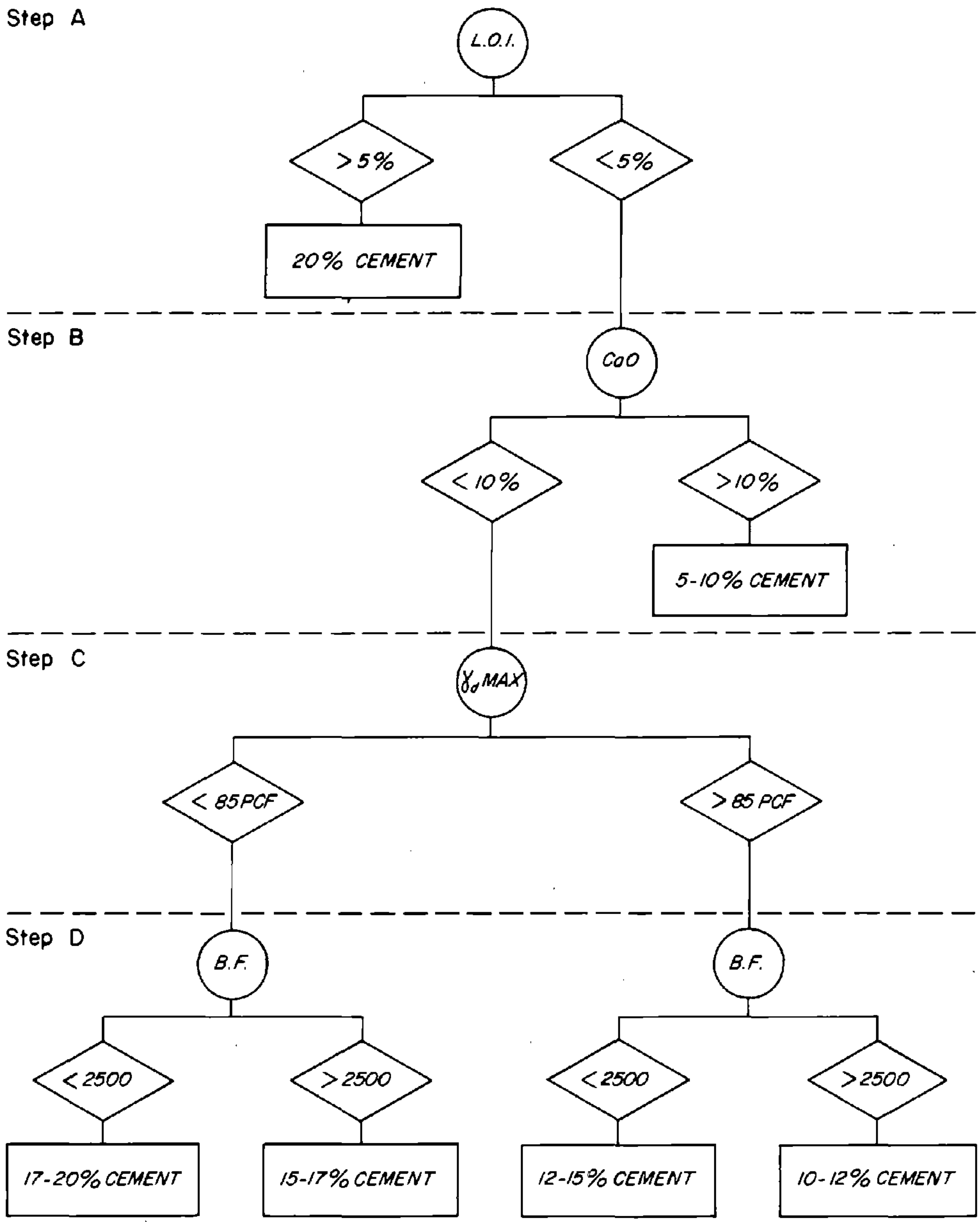

Figure 23. Guidelines for selection of cement-stabilized fly ash trial mixes.

Step 6. Perform Seven-Day Unconfined Compression Tests - Unconfined compressive strength tests are performed on test cylinders of each trial mix which have been cured for seven days at a constant temperature of 70±3°F (21±2°C) under moist conditions. Dimensions of the cylinders should be such that the length is twice the diameter. If this is not possible, the resultant strengths from cylinders with other ℓ/d ratios should be factored in accordance with Table 9. Cylinders should be molded at maximum dry density and optimum moisture content of the trial mix in accordance with AASHTO T 134-70, and wrapped tightly in cellophane or plastic bags or coated with paraffin to prevent moisture loss. At the end of seven days, cylinders are capped in accordance with AASHTO T 231-74, except that the sulphur capping compound is recommended, and broken in accordance with AASHTO T 22-74. It is recommended that more than one cylinder be broken per trial mix and the test results averaged to obtain an unconfined compressive strength for the trial mix.

Step 7. Select Revised Trial Mixes - If the trial mix for any representative sample fails to produce a seven-day unconfined compressive strength of 400-450 psi (2760-3100 kPa), a second trial mix should be developed having a lesser or greater cement content, depending upon whether the average trial mix strength was greater or less than the specified range. The gross approximation that doubling the cement content doubles the strength can be used. Although trial mixes with seven-

Table 9. Strength Correction Factors for compressive strength cylinders.(1)

LENGTH TO DIAMETER RATIO ℓ/d	STRENGTH CORRECTION FACTOR S.C.F.
2.00	1.00
1.75	0.99
1.50	0.97
1.25	0.94
1.00	0.91

Adjusted Strength = Measured Strength x S.C.F.

day strengths above 450 psi (3100 kPa) have adequate strengths for base/subbase applications, they do not represent the most economical mix for that particular sample, and are therefore revised. The second trial mix is recycled through Steps 5 and 6 as previously described. The mix revision and cycling process continues until a trial mix is developed which satisfies the compressive strength criterion. In general, a maximum of three trial mixes per sample will be required before the strength criterion is satisfied.

<u>Step 8. Perform 28-Day Unconfined Compression Tests</u> - The trial mix for each representative sample which satisfies the seven-day strength criterion is tested to verify the mix's strength gain with time. Unconfined compression tests are performed in the same manner as outlined in Step 6 except that the test cylinders are cured for 28 days. The 28-day strength is generally about twice the seven-day strength. It is unusual to encounter a fly ash-cement mix which exhibits no strength gain with time after the initial cure. If a trial mix fails to produce a strength gain, however, it is recommended that the source from which the corresponding fly ash sample was secured be eliminated from consideration as a supply of fly ash, as a non-reactive fly ash could have serious consequences on the long-term durability of a construction project.

If durability testing is required due to anticipated severe field conditions, it should be performed on the trial mix which satisfies both seven-day and 28-day strength criteria. Failure of a trial mix to meet durability criteria will necessite increasing the cement content of the trail mix until the criteria is met.

<u>Step 9. Determine Design Mix for Each Sample</u> - The design mix for a representative sample of fly ash is equivalent to that trial mix which has an average seven-day unconfined compressive strength of 400-450 psi (2760-3100 kPa) when tested in accordance with Step 6, and which shows an increase in strength between seven-day and 28-day unconfined compressive tests (and which meets durability criteria when necessary). A design mix is determined for each representative sample from a particular source.

Although the cement content of the trial mixes is denoted as a certain percent of the dry weight of fly ash, which is convenient for laboratory purposes, this does not provide an accurate indication of the actual amount of cement required when comparing design mixes of different fly ash samples. Therefore, another nomenclature is developed for design mixes: the pounds (kg) of cement required per cubic foot of compacted mix, based on the maximum dry density and optimum moisture content of the mix as determined by AASHTO T 134-74. Thus, a trial mix with 10 percent cement and a maximum dry density of 90 pcf (1440 kg/m^3) would contain 8.2 pounds (3.7 kg) of cement per cubic foot, whereas a 10 percent cement trial mix with a maximum dry density of 80 pcf (1280 kg/m^3) would contain 7.3 pounds (3.3 kg) of cement per cubic foot.

Step 10. Select Source Construction Mix - The construction mix for each source is determined in the following manner:

1. The highest cement content, in terms of pounds (kg) per cubic foot, of all the design mixes from a source becomes the cement content of the construction mix.

2. The optimum moisture content of the design mix with the highest cement content is increased by two percent, and this becomes the specified moisture content of the construction mix.

Step 11. Select a Plant Construction Mix - If more than one source of fly ash is available at a given power plant, each source is run through testing Steps 1 to 10. Then, as illustrated in Figure 22, the resultant construction mix for each source is compared, and the construction mix with the highest cement content is chosen as the overall construction mix for the particular combination of sources or the entire power plant, whichever the case may be. Understandably, this will result in an over-design for the other sources. If the over-design is economically unacceptable, then consideration should be given tc eliminating the source requiring the highest cement content.

Lime-Stabilized Fly Ash Mixes

1. Design Criteria

The mix design criteria for lime-stabilized fly ash mixes differs from that for cement-stabilized fly ash mixes in that the slower rate of strength gain for fly ash-lime must be taken into consideration:[4]

a. The 28-day unconfined compressive strength of the mix, when cured under moist conditions and at 70±3°F (21±2°C), must be 550-600 psi (3790-4135 kPa).

b. The unconfined compressive strength of the mix must increase with time.

2. Laboratory Testing Program

The same logic indicated in Figures 21 and 22 applies to fly ash-lime mixes except that certain testing details differ. The variations from Figure 21 are as follows:

Step 4. Select Trial Mixes - The guidelines for trial mix selection outlined in Figure 23 apply only to fly ash-cement mixes. Specific guidelines applicable to fly ash-lime mixes have not as yet been developed; however, the same trends regarding results of initial laboratory tests still apply. The amount of stabilizer necessary for a given set of parameters would probably be somewhat different than those indicated in Figure 23, however.

Step 6. Perform 28-Day Unconfined Compression Tests - Procedures for Step 6 are the same except that the curing period is extended to 28 days.

Step 7. Select Revised Trial Mixes - The criterion for revision of a trial mix is failure to achieve a 28-day unconfined compressive strength of 550-600 psi (3790-4135 kPa).

Step 8. Perform 45-Day Unconfined Compression Tests - Procedures for Step 8 are the same except that the curing period is extended to 45 days.

Step 9. Determine Design Mix for Each Sample - Design mix is determined in the same manner except that the criteria which it must satisfy are those stated above for lime-stabilized fly ash mixes.

D. Thickness Design

The procedure developed for soil-cement pavements by the Portland Cement Association[13] has been used for stabilized fly ash pavements in this country due to apparent mechanical similarities between the two materials.[6] It should be kept in mind, however, that the particular time-strength gain characteristics of fly ash-cement mixtures makes the cured mixtures less prone to fatigue effects than most soil-cement mixtures.[3]

The design procedure consists of the determination of two parameters, the subgrade modulus and the fatigue factor, which are then entered into a design chart to yield an initial base course thickness. The required thickness of bituminous wear surface is determined from the initial base course thickness. The base course thickness is then adjusted to account for the additional thickness of the wear surface.[6]

Subgrade Characteristics

The measure of subgrade strength used in this procedure is the subgrade modulus, k, as determined from field plate bearing tests. However, other acceptable methods of testing, such as California Bearing Ratio (CBR) and Resistance Value (R), can be used and converted to equivalent k-values using the broad relationships shown in Figure 24. Where light traffic conditions are expected, subgrade strengths can be estimated from the soil classification if field tests are impractical.

Design Period

The design period, n, generally assumed is 20 years. This does not represent the actual pavement life, but is considered to be the period of time between construction of the pavement and the first resurfacing or overlay.[2] Other design periods can be used at the designer's discretion.

Traffic

Four traffic parameters are necessary for determination of the fatigue factor:

1. Average Daily Traffic (ADT) - anticipated bi-directional volume during the first year of operation.

2. Percentage of Trucks - all single-axle, four-tire commercial vehicles as well as larger trucks with three or more axles.

3. Annual Traffic Growth Rate (r) - expected annual increase of the ADT. The growth rate is used in conjunction with the design period to develop the projection factor (P) from the following relationship:[2]

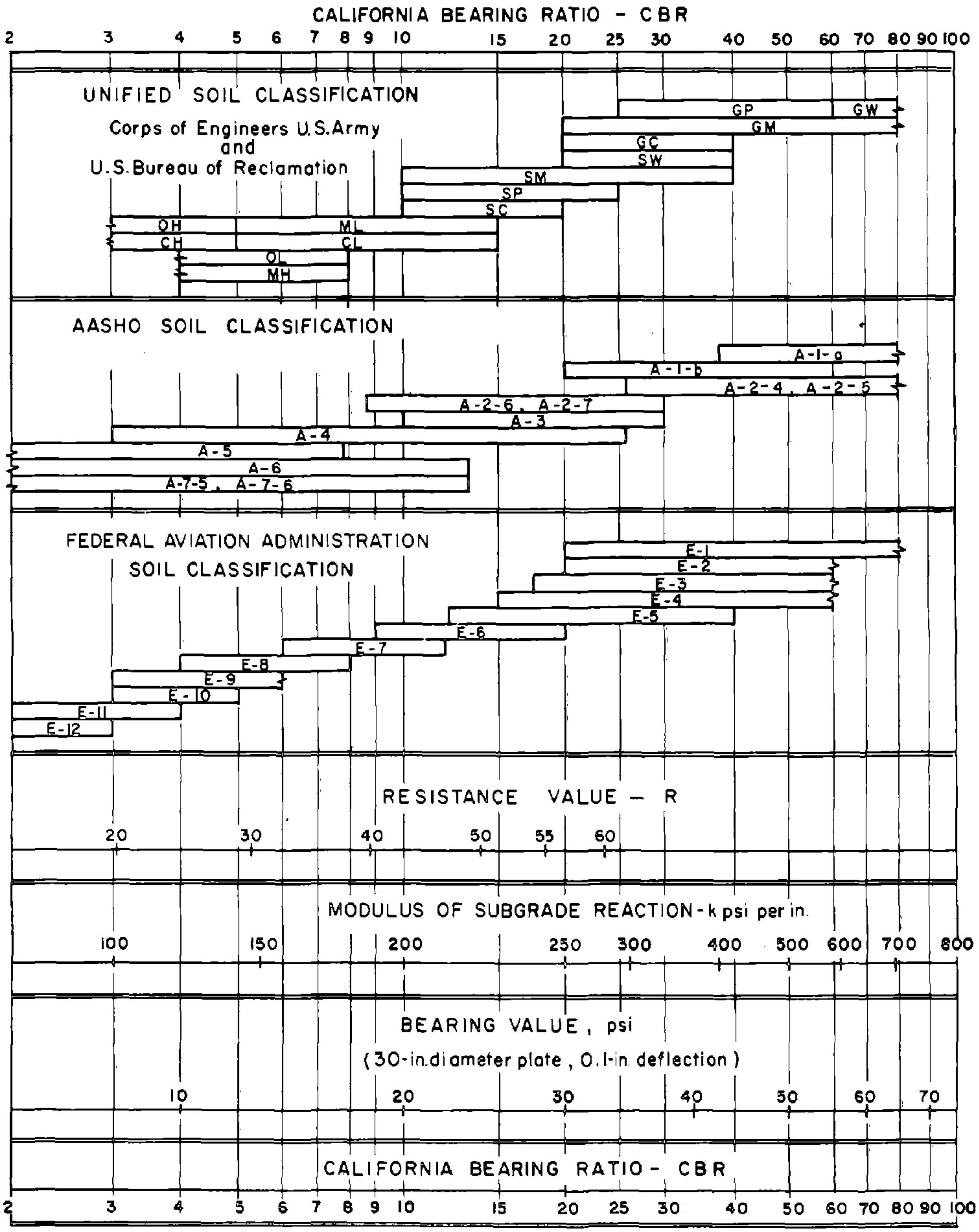

Figure 24. General relationships between soil classifications and bearing values.(12)

$$P = \frac{(1 + r)^n - 1}{20r}$$

Values of the projection factor can be found in Table 10.

4. Axle Load Distribution - the number of axles within each load group that can be expected for a given number of trucks. An example of an axle load distribution is shown in Table 11.

Fatigue Factor

The fatigue factor represents the total fatigue consumption of the pavement over the design period for a given loading configuration. The fatigue factor is calculated in the following manner:

1. The total number of trucks anticipated during the design period is calculated.

Table 10. Projection Factors.(2)

DESIGN PERIOD, n years	PROJECTION FACTOR, P, FOR ANNUAL GROWTH RATE, r					
	1	2	4	6	8	10
1	0.05	0.05	0.05	0.05	0.05	0.05
2	0.10	0.10	0.10	0.10	0.10	0.10
4	0.20	0.21	0.21	0.22	0.22	0.23
6	0.31	0.32	0.33	0.35	0.37	0.39
8	0.41	0.43	0.46	0.50	0.53	0.57
10	0.52	0.55	0.60	0.66	0.72	0.80
12	0.63	0.67	0.75	0.84	0.95	1.07
14	0.75	0.80	0.92	1.05	1.21	1.40
16	0.86	0.93	1.09	1.28	1.52	1.80
18	0.98	1.07	1.28	1.55	1.87	2.28
20	1.10	1.21	1.49	1.84	2.29	2.86
25	1.41	1.60	2.08	2.74	3.66	4.92
30	1.74	2.03	2.80	3.95	5.66	8.22
35	2.08	2.50	3.68	5.57	8.62	13.55

Table 11. Example of a truck axle load distribution matrix (Pittsburgh, Pa.).[6]

AXLE LOAD GROUP pounds (kg)	AXLES PER 1,000 TRUCKS
(a) Single Axles	
14,000 - 15,999 (6350-7257)	81.8
16,000 - 17,999 (7258-8164)	86.9
18,000 - 19,999 (8165-9071)	36.8
20,000 - 21,999 (9072-9978)	19.4
22,000 - 23,999 (9979-10885)	6.32
24,000 - 25,999 (10886-11793)	1.84
26,000 - 27,999 (11794-12700)	0.24
(b) Tandem Axles	
24,000 - 25,999 (10886-11793)	67.6
26,000 - 27,999 (11794-12700)	67.6
28,000 - 29,999 (12701-13607)	67.6
30,000 - 31,999 (13608-14514)	40.6
32,000 - 33,999 (14515-15421)	22.1
34,000 - 35,999 (15422-16329)	10.3
36,000 - 37,999 (16330-17236)	2.2
38,000 - 39,999 (17237-18143)	2.9
40,000 - 41,999 (18144-19050)	0.32
42,000 - 43,999 (19051-19957)	0.32
44,000 - 45,999 (19958-20864)	0.22
46,000 - 47,999 (20865-21772)	0.16
48,000 - 49,999 (21773-22679)	0.16

2. The total number of axles in each load category expected during the design period is calculated from the results of (1) and the given axle load distribution.

3. The fatigue effect contributed by each axle load group is determined from (2) and the fatigue consumption coefficients listed in Table 12.

4. The individual fatigue effects for each axle load group are added to yield the fatigue factor.

For residential streets and secondary roads where an axle load distribution is not available, the fatigue factor can be estimated from Table 13.

Table 12. Fatigue consumption coefficients.[13]

AXLE LOAD GROUP pounds (Kg)	FATIGUE CONSUMPTION COEFFICIENT
(a) Single Axles	
10,000 - 11,999 (4536-5442)	0.0018
12,000 - 13,999 (5443-6349)	0.0200
14,000 - 15,999 (6350-7257)	0.1600
16,000 - 17,999 (7258-8164)	1.0000
18,000 - 19,999 (8165-9071)	5.2
20,000 - 21,999 (9072-9978)	23.3
22,000 - 23,999 (9979-10885)	93.
24,000 - 25,999 (10886-11793)	337.
26,000 - 27,999 (11794-12700)	1,130.
28,000 - 29,999 (12701-13607	3,530.
(b) Tandem Axles	
18,000 - 19,999 (8165-9071)	0.0018
20,000 - 21,999 (9072-9978)	0.0081
22,000 - 23,999 (9979-10885)	0.0310
24,000 - 25,999 (10886-11793)	0.1070
26,000 - 27,999 (11794-12700)	0.3410
28,000 - 29,999 (12701-13607)	1.0000
30,000 - 31,999 (13608-14514)	2.74
32,000 - 33,999 (14515-15421)	7.1
34,000 - 35,999 (15422-16329)	17.5
36,000 - 37,999 (16330-17236)	41.1
38,000 - 39,999 (17237-18143)	93.
40,000 - 41,999 (18144-19050)	203.
42,000 - 43,999 (19051-19957)	431.
44,000 - 45,999 (19958-20864)	890.
46,000 - 47,999 (20865-21772)	1,790.
48,000 - 49,999 (21773-22679)	3,530.

Table 13. Representative fatigue factors for light-traffic pavements.(13)

TYPE OF ROAD	ADT	TOTAL TRUCKS[1] % (approx)	HEAVY TRUCKS[2] % (approx)	FATIGUE[3] FACTOR
Residential Street (local)	300 to 700	8	3	5,000 to 12,000
Residential Street (collector)	700 to 4,000	8	3	12,000 to 20,000
Secondary Roads	Up to 2,000[3]	14 to 20	5 to 8	12,000 to 30,000

[1] All commercial vehicles, including two-axle, four-tire vehicles.

[2] Excludes panels, pickups, and other two-axle, four-tire vehicles that are seldom heavy enough to affect design thickness.

[3] Ranges are based on the following characteristics of street and secondary road traffic: (1) one-half the indicated number of heavy axle loads, one direction; (2) axle-load distributions varying from 12,000 to 20,000 lb. on individual axles; (3) weighted averages of axle loads varying between 13,000 and 16,000 lb. on individual axles.

Initial Base Course Thickness

Once the subgrade modulus and fatigue factor have been determined, they are entered into the design chart in Figure 25 to yield an initial base course thickness. This initial thickness can be further adjusted to account for the load-spreading capacity of the wear surface.

Bituminous Wear Surface Thickness

A bituminous wear surface thickness should always be placed on a cement- or lime-stabilized fly ash base course to protect the base course from both water and abrasion. The thickness of bituminous wear surface can be related to the initial base course thickness by the chart shown in Figure 26. In addition to a curve representing the minimum required thickness of wear surface, Figure 26 contains a curve of recommended thickness. This curve is for use in frost areas where snowplows are used, and in situations where it is desired to minimize reflective cracking which may occur in the wear surface as a result of shrinkage cracking in the base course, a phenomenon common to stabilized bases.(10,11)

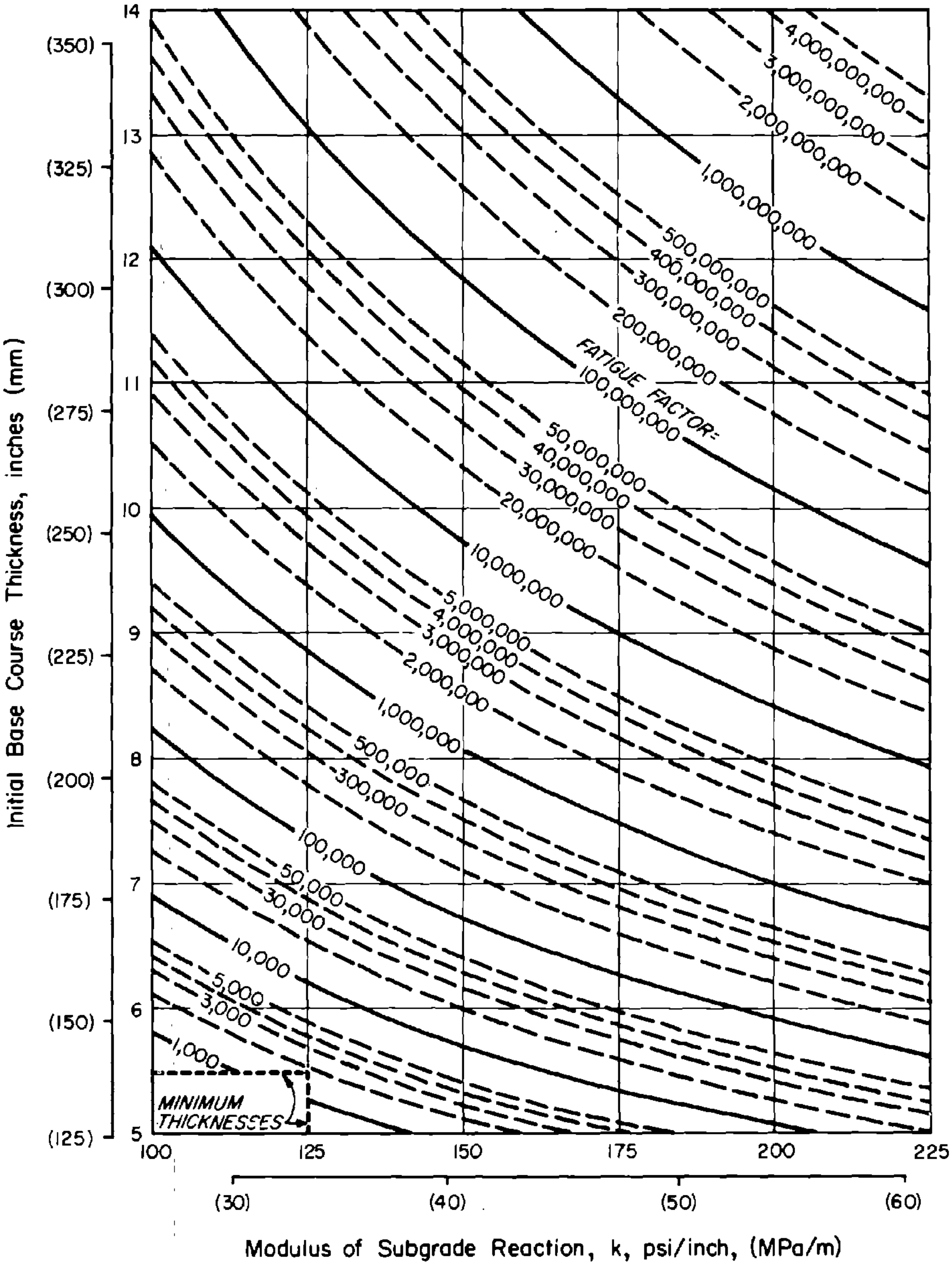

Figure 25. Initial base course thickness design chart.[6]

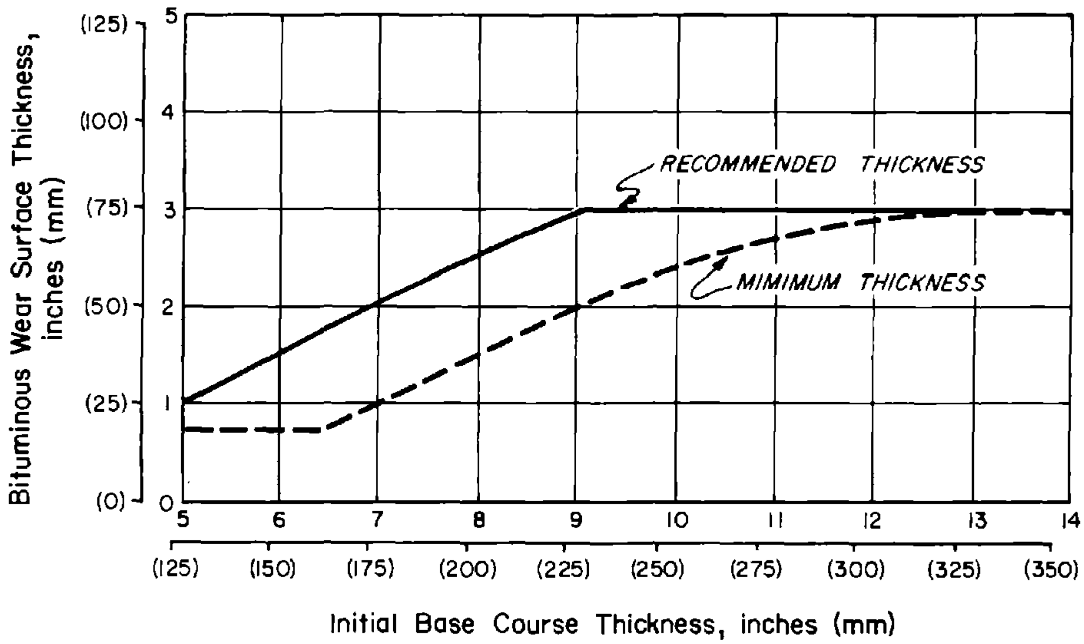

Figure 26. Bituminous wear surface thickness design chart.(6)

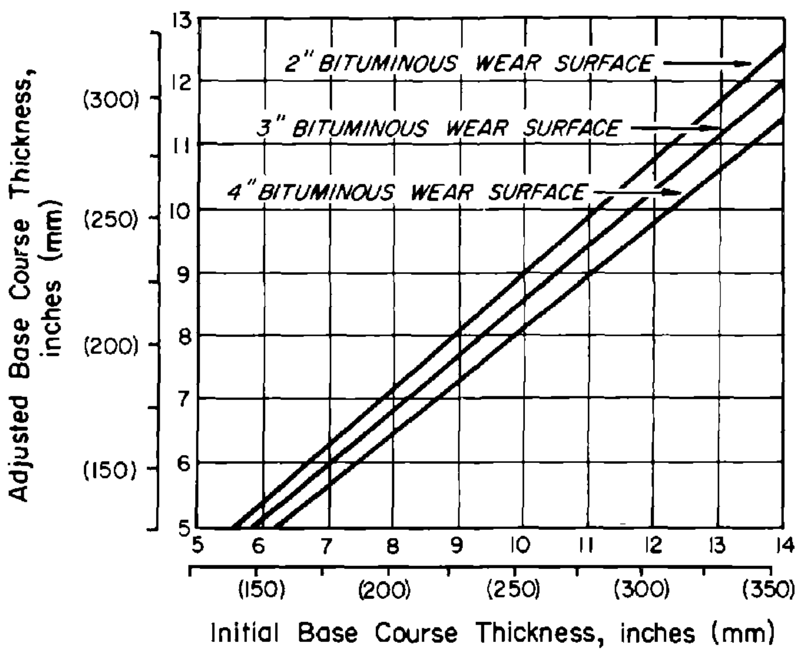

Figure 27. Adjusted base course thickness design chart.(6)

Adjusted Base Course Thickness

Although the actual structural contribution of the wear surface thickness has not been fully determined,(13) a reduction in the initial base course thickness can be made to account for the load-spreading capacity of the wear surface thickness. The adjusted base course thickness can be determined from Figure 27.

Pavement Design Example

The preceding thickness design procedure is illustrated in the following pavement design example.(6)

Given: Commercial Collector Street

ADT	=	3000 vehicles
% Heavy Trucks	=	3%
Annual Growth Rate, r	=	1.5%
Design Period, n	=	20 years
No. of Lanes	=	2
Subgrade Modulus, k	=	125 psi/inch (34 MPa/m)
Axle Load Dist'n	=	Pittsburgh (Table 11)

Step

1. No. of trucks/day in design lane = 3000 vehicles/day ÷ 2 lanes x 3% = 45 trucks/day
2. Projection factor = For r of 1.5% and n of 20 years, from Table 10 = 1.16
3. Total truck traffic in design lane during design period = 45 trucks/day x 1.16 x 365 days/year x 20 years = 381,060 trucks
4. Fatigue factor (see Table 14) = 1,700,000
5. Initial base course thickness, from Figure 25 = 9-1/4 inches (235 mm)
6. Bituminous wear surface thickness, from Figure 26 = 3 inches (76 mm)

Table 14. Example calculations for determining fatigue factor.[6]

AXLE LOAD GROUP kips A	AXLES PER 1,000 TRUCKS B	AXLE LOADS IN DESIGN PERIOD (B X Total Trucks/1000) C	FATIGUE CONSUMPTION COEFFICIENT D	FATIGUE EFFECT (C X D) E
		(a) Single Axles		
14-16	81.8	31,171	0.16	4,987
16-18	86.9	33,114	1.00	33,114
18-20	36.8	14,023	5.2	72,920
20-22	19.4	7,393	23.3	172,257
22-24	6.32	2,408	93.0	223,944
24-26	1.84	701	337.0	236,237
26-28	0.24	91	1,130.0	102,830
		(b) Tandem Axles		
24-26	67.6	25,760	0.107	2,756
26-28	67.6	25,760	0.341	8,784
28-30	67.6	25,760	1.00	25,760
30-32	40.6	15,471	2.74	42,391
32-34	22.1	8,421	7.1	59,789
34-36	10.3	3,925	17.5	68,688
36-38	2.2	838	41.1	34,442
38-40	2.9	1,105	93.0	102,765
40-42	0.32	122	203.0	24,766
42-44	0.32	122	431.0	52,582
44-46	0.22	84	890.0	74,760
46-48	0.16	61	1,790.0	109,190
48-50	0.16	61	3,530.0	215,330
			Fatigue Factor	1,668,292
			Round off to	1,700,000

7. Adjusted base course thickness, from Figure 27 = 8 inches (203 mm)

8. Final pavement configuration:

Stabilized Fly Ash Base Course	=	8 inches (203 mm)
Bituminous Wear Surface	=	3 inches (76 mm)

E. Construction Procedures

Construction procedures are based on techniques developed for stabilized fly ash pavements in Europe and demonstration projects in the United States.(6) The following construction procedures, although specified for base course construction, are applicable to subbase course construction as well.

Materials

1. Fly Ash

The fly ash to be used for the construction project should be from a source which has been tested recently in accordance with the laboratory testing program outlined previously for either cement or lime stabilization, whichever the case may be; and should satisfy the appropriate strength criteria described therein when mixed with a certain amount of stabilizer and a certain percentage of moisture necessary to achieve the maximum density of the mix in accordance with AASHTO T 134-74. The moisture content of the fly ash as-received should be determined and compensated for in the mixing operations.

2. Water

Water to be used in the construction mix should be clean and free of vegetable matter, acid, oil, alkali, sugar, or other substances harmful to the finished product.

3. Cement or Lime

The particular type of cement or lime specified for a project should be the same type as used in the laboratory testing program. Cement most often specified for stabilization meets the requirements of ASTM C 150-73a for Type I. Lime which meets the requirements of ASTM C 207-49 for Type N is often specified for stabilization, although Type L limes have also been successfully used.

4. Construction Mix

The various proportions of the materials comprising the base course construction mix are most conveniently specified in terms of pounds (kg) of fly ash and either cement or lime, and gallons (m^3) of water per cubic foot (m^3) or square yard (m^2) of compacted mix. The amount of water specified should be equivalent to the optimum moisture content of the fly ash-stabilizer mix plus two percent as determined by AASHTO T 134-74.

Subgrade Preparation

The subgrade should be shaped to the desired crown and grade and proofrolled to the degree of compaction necessary to produce the subgrade strength used in the design procedure. The subgrade should be moist in order to prevent absorption of moisture from the base course, but not wet. Any unsuitable material; ruts; and soft or wet areas caused by improper drainage, equipment, or any other cause should be removed, and the area backfilled with suitable material and compacted.

Mixing

A number of mixing methods can be used for stabilized fly ash. Generally, these methods can be classified as central mixing methods and mix-in-place methods.

The first method, central mixing, is usually done with a concrete-type batch mixer or a pugmill mixer and offers a high degree of quality control. A permanent batch plant may be available at the power station or at a concrete central batch plant. Pugmills or small concrete-type batch mixers can also be brought on-site. An example of an on-site pugmill mixing operation is shown in Figure 28. Accurately controlled amounts of fly ash, cement or lime, and water are introduced into a mixing chamber where they are mixed until a uniform composition is obtained. Batching by weight is generally recommended as opposed to batching by volume. The mixture is then hauled to the construction site in covered trucks to minimize evaporation losses and protect against sudden rainfall.

The second method, mix-in-place, involves the distribution of fly ash evenly over the work area, the distribution of cement or lime over the fly ash, and the subsequent addition of moisture and mixing with travelling mixing machines, harrows, or a similar apparatus. While this may

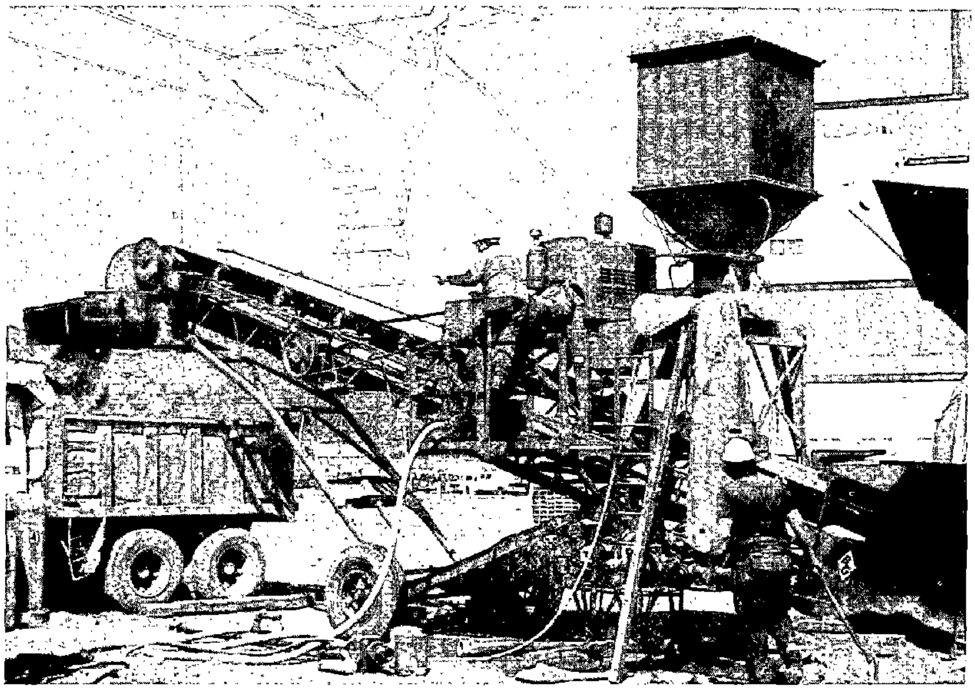

Figure 28. Typical pugmill operation for central mixing of cement-stabilized fly ash mixtures.

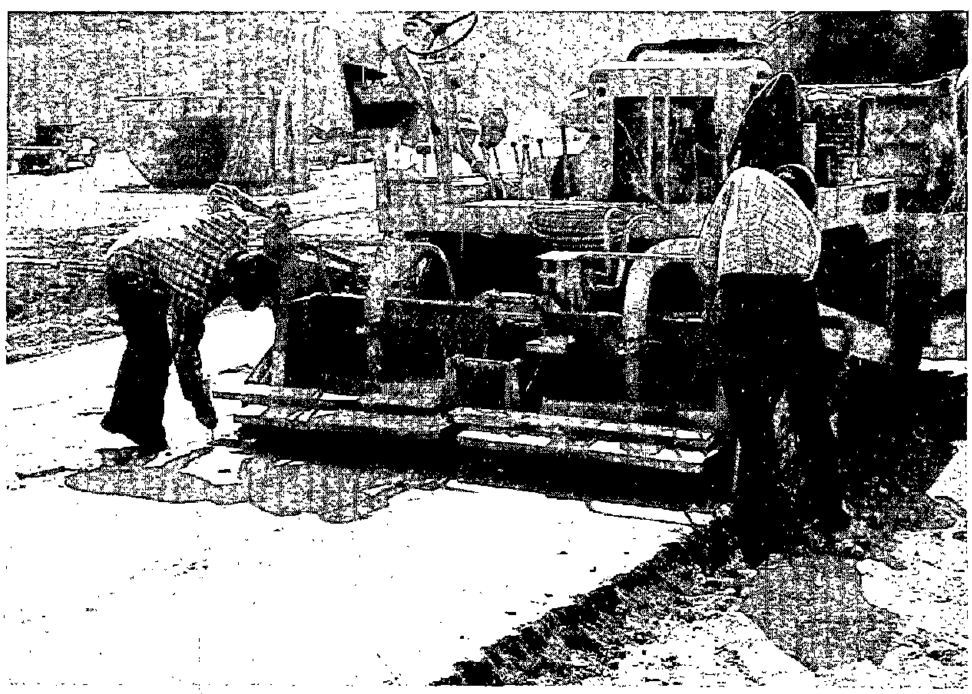

Figure 29. Use of an asphalt paver to spread cement-stabilized fly ash layer.

be the simplest mixing method, it is the least desirable due to the non-uniformity of the resultant mixture and the potential environmental problems of "dusting" by dry cement, lime, or fly ash. If this method must be used, the fly ash should be watered adequately before spreading to prevent dusting, and additional cement or lime should be added to compensate for any that might be blown away during spreading. Mixing of the base course by the windrow method is not recommended.

Spreading

If the subgrade is dry at the time of spreading, it should be moistened in order to prevent the absorption of moisture from the base course. The base course should not be spread when the temperature of either the subgrade or the base course is less than 40° F (4° C).

At the time of spreading, the moisture content of the base course mixture should be two percentage points over optimum to compensate for moisture loss during spreading and compaction operations.

For all mix-in-place methods, the spreading and mixing operations are combined. When central mixing methods are used for roadway construction, the mixture is spread by means of mechanical spreaders, such as a jersey box or asphalt paver, as illustrated in Figure 29. In the case of parking lot construction, the mixture can be tailgated from dump trucks and spread by a dozer. In either case, spreading methods should result in a uniform, uncompacted layer the compacted thickness of which equals the required design thickness. The uncompacted thickness necessary to produce the required design thickness can be determined in a test strip. The compacted thickness of a single layer should not exceed 8 inches (203 mm). For cases where the required compacted thickness is in excess of 8 inches (203 mm), the base course should be constructed in multiple layers. The value of maximum recommended thickness may vary with the type of spreading and compaction equipment used.

Spreading should progress so that no more than 30 minutes elapse between adjacent passes. A construction joint should be formed along the edge of the previous pass if more than 30 minutes elapse between adjacent passes. Spreading operations should be terminated during periods of rain.

Compaction

No more than 60 minutes should elapse between the start of moist mixing, on-site or off-site, and the start of compaction operations. Ideally, the compaction equipment should follow immediately behind the spreading equipment.

It is recommended that the uncompacted layer receive at least one pass by a track vehicle, such as a dozer, prior to allowing regular compaction equipment on the base course. In the case of parking lot construction, this has already been accomplished by the dozer-spreading operation.

The British have obtained their most satisfactory compaction results with pneumatic-tired rollers of the 10-ton (9 Mg) adjustable variety, either self-propelled or towed. Vibrating rollers with 1 to 1-1/2 tons (0.9-1.4 Mg) dead weight have been used with only slightly less satisfactory results. When larger production rates are required, larger vibratory rollers of the 6 to 10 ton (5.5-9 Mg) variety can be used. The speed of the vibratory roller should not exceed two to three miles per hour, and the rate of vibration should be checked with a vibrometer to achieve optimum efficiency. Care should be taken with vibratory rollers not to overstress the surface. Roughly four to eight passes are necessary to achieve the desired compaction. The suitability of a particular piece of compaction equipment and the actual number of passes required should be verified on a test strip.

The minimum dry density requirement that should be specified for stabilized-fly ash base course mixes is 100 percent of maximum dry density as determined by AASHTO T 134-74.

Any uncompacted or partially compacted base course mixture which is left undisturbed for more than two hours or is wetted by rain so that the average moisture content is more than two percentage points over optimum should be removed and replaced.

Finishing

If necessary, the base course should be fine-graded with a motor patrol. The surface should then be scarified and proofrolled to insure a finished surface free of ridges, cracks, ruts, and compaction planes.

Joints

Straight transverse and longitudinal joints should be formed at the end and edges of each day's construction by cutting back into the completed work to form a true vertical face free of loose or shattered material. All material resulting from the trimming operation should be removed from the area prevent mixing with fresh base course material. When the bituminous wear surface is constructed for a roadway, it should be placed so that the wear surface joints coincide with the base course longitudinal joints.

The engineer may consider the sawcutting of roadway pavement joints at regular intervals to control reflective cracking that may occur as a result of shrinkage cracks in the base course.

Multiple Layers

If the specified compacted thickness of the stabilized fly ash base course is greater than 8 inches (203 mm), it may be necessary to construct the base course in multiple layers in order to insure proper spreading and/or adequate compaction, with no compacted layer less than 4 inches (102 mm) in thickness. Each layer should be scarified prior to constructing another layer on top of it. If the upper layer is not constructed the same day as the lower layer, the lower layer should be cured until the upper layer is constructed, for a period of up to seven days. Central mixing methods are recommended for multiple layer construction, as mix-in-place methods can produce a thin zone of inadequately stabilized fly ash between the upper and lower layers.

Curing

Once the base course has been constructed, it is desirable to construct the bituminous wear surface immediately. If it is not feasible to do this, provisions should be made to protect and cure the base course until the bituminous wear surface is constructed, or for a period of up to seven days. Either of two curing materials described below should be applied within 30 minutes of the completion of finishing operations and after the surface of the base course has been broomed free of all loose and foreign material and/or moistened and rolled to integrate loose and dry surface material:

1. A bituminous curing material, preferably a rapid-curing seal coat, such as RS-1, can be applied at the rate of 0.15 to 0.30 gallons per square yard (0.00068-0.00136 m^3/m^2) as illustrated in Figure 30. If necessary, sufficient water to fill any surface voids in the base course should be applied immediately before the application of the bituminous curing material.

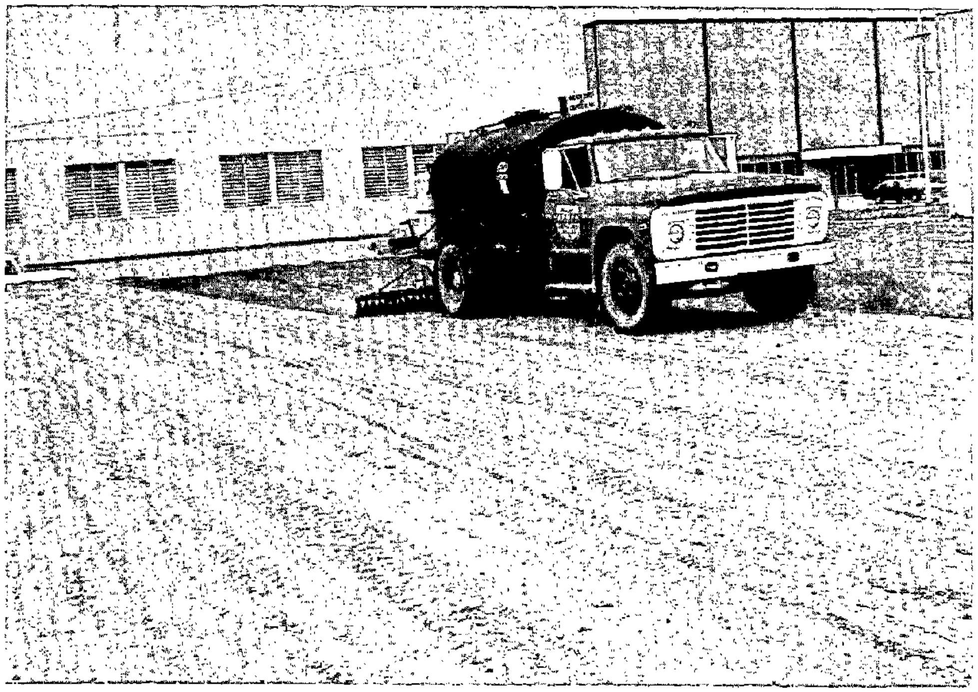

Figure 30. Application of bituminous seal coat on finished cement-stabilized fly ash base course.

2. A moist cure can be applied in lieu of a bituminous curing material using a water truck or other approved means to spray the surface of the base course with water at regular intervals during the daylight hours to prevent drying of the surface. At no time should the moisture content of the surface of the base course be allowed to fall below the optimum moisture content.

Traffic

No traffic should be permitted on the pavement until the bituminous wear surface has been constructed. In addition, if the wear surface is constructed less than a week after base course construction, traffic should not be permitted on the pavement until seven satisfactory curing days have elapsed since the construction of the base course. A satisfactory curing day is any day when the temperature of the completed base does not fall below 50°F (10°C). If at all possible, longer curing periods should be provided for lime-stabilized fly ash base courses.

Test Strips

It is recommended that a test strip be constructed prior to actual construction of parking lots or roadways in order to verify compaction criteria and evaluate the adequacy of the compaction equipment to be used on the job. The satisfactory test strip remains in place and becomes a section of the completed roadway or parking lot. Additional test strips should be constructed when there is a change in either compaction equipment or material (type or source).

The material and equipment used for the test strip should be exactly the same as that being used for the overall project. The method of construction (i.e., mixing, spreading, compacting, etc.) should also be the same. After an initial number of passes by the compaction equipment, and after each pass thereafter, density measurements should be taken to determine the effect of each additional pass of equipment. If the specified compaction requirements cannot be met within roughly eight passes, it would be advisable from an economic standpoint to either increase the pressure (weight) of the equipment being used or change the type of equipment in use rather than continuing to increase the number of passes. A minimum test strip area of 200 square yards (167 m^2) is recommended.

Tests

During construction of the stabilized fly ash base course, a certain number of tests on the base course material are recommended for quality control purposes. The samples for testing should be taken in accordance with good sampling techniques. The tests are as follows:

1. Moisture-Density Relationship (AASHTO T 134-74)

2. Moisture Content (AASHTO T 239-73 or ASTM D 2216-71)

3. Cement Content of Freshly Mixed Base Course Material (ASTM D 2901-70) or Lime Content of Freshly Mixed Base Course Material (AASHTO T 232-70).

4. In-place Density (AASHTO T 238-73, AASHTO T 205-64, or AASHTO T 191-61)

5. Unconfined Compressive Strength: if cement is used, 7- and 28-day; if lime is used, 28- and 45-day (AASHTO T 22-74)

6. Depth of Mixing for Mix-in-place Methods (visual).

For convenience, Proctor specimens molded in the field at maximum dry density and optimum moisture content can be used for the unconfined compressive strength tests, with the strength results factored for the appropriate ℓ/d ratio.

Construction Cut-Off Dates

The general guideline for determining the construction cut-off date for stabilized fly ash base course is that the ambient air temperature should not fall below 50°F (10°C) for a period of seven days following completion of the base course. The pozzolanic reaction in the base course material ceases at temperatures below 40°F (4°C), although it continues once the temperature is increased. If construction takes place early or late in the season, or if unseasonably cold weather occurs during the curing period, the base course should be protected from freezing by a covering of suitable material, such as hay or straw, and consideration should be given to delaying the opening of the finished pavement to traffic.

In the mid-Atlantic states, the recommended construction period for cement-stabilized fly ash base/subbase course is April 15 through October 15. For lime-stabilized fly ash, the recommended construction period in the mid-Atlantic region is mid-May to late September or mid-October. It is suggested that the engineer refer to the construction specifications of his respective state highway department for construction cut-off dates for lime-pozzolan-aggregate or soil-cement. These dates can be safely applied to cement-stabilized fly ash base/subbase courses, and can be easily adjusted for slower-curing lime-stabilized fly ash base/subbase courses.

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V. FLY ASH AND LIME-FLY ASH IN STABILIZED SOIL PAVEMENTS AND SOIL MODIFICATION

A. Factors Affecting Utilization

Considerable research, testing, field trials, and actual construction projects have been done with lime-fly ash-aggregates (see Section III). These mixtures, used for base and subbase construction, generally entail the use of good quality coarse- and fine-grained aggregates, either native to the site or borrow material. However, much research has also been done on the use of lime-fly ash-soil mixtures incorporating fine-grained soils, such as silts and clays, which occur naturally at the site. The resulting mixtures, if designed to be economically competitive with other methods of construction, are not usually as high quality as the lime-fly ash-aggregate mixtures, in part because of the initial lack of mechanical stability in the unstabilized soils and the greater tendency towards frost-susceptibility in fine-grained soils. Nevertheless, the lime-fly ash-soil mixtures have been found to be highly serviceable and economical in three areas of roadway construction:

- Base course for secondary roads, parking lots, etc., where heavy traffic loads are not anticipated;
- Subbase beneath conventional pavements; and
- Subgrade improvement to provide additional support for the pavement and/or remedy undesirable subgrade conditions to expedite construction.

Thus, this section presents the details associated with design and construction of lime-fly ash-soil mixtures incorporating primarily fine-grained soils. In cases where stabilization of granular materials is desired, reference should be made to Section III.

A limited amount of testing and a few field trials have been undertaken using fly ash alone as a stabilizer. The results have been very promising; thus, this method of soil stabilization is also included in this section. In this case, both granular and fine-grained materials are discussed.

Soil stabilization generally refers to the physical and/or chemical methods used to improve natural soils or soil-aggregates for use in some engineering application.

Soil stabilization is used predominantly in the construction of roadways, parking areas, runways, and foundations.[19] Soil stabilization can eliminate the need for expensive borrow materials; expedite construction by improving particularly wet or unstable subgrade; effect savings in pavement thicknesses by improving subgrade conditions; and permit the substitution in the pavement cross-section of low cost materials for conventional and less economical materials.

Chemical stabilizers which have been used in the past include cement, lime, lime-fly ash, bituminous materials, and chemicals such as calcium chloride and sodium chloride.[20] Among these, the first three have been used most extensively.

There are two primary mechanisms by which these three stabilizers can render a soil more suitable for engineering applications: (1) improvement of the soil's inherent properties, such as decreasing its plasticity index, improving drainage characteristics, and decreasing volumetric shrinkage;[18] and (2) cementation of the individual soil grains to produce an increase in both strength and durability characteristics. In lime stabilization, a marked improvement in the properties of many soils occurs. In addition, the lime reacts with any pozzolanic material present in the soil, resulting in cementation action.[10] In cement stabilization, the cement reacts with water to produce certain compounds which are effective in cementing the soil particles together.[10] The strength development does not depend upon chemical interaction between the cement and the soil. However, lime which is generated during the cement hydration process may modify the soil's properties and can react with any pozzolanic material in the soil to produce further cementation. Lime-fly ash stabilization produces improvements in soil properties, due to the presence of lime, and also results in significant cementation due to the reaction between lime and the pozzolanic fly ash. The cementation process is somewhat similar to that in cement stabilization in that the process does not depend on interaction with the soil, although the presence of pozzolanic material in the soil can modify the lime-fly ash reaction.[10]

As a result of the low cost of fly ash and its excellent pozzolanic properties, there are many cases in which lime-fly ash stabilization is more advantageous than lime or cement stabilization. Depending, of course, on the soil type, lime-fly ash stabilization can produce greater strengths and improved durability when compared to lime stabilization. In locations where lime is cheaper than cement, lime-fly ash stabilization can often produce material of comparable long-term strength and durability at a reduced cost when compared to cement stabilization.

As indicated previously, stabilization of soils with fly ash alone is still in the developmental stages, although it shows promise of becoming a very effective and economical technique. This method is most successful when fly ash with self-hardening properties is used (see Section II). In general, fly ash of this type has a high free lime content. This free lime produces favorable changes in the properties of several soil types and, in addition, reacts with the siliceous and aluminous compounds in the fly ash to produce a very effective cementing action.

B. Case Histories

A number of states have reported testing programs, test sections, or actual construction projects utilizing fly ash in soil stabilization. Among these are Alabama, North Dakota, Iowa, Virginia, Missouri, West Virginia, Arizona, and Maryland. As is the case with most other forms of fly ash utilization in highway construction, this application has not been in widespread use in this country. Not much literature is available on this use in other countries, with the exception of England, where fly ash has been used successfully as a soil modifier.

A number of case histories of trial and actual projects does indicate the great potential for fly ash in soil stabilization and modification. A few are reported below.

Lime-Fly Ash-Soil Base for Roadway
Gorgas to Parrish, Alabama

In November of 1960, construction began on an 8-mile (13 km) section of roadway in Alabama between the towns of Gorgas and Parrish.[11] The in-place soils varied from a dense mass of clay and weathered yellow shale to a mixture of kaolinite and fine sand. A mixture of 15 percent Gorgas fly ash and five percent hydrated lime was added to the soils by mix-in-place methods. Two 4-inch (102 mm) layers were constructed, compacted to maximum density, and surfaced with asphalt. The ensuing traffic on the road was very heavy. Loaded coal trucks and a high volume of passenger vehicles used the road daily. The pavement has given very good service.

Lime-Fly Ash Subgrade Modification
Interstate 29, North Dakota

It was feared that prolonged rainfall would render the

A-7 subgrade soil unworkable along portions of the I-29 construction route in eastern North Dakota.[14] In order to expedite construction and prevent work delays during the relatively short construction season, three percent lignite fly ash and three percent hydrated lime were spread on the subgrade, disced, and mixed with water using a single-pass mixer-pulverizer. The mixture was allowed to "mellow" one to two days, allowing the lime reaction to reduce the plasticity index of the soil. The compacted thickness of the modified subgrade layer was 6 inches (152 mm). No structural value was assigned to the modified layer. A pavement consisting of 8 inches (203 mm) of continuously reinforced Portland cement concrete and 3 inches (76 mm) of asphaltic concrete was constructed over the modified subgrade. Figures 31 and 32 show various phases of the construction of I-29.

Fly Ash Subgrade Modification
Industrial Road, Kansas City, Missouri

The clay subgrade beneath a road in an industrial area near Kansas City, Missouri, was highly plastic with a liquid limit of 65 percent, plasticity index of 43 percent, and California Bearing Ratio of 3.5 percent.[13] The resultant pavement design was 12 inches (305 mm) of full depth asphalt, or the equivalent. It was decided to reduce the required pavement thickness by improving the subgrade. Laboratory investigations indicated that 15 percent fly ash from Hawthorne Power Station was effective in reducing the plasticity index to 18 percent and the liquid limit to 45 percent. There was little change in the laboratory unconfined compressive strength at zero days, but at 28 days the strength of the clay-fly ash mixture was seven times that of the clay alone. The fly ash and clay were mixed on-site by mix-in-place methods. Two layers 4-1/2 inches (114 mm) each in thickness were constructed. Construction of the modified subgrade is shown in Figures 33 through 35. The original asphalt pavement thickness was reduced from 12 inches (305 mm) to 9 inches (229 mm). Field CBR values for the clay-fly ash subgrade were 9 percent unsoaked and 12.5 percent soaked. After a period of two years, the pavement, shown in Figure 36, was still giving excellent service under light traffic.

Fly Ash Moisture Modification of Embankment Borrow Material, A.27 Trunk Road Improvement, Arundel, Sussex, England

A 60-foot (18.3 m) high embankment for the A.27 Trunk Road near Arundel, Sussex, was being constructed of borrow material composed of hoggin, sand, running sand, sandy clay,

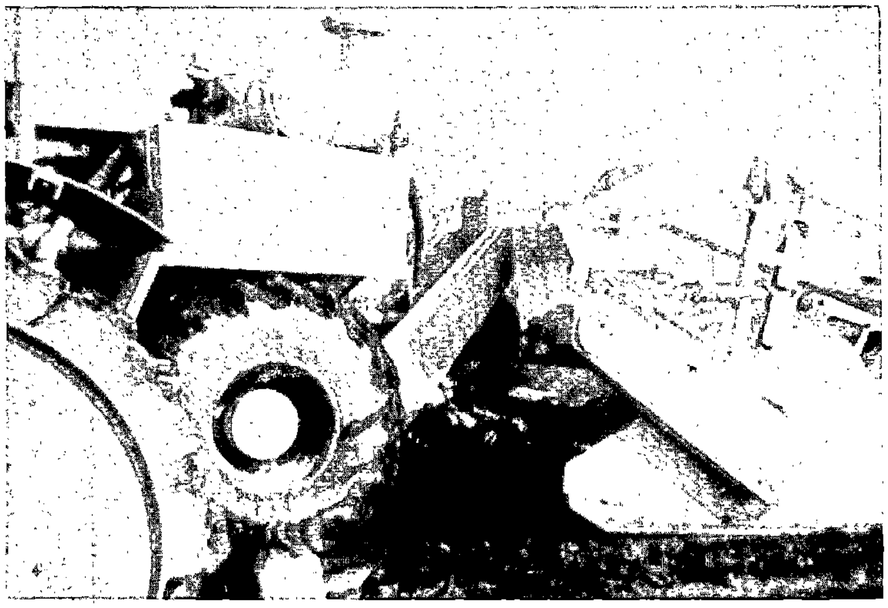

Figure 31. Mixing fly ash, lime, and soil with single-pass mixer-pulverizer on I-29.

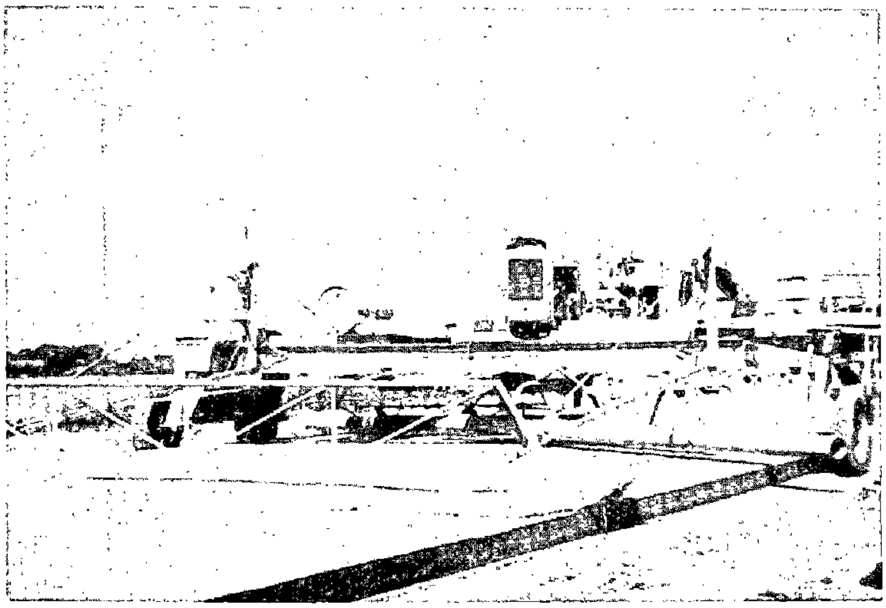

Figure 32. Finishing continuously reinforced Portland cement concrete pavement constructed on lime-fly ash-modified subgrade along I-29.

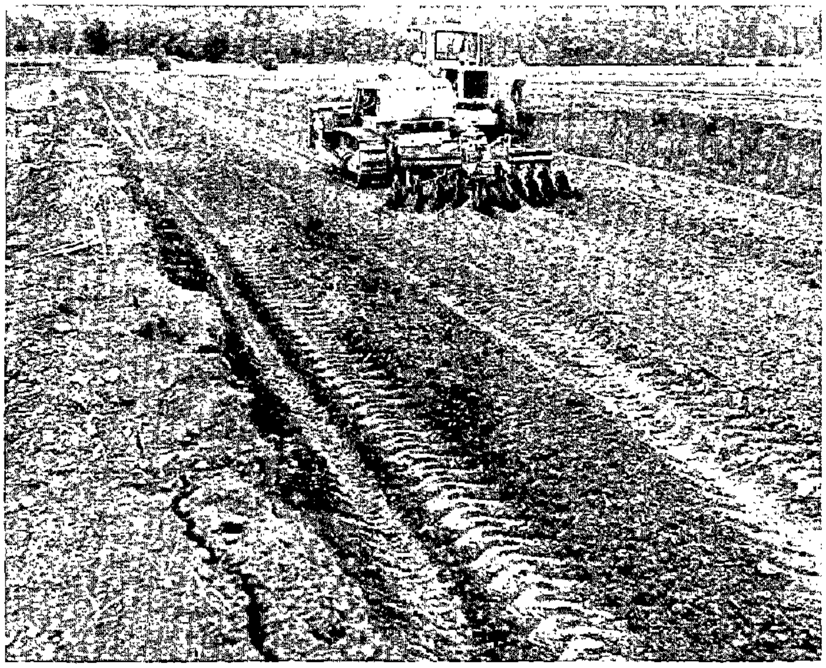

Figure 33. Fly ash being disced into highly plastic clay subgrade for industrial access road.

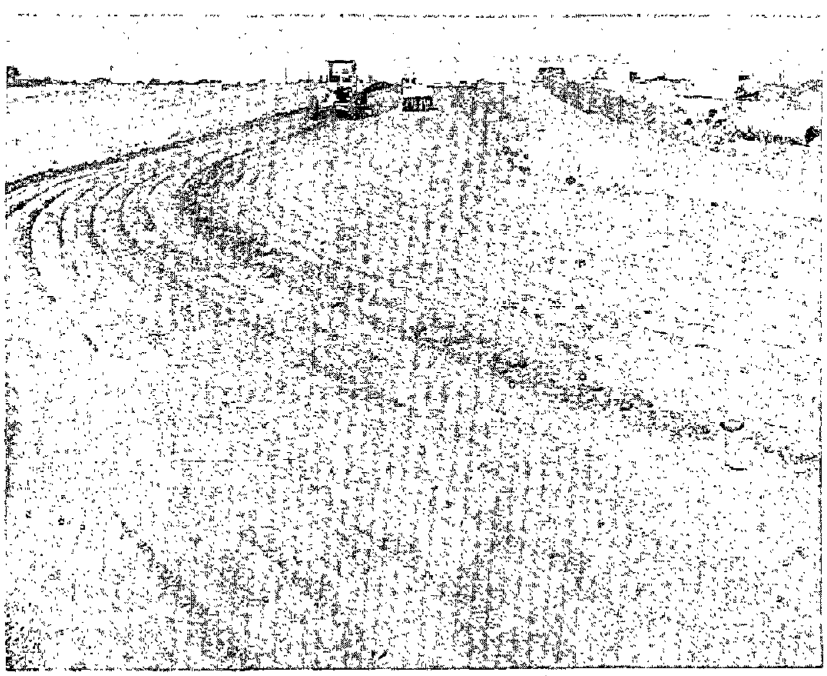

Figure 34. Compaction and grading of fly ash-modified subgrade for access road.

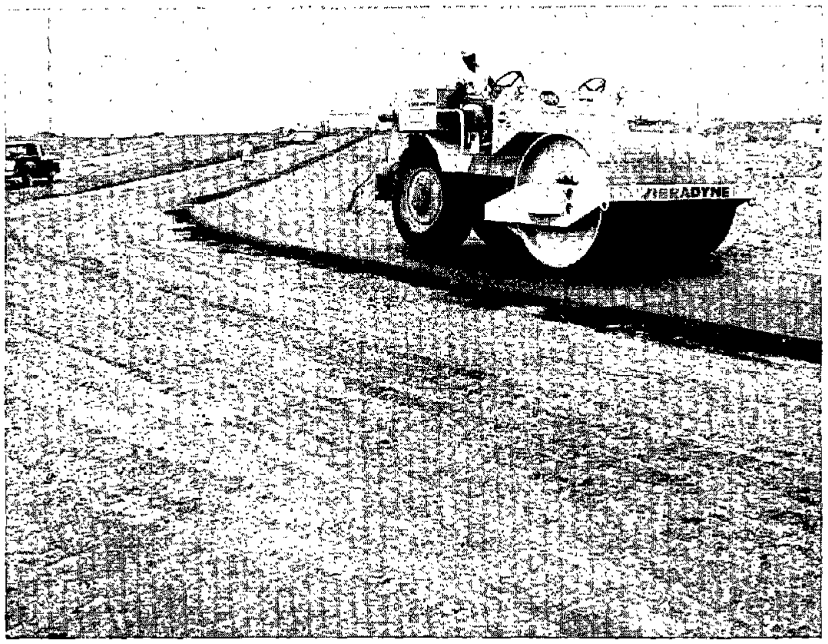

Figure 35. Construction of asphalt pavement on fly ash-modified subgrade.

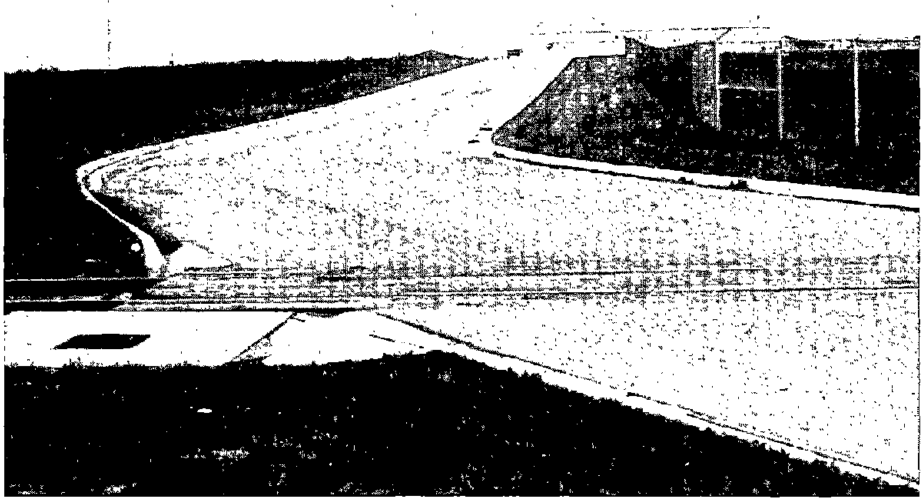

Figure 36. Completed industrial access road on fly ash-modified subgrade two years after construction.

and clay.[3] Inclement weather resulted in a very high moisture content in most of the borrow materials. The materials became so unworkable and difficult to place that the job came to a standstill. It was decided to apply fly ash to the embankment materials in an effort to improve their workability. About 13,000 cubic yards (10,000 m^3) of fly ash from Brighton "B" Power Station was brought to the site. Only enough water had been added to the fly ash to prevent dusting. The fly ash was stockpiled at one end of the site. As each layer of material was spread, fly ash from the stockpile was deposited over the surface in 3- to 4-inch (102 mm) depths by the scrapers on their return journey. Bulldozers then tracked the fly ash into the underlying material. The fly ash was successful in absorbing excess moisture from the fill material, thereby reducing its moisture content and plasticity. It was then possible to carry out compaction procedures. As a result, construction of the embankment continued into November.

C. Lime-Fly Ash-Soil Base and Subbase Courses

Strength and Durability Criteria

In general, fine-grained soils rarely have adequate strength for use as base or subbase courses in the pavement cross-section. In addition, the frost susceptible characteristics of most fine-grained soils make them unsuitable paving materials in areas where frost penetrates the ground. The addition of lime and fly ash, in many cases, greatly increases the strength of the soil and improves its durability characteristics to the point where it can be used economically as a base or subbase material.

The suitability of a lime-fly ash-soil mixture for use as a base or subbase material is usually determined by its unconfined compressive strength, although other measures such as shear strength, bearing value, or load deflection value, have been used.[2] The durability, that is, the mixture's ability to withstand potentially damaging freezing-thawing and wetting-drying action, is often measured in terms of weight loss, although other measures, such as strength reduction, absorption, and softening, have also been used.[2]

Much work has been done with soil-cement mixtures both in this country and abroad. Rather well-defined strength and durability criteria have been developed by the Portland Cement Association (PCA) and the British Road Research Laboratory (RRL). There are apparent similarities between soil-cement and lime-fly ash-soil mixtures which make it reasonable to assume that similar criteria could be adopted for selecting suitable lime-fly ash-soil mixtures. However, one important difference between the two materials prohibits direct application of soil-cement criteria to lime-fly ash-soil. Soil-cement criteria, which is based on the performance of the mixture after seven days, was developed on the basis that soil-cement attains approximately 50 percent of its ultimate strength in seven days.[10] The strength development in lime mixtures generally proceeds at a slower rate. It is estimated that lime-fly ash-soil mixtures attain only 10 percent of their ultimate strength in seven days. Therefore, seven-day criteria give very little indication of the eventual performance of these mixtures in the field. It is felt by many that criteria based on a 28-day curing period are more indicative of a lime-fly ash-soil mixture's suitability for base and subbase applications.[10,15,20]

The American Society of Testing Materials (ASTM) has developed a specification for the use of fly ash with lime in lime-fly ash-soil mixtures. This specification (ASTM C 593) establishes minimum unconfined compressive strength and durability requirements for mixtures utilizing coarse-grained soil. These requirements are often specified for projects where lime and fly ash are used to stabilize fine-grained soils. The unconfined compressive strength criterion of 400 psi (2760 kPa) in seven days under accelerated curing conditions has proven to be quite acceptable, except that recommendations have been made for reducing this requirement to as low as 100 psi (690 kPa) for subbase applications.[20] The accelerated curing at 100° F (38° C) produces a seven-day approximation of the 28-day strength of a mixture under ambient conditions.

The durability criterion in ASTM C 593 need be applied only in those geographic areas where frost penetration of the base or subbase is expected. ASTM C 593 originally based durability criteria on a freeze-thaw brushing test. However, it was felt by many that the test, which requires a sample to be brushed after each of 12 freezing-thawing cycles, is unduly abrasive to fine-grained soil mixtures and generally unrepresentative of field conditions.[10,20] Thus, ASTM has adopted the vacuum saturation testing method as a replacement for the freeze-thaw brushing test in ASTM C 593. The vacuum saturation method is discussed in Reference 8. Basically, a sample is subjected to a vacuum for a specified period of time and then soaked. The unconfined compressive strength of stabilized soil samples at the end of this test has been shown to correlate very well with the strength of samples which have been subjected to five or ten cycles of the freezing-thawing (but not brushed).[8] The criterion specified for the strength at the end of the test has been suggested as 400 psi (2760 kPa), which allows for essentially no loss of compressive strength between the start and the end of the durability test. The advantage of the vacuum saturation test is that it can be performed in about an hour, whereas the standard durability test requires about 24 days to perform.

Other durability criteria based on the compressive strength of a specimen after being subjected to some form of weathering test have been used.[8,10] One form of test developed for soil-cement is to subject a sample to the appropriate number of freeze-thaw or wet-dry cycles specified in AASHTO T 136-70 or T 135-70 without brushing the sample. The sample is tested in unconfined compression, and its strength then compared to the strength of a similar sample which has been cured for the same period of time under normal laboratory curing conditions. The durability ratio, which is the weathered strength divided by the unweathered strength, is used as a measure of the mixture's ability to remain durable. The British RRL utilizes a test similar to this, except that the conditions for freeze-thaw differ slightly from the AASHTO test, and samples are subjected to 14 cycles instead of 12.[10] The required durability ratio for acceptable performance is 80 percent. Iowa State University has developed the Iowa freeze thaw-test based on the same principle as the British test.[17] An index of resistance, similar to a durability ratio, of at least 80 percent is desired for climatic conditions in Iowa.

Table 15 summarizes the various strength and durability criteria which are applicable to lime-fly ash-soil mixtures to be used in base and subbase applications.

Mix Design Concepts

The strength and durability of a lime-fly ash-soil mixture is influenced by a number of factors as illustrated in Figure 37.[4] Those which are normally taken into consideration in the mix design process include:

1. soil type
2. fly ash type
3. lime type
4. proportion of stabilizers to soil
5. ratio of lime to fly ash

Table 15. Commonly used strength and durability criteria for lime-fly ash-soil mixtures in base and subbase courses.

AGENCY	TEST	CRITERIA
	(a) Strength	
ASTM	ASTM C 593 Unconfined Compression Test 7 day cure at 100° F (38°C)	min. 400 psi (2760 kPa)
British Road Research Laboratory	Unconfined Compression Test 28-day cure	min. 250 psi (1720 kPa), except 400-500 psi (2760-3450 kPa) for clay soils and severe climatic conditions
	California Bearing Ratio	80% immediately beneath surface, and decreasing with depth
	(b) Durability[1]	
ASTM	ASTM C 593 Vacuum Saturation Method[2]	min. 400 psi (2760 kPa)
Portland Cement Association	AASHTO T 135-70 and T 136-70 Wet-Dry and Freeze-Thaw Brushing Tests	7-14% allowable weight loss, exact value dependent upon soil grain size
British Road Research Laboratory	Durability Ratio (ratio of weathered strength to unweathered strength)	min. 80%
	Iowa Freeze-Thaw Test, Index of Resistance (ratio of weathered strength to unweathered strength)	min. 80%

[1]Applicable in regions where climatic conditions are a factor in pavement performance.

[2]Approved revision; replaces freeze-thaw brushing test; not published at time of report.

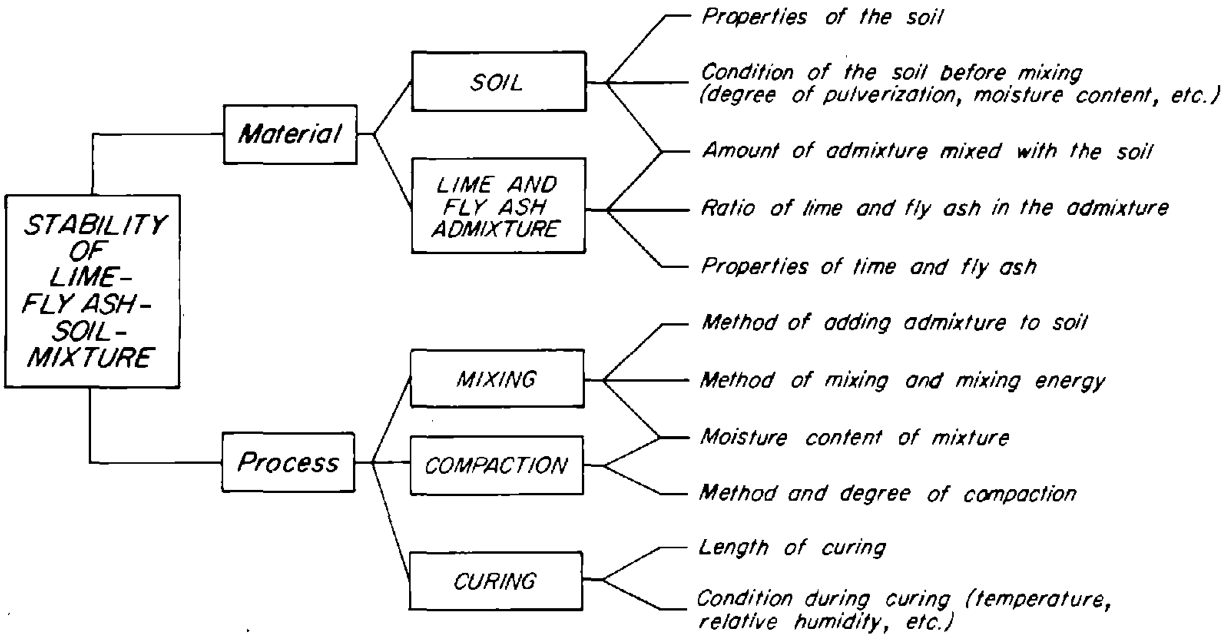

Figure 37. Factors influencing the quality of a lime-fly ash-soil mixture.[4]

6. dry density and moisture content of compacted mixture

7. age of mixture

8. temperature

Some of these factors are interdependent, to a certain degree. Each is briefly discussed below.

1. Soil Type

The addition of lime and fly ash has varying effects on different types of soil. In general, soils containing sulphates or organic matter are not suitable for stabilization.[10]

> Clays - Clays containing montmorillonite minerals react readily with lime, with the effect of immediate reduction in plasticity and gradual pozzolanic strength development. The lime-clay reaction, however, proceeds in preference to the lime-fly ash reaction. Fly ash is of greater benefit when higher percentages of lime are used.[5]

Clays containing illite, chlorite, vermiculite, or kaolinite may be slightly pozzolanic in themselves. Performance with lime can be improved with the addition of fly ash. Best ratios of lime to fly ash are usually in the range of 1:9 to 4:6.[5]

In areas where durability is not crucial, the total proportion of lime and fly ash is governed by economics and usage. Where the treated clay is likely to be subjected to freeze-thaw action, the total lime-fly ash content necessary is in the order of 25 to 30 percent. The stabilized clays are more frost resistant at an early age than untreated clays.[5]

Silty Soils - Silty soils with less than 10 or 12 percent clay may be somewhat pozzolanic, depending upon mineral composition. In these cases, the best lime-fly ash ratio will be on the order of 1:2. Silty soils which contain sufficient amounts of montmorillonite clay will benefit from larger amounts of fly ash. Freshly stabilized silty soils in which strength gain has not yet occurred to any great degree are highly frost susceptible.[5]

2. Fly Ash Type

ASTM C 593 places two constraints on fly ash to be used in lime-soil mixtures. The requirements, which pertain to maximum allowable water soluble fraction and to gradation, are outlined in Table 16.

Table 16. ASTM C 593 requirements pertaining to fly ash for use with lime-soil mixtures.

ITEM	CRITERIA %
Water Soluble Fraction	max. 10
Fineness-amount retained when wet seived:	
No. 30 (595 μ) sieve	max. 2.0
No. 200 (74 μ) sieve	min. 30.0

Research has established that several chemical and physical characteristics of fly ash can be used as indicators of how well a fly ash performs in a lime-fly ash-soil mixture. The best physical indicator is the fineness, or specific surface, as determined in accordance with ASTM C 311-68. The finer the fly ash particles, the greater the rate of pozzolanic reaction. The British recommend a minimum fineness of 2,750 cm^2/gm,[10] although this lower limit has not been used in most American work.

The most important chemical indicator is the carbon content of the fly ash, measured as a loss-on-ignition in accordance with ASTM C 311-68. High carbon content tends to inhibit the pozzolanic reactivity of a fly ash as well as decrease its density. The British recommend a maximum loss-on-ignition of seven percent.[10] Researchers in this country, however, tend to use an upper limit of ten percent loss-on-ignition as still being an indicator of good cementation potential with lime.[7]

High calcium oxide (CaO) contents have been found to be very advantageous in stabilization studies.[9,13,15,21] A high CaO content on the order of ten percent or greater is usually indicative of the presence of substantial amounts of free lime, which not only has a beneficial effect on a soil's physical properties, but which reacts with the siliceous and aluminous compounds in the fly ash to produce cementation. In several cases, fly ashes with high CaO contents have been used to satisfactorily stabilize soils without the use of lime.[13,15,21]

3. Lime Type

The two classes of lime which are used in soil stabilization are hydrated lime [$Ca(OH)_3$] and quicklime (CaO).[19] Limes with very high calcium contents are known as "calcitic", and limes containing magnesium oxide are known as "dolomitic". Both major classes of lime have been found to be successful in changing soil plasticity.[5] However, there is a certain disadvantage to the use of quicklime in soil stabilization in that quicklime is highly caustic.[5] Great care must be taken by workers during construction. Thus, slurry methods for quicklime are generally preferred to dry application.

In base and subbase course application, particular types of lime appear to give better performance than others. Dolomitic monohydrate lime [$Ca(OH)_2 . MgO$] generally gives better strengths in lime-fly ash-soil mixtures than calcitic hydrated lime [$Ca(OH)_2$] in normal amounts and when cured under ambient conditions,(17) except with some kaolinitic soils where strengths are approximately the same (although this may vary with fly ash type).(2) For clayey soils, calcitic hydrated lime is a more effective stabilizer at low lime contents on the order of three percent, but at higher lime contents, dolomitic monohydrate lime produces greater strengths.(17)

4. Proportion of Stabilizer to Soil

Unlike granular soils for which an optimum amount of stabilizer can be selected on the basis of maximum dry density, there appears to be no optimum amount of stabilizer for fine-grained soils.(9,18,19) For a given lime to fly ash ratio, the strength of the lime-fly ash-soil mixture increases as the total amount of stabilizers is increased.(4) This particular behavior is illustrated in Figure 38 for a Texas clay. The selection of a total stabilizer percentage is usually based on economic considerations.

5. Ratio of Lime to Fly Ash

Generally speaking, mix proportions are seldom critical for a given soil, and a number of mixes could be used which would result in approximately the same mixture performance.(5) The problem becomes one of choosing the most economical combination which satisfies specified criteria.

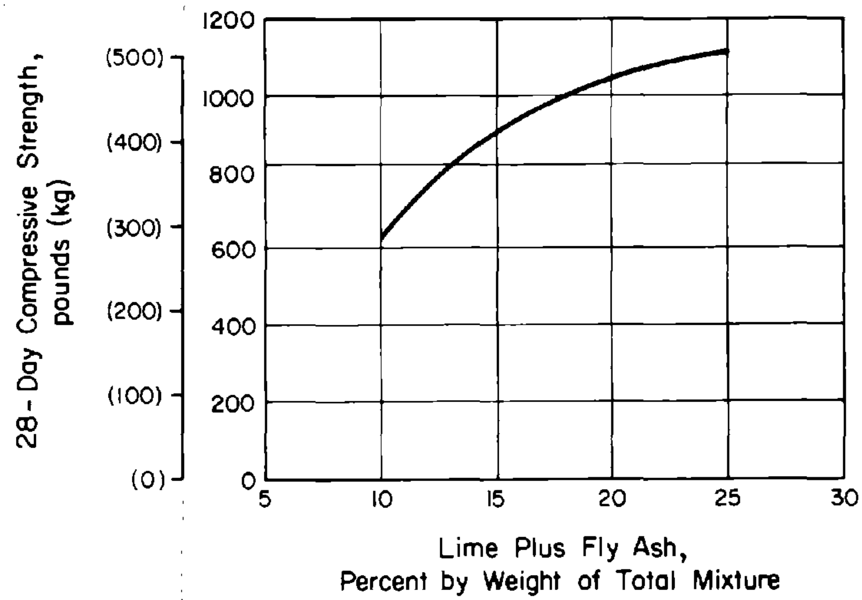

Figure 38. Variation of 28-day compressive strength with lime plus fly ash content (strength is expressed as total load).(4)

General guidelines for fine-grained soils are lime contents of five to nine percent and fly ash contents of 10 to 25 percent.[17] The most practical range of ratios of lime to fly ash appears to be from 1:9 to 1:1. Soils containing expansive clays will require the larger lime to fly ash ratios to ensure that there is adequate lime for both the lime-clay and lime-fly ash reactions.

6. Dry Density and Moisture Content of Compacted Mixture

The compressive strength of a lime-fly ash-soil mixture increases with increasing density to a point near maximum dry density as determined by the standard Proctor test (AASHTO T 99-74). Experiments have shown that, for most lime-fly ash-soil mixtures, maximum compressive strength is obtained at a moisture content slightly less than the optimum moisture content required for maximum dry density.[4] In the case of soils containing montmorillonite clay, the moisture content required for maximum compressive strength is slightly greater than the optimum moisture content for maximum dry density. Generally, however, the difference between the two moisture contents is not great, and since the optimum moisture content for maximum density is more easily determined, this moisture content is usually used in the laboratory testing program for determining mix design.

Studies have shown that densities above those produced by the standard Proctor method result in greater durability for lime-fly ash-soil mixtures.[12] In addition, compaction of lime-fly ash-silt and lime-fly ash-clay mixtures at modified Proctor densities (AASHTO T 180-74) actually resulted in greater strength gains during wetting-drying tests than during standard moist curing over an equal period of time. Mixtures which are not durable at standard Proctor densities might be made durable through greater compactive effort. Thus, if it is anticipated that field compaction in excess of standard Proctor can be economically obtained, it may be advantageous to specify these greater densities in design and construction.

7. Age of Mixture

Strength development in lime-fly ash-soil mixtures occurs at a slower rate than in soil-cement mixtures, but continues over a long period of time. It is estimated that about 10 percent of a lime-fly ash-soil mixture's ultimate strength is developed in seven days' time[10] under normal curing conditions (moist cure at 70° F or 21° C), and about 50 percent of its ultimate strength will be attained at the end of 28 days.(17) This rate of strength gain varies, of course, with the soil, lime, and fly ash and can continue for a period of years. Durability is likewise a function of the age, or curing period, of the mixture.

The greatest impact of the rate of age-hardening of lime-fly ash-soil mixtures is on the recommended construction period. The slower strength development of these mixtures makes it imperative that they be constructed during the warm months and at least a month prior to the onset of the first frost.(19) Certain accelerators can be added to the mixtures to increase the rate of strength development, thereby permitting construction to continue later in the season.(5,6,16) The best results to date have been achieved with the use of powdered sodium carbonate in quantities as low as one-half percent. Portland cement in low percentages can also be used as an effective stabilizer.

8. Temperature

As with any pozzolanic reaction, the cementation in a lime-fly ash-soil mixture proceeds more rapidly at higher temperatures and ceases at temperatures below 40° F (4° C). Warmer temperatures again reactivate the pozzolanic reaction after periods of cold weather, and the reaction proceeds until the chemical compounds participating in the reaction are depleted.

In the mix design process, it is recommended that all testing be based on a 28-day curing period at 70±3° F (21±2° C) and 100 percent relative humidity, where possible. It has been common practice in the past to test lime-fly ash-soil mixtures under an accelerated curing period of seven days at 140° F (60° C) as an approximation of the condition of the mixture at the end of a 28-day cure at 70° F (21° C). However, certain pozzolanic reactions may occur at higher temperatures and not at lower temperatures. In addition, the relationship between age, temperature, and strength is not the same for all lime-fly ash-soil mixtures.(5) Thus, strengths at the end of a 7-day high-temperature curing period may not be a good approximation of strengths after 28-days of curing at normal temperatures for all lime-fly ash-soil mixtures.

Laboratory Testing Program

The laboratory testing program for base or subbase course mix design can be divided into four steps:

1. trial mix selection

2. moisture-density tests

3. strength tests

4. durability tests

The specific tests to be performed in Steps 3 and 4 are a function of the design criteria selected by the designer in accordance with the recommendations presented previously and/or accepted practice in his state.

1. Trial Mix Selection

Due to the great variability among soils and fly ashes, guidelines more specific than those given previously have not been developed for determining the required proportions of soil, lime, and fly ash to produce a mixture which satisfies specific design criteria. Several general methods have been developed, however, for selecting trial mixes for testing. The most rigorous approach is preparation of trial mixes for every conceivable combination of the three materials within the general guidelines previously given.(5) The numerous trial mixes are then prepared and strength tested, the results being a grid of data. From the data, strength contours can be drawn and a range of satisfactory mixes selected for further testing and economic comparison. An example of the grid approach is shown in Figure 39.

Since one of the goals of any testing program is to develop an economical mix, it is possible to short-cut the testing program by initially eliminating all uneconomical trial mixes.(5) Basically, the method, known as the Iowa State equal-cost-line method, consists of establishing an upper limit on the amount of lime which can compete economically with other types of construction. This is Point A of Figure 40. The cost of handling the fly ash is estimated, expressed as a percentage of lime, and subtracted from Point A, yielding Point B. The equal-cost line is determined from the relative costs of lime and fly ash and represents the economic "ceiling" for combinations of lime and fly ash. A minimum three percent requirement is placed on the amount of lime as compensation for imperfect field mixing conditions. Trial mixes are then selected from within the resultant triangle BCD. The details of the equal-cost-line method are contained in Appendix B.

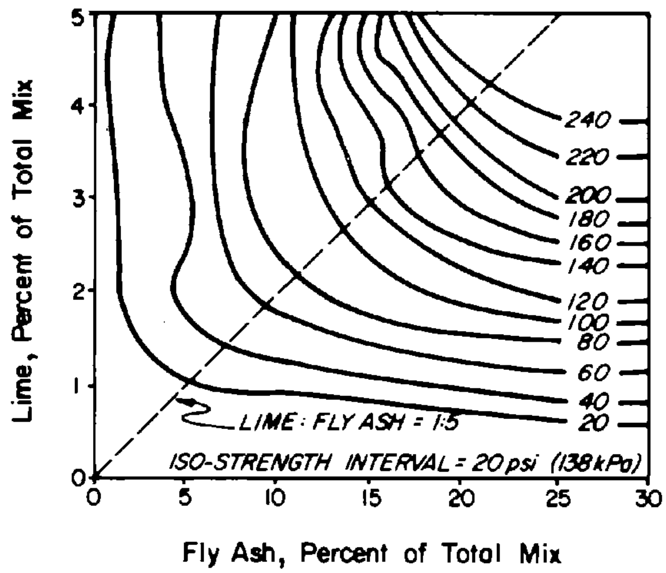

Figure 39. Compressive strength contours developed from grid approach to trial mix selection and testing for lime-fly ash-soil mixtures.(5)

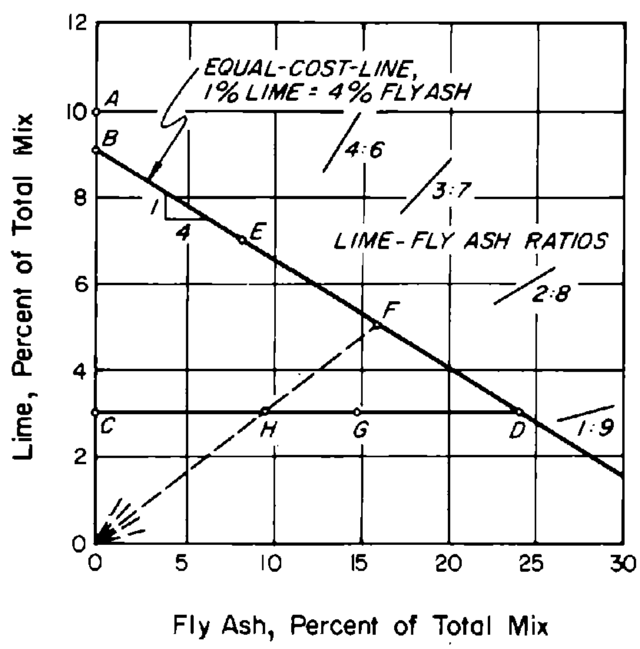

Figure 40. Example of Iowa equal-cost-line method of trial mix selection for lime-fly ash-soil mixtures.(5)

2. Moisture-Density Tests

Since it is desirable to perform all strength and durability testing at maximum dry density and optimum moisture content, moisture-density tests are usually performed on the trial mixes. In most cases, the moisture-density relationship is determined in accordance with AASHTO T 99-74.

If a large number of trial mixes are being tested, a short-cut procedure can be used to estimate the optimum moisture content for preparing strength testing samples. The method is based on interpolation between known optimum moisture contents.[5] In Figure 41, the optimum moisture contents for the 100 percent soil mix, 50:50 soil-lime mix, and 50:50 soil-fly ash mix are determined by normal laboratory methods. The respective optimum moisture contents are indicated at the appropriate corner of the equilateral triangle. Intermediate material percentages are scaled off on the outside of the triangle, and intermediate water contents are scaled off between each corner on the inside of the triangle. The water contents are then connected by

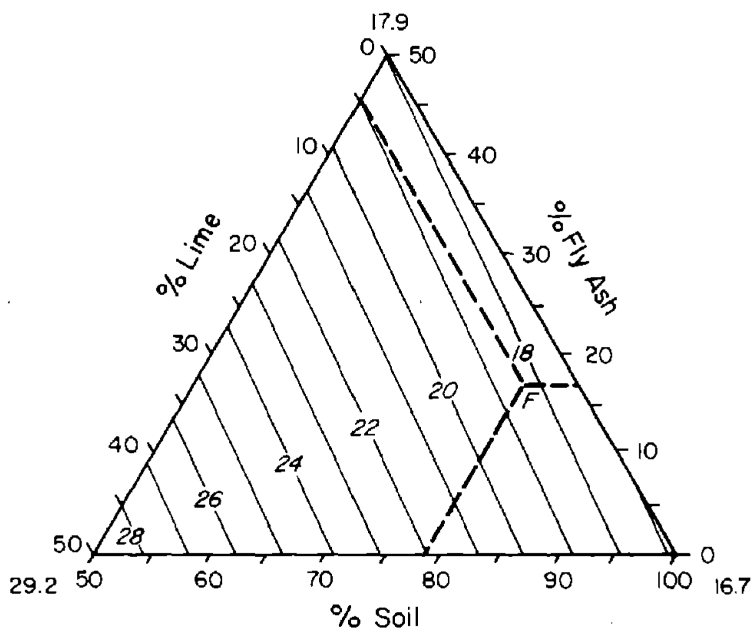

Figure 41. Short-cut method for estimating optimum moisture content of lime-fly ash-soil mixtures.[5]

lines. The estimated optimum moisture content for any combination of the three materials can then be read from the chart. A different plot is required for each soil, lime, or fly ash type. The trial mix chosen for design is subjected to the moisture-density test to more accurately determine the optimum moisture content for construction purposes and for molding specimens for durability testing.

3. Strength Tests

The most common strength test used for lime-fly ash soil mixtures is the unconfined compression test. Specimens are usually molded to cylindrical shapes 2 inches in diameter by 2 inches in height (51 mm x 51 mm), 2 inches in diameter by 4 inches in height (51 mm x 102 mm), 4 inches in diameter by 4.56 inches in height (102 mm x 116 mm) (standard Proctor mold size), or 6 inches in diameter by 8 inches in height (152 mm x 203 mm).[5] Where length to diameter ratios of less than 2:1 are used, strength correction factors can be applied, although this is not always done. After molding, specimens are wrapped in plastic film or sealed in containers to prevent carbonation of the lime and moisture loss. Specimens are stored in 100 percent relative humidity at some specified temperature, usually 70± 3° F (21±2° C), for a specified time, usually 28 days. It is common practice to soak 28-day specimens for the last 24 hours before testing to approximate the worst field conditions. Larger specimens require capping with plaster of paris or a sulphur compound prior to testing. Unconfined compression tests of lime-fly ash-soil specimens are performed in much the same manner as tests of concrete cylinders (AASHTO T 22-74).

4. Durability Tests

In areas where durability tests are necessary, durability specimens for freeze-thaw tests are molded and cured in the same manner as compression test specimens, except that Proctor-sized specimens are usually used. If AASHTO T 136-70, the freeze-thaw test for soil-cement specimens, is to be performed, it is suggested that the brushing of specimens be eliminated and a residual compressive strength criterion be used in place of a weight-loss criterion. Also, curing periods prior to testing should be extended to 14 or 28 days instead of the seven days used for soil cement. Compression testing after weathering cycles is performed as above. Wet dry tests (AASHTO T 135-70) seem to have little detrimental effect on lime-fly ash-soil specimens, therefore they are usually eliminated.[5] Details pertaining to the previously mentioned Iowa freeze-thaw test are contained in Reference 17. The vacuum saturation test method is detailed in Reference 8.

For durability tests based on cyclic freezing and thawing, the appropriate number of cycles is a function of climatic conditions and location of the lime-fly ash-soil mixture in the pavement cross-section (i.e., the thickness of cover).[5] In a temperate area, for instance, 12 cycles is the maximum number recommended for evaluation of a base course with 2 inches (51 mm) of surfacing, and four cycles for evaluation of a subbase. Calculation techniques for determining the appropriate number of cycles are contained in Reference 1.

D. Fly Ash-Soil Base and Subbase Courses

Numerous studies have indicated that some fly ashes are capable of stabilizing soils without the use of lime.[13,14,20] These fly ashes usually, but not always, contain high quantities of calcium oxide. Low carbon content, as measured by loss-on-ignition, is usually required also.

Table 17. Stabilization of soils with fly ash alone for base course application.[15]

SOIL TYPE	PERCENT FLY ASH REQUIRED TO PRODUCE 400 psi (2760 kPa) IN 28 DAYS UNDER NORMAL CURING CONDITIONS[1]	
	FLY ASH A	FLY ASH B
Crushed Solon Limestone	7	15
Crushed Rapid Limestone	32	37
Limestone Dust	22	22
Bottom Furnace Ash	29	38
Dune Sand	36	—[2]
Colfax Mix	37	29
Friable Loess	—[2]	—[2]
Gumbotil	—[2]	—[2]

[1] Percent by weight of the total dry mix.

[2] Could not adequately stabilize soil in normal quantities.

Tables 17 and 18 contain strength test results for various fly ash-soil mixtures. Table 19 contains pertinent physical and chemical information about the fly ashes in Tables 17 and 18. It can be said that, generally, some fly ashes can adequately stabilize non-plastic coarse-grained soils such as gravel, sand, and slag.(15) Fly ashes having calcium oxide contents of 20 percent or greater have been found to adequately stabilize fine-grained plastic soils such as clay, as well as coarse-grained soils. It should be noted, however, that some fly ashes with high calcium oxide contents can suffer strength reductions if there is a delay between mixing and compaction.(21) Certain retardants can be added to delay the hydration of the fly ash. Small percentages of salt have been found effective.

The suitability of a particular fly ash can be determined by subjecting various mixtures of the fly ash and the

Table 18. Effect of fly ash on strength and bearing characteristics of several soils.(13,21)

SOIL	FLY ASH	RATIO SOIL:FLY ASH	CURE days	STRENGTH	
				CBR %	COMPRESSIVE STRENGTH psi (kPa)
Missouri Clay	C	100:0	3	3.5	
		85:15	3	7	
		85:15	3	12.5	
		85:15	3		538 (3710)
Arkansas Clay	D	100:0	7		190 (1310)
		80:20	7		410 (2830)
		100:0		4	
		80:20		15	
Arkansas Sand	D	100:0	7		4 (30)
		80:20	7		730 (5030)
		100:0		22	
		90:10		104	

Table 19. Significant characteristics of some fly ashes successfully used alone as stabilizers for soils (refer to Tables 17 and 18). (13,15,21)

FLY ASH	SOURCE	POWER PLANT	HANDLING	CaO %	L.O.I. %	SPECIFIC SURFACE cm^2/gm
A	Missouri Coal	Montrose, Kansas City, Mo. Power & Light Co.	Mechanical Collector	11.6	2.8	1.730
B	Iowa Coal	Des Moines, Iowa Power & Light Co.	Mechanical Collector	5.8	0.2	1,460
C	N/Av	LaCygne, Kansas City, Mo., Power & Light Company	Scrubber, Limestone modified	High	N/Av	N/Av
D	Wyoming Coal	N/Av	Electro-static Precipitator	20.0	0.0	N/Av

soil to be stabilized to the same laboratory testing program outlined for lime-fly ash-soil mixtures. A rapid testing procedure has been developed, however, for evaluating the suitability of a fly ash.[15] Compacted specimens of the fly ash are wrapped in polyvinylidene chloride and sealed with cellophane tape. The specimens are preheated for two hours at 140° F (60° C), then autoclaved at 248° F (120° C) and one atm (100 kPa) pressure for 24 hours. Upon removal from the autoclave, specimens are soaked for two hours in distilled water, and tested in unconfined compression. Fly ashes with strengths of 600 psi (4140 kPa) or greater are considered to be very suitable for stabilizing soils for base and subbase applications.[15]

E. Thickness Design

No specific thickness design methods have been developed for lime-fly ash-soil mixtures utilizing fine-grained soils or for fly ash-soil mixtures. In the case of lime-fly ash-aggregate high quality base material, it is possible to reduce the thickness which would normally be required for unstabilized base course materials.[20] In lime-fly ash-soil or fly ash-soil mixtures, where the purpose of stabilization is to improve an otherwise substandard material to the point where it can be used as a base or subbase material, no thickness reduction is recommended.[20] Recommended minimum thicknesses for stabilized layers are shown in Table 20.

F. Soil Modification

The purpose in soil modification is to improve certain soil characteristics which enable the subgrade to provide adequate support for the pavement, or simply to expedite construction in areas where undesirable soil characteristics are making construction activities difficult. Lime-fly ash mixtures and fly ash alone have been used quite successfully in this application.

The addition of lime and fly ash to fine-grained soils is effective in absorbing excess moisture, reducing plasticity, modifying soil texture, and decreasing volumetric shrinkage.[18,19] The long-term pozzolanic action which occurs between the lime and fly ash, in conjunction with the improvement in soil characteristics, can substantially

Table 20. Recommended minimum thickness of lime-fly ash-soil and fly ash-soil mixtures.[20]

APPLICATION	RECOMMENDED MINIMUM THICKNESS inches (mm)
Base Course	6 (152)
Subbase Course	4 (102)
Subgrade Modification	4 (102)

improve the load-carrying capacity of the subgrade. This is often evident as a measurable increase in California Bearing Ratio (CBR).

The improvement in characteristics of various soils by the addition of lime-fly ash and fly ash alone is illustrated in Table 21.

There is no standard procedure to be followed nor specific criteria established for evaluating soil modification measures.[20] The goal in soil modification is an improvement in classification. An evaluation of how effective various combinations of stabilizers (lime and fly ash) are can be done simply by monitoring the improvement in the soil characteristics or properties of concern as the amount of stabilizer is varied. The cost of subgrade modification is offset by increased pavement serviceability; decreased pavement thickness, if the subgrade's support capabilities are measurably increased; or savings in construction time, where poor subgrade conditions are hindering the progress of work. A recommended minimum thickness of modified subgrade layers is shown in Table 20.

G. Construction Procedures

The construction procedures detailed in Section III for lime-fly ash-aggregate can be applied to lime-fly ash-soil and fly ash-soil mixtures with the following exceptions and qualifications.

Mixing

Since most soils to be stabilized or modified will be in-situ, mix-in-place techniques are generally more economical than central mixing. However, the quality of the resulting mixtures is usually less than that produced by central mix methods. Strict control should be exercised during the mix-in-place operations, with frequent checks on mixing efficiency. This can be done by trenching through the in-place material and inspecting the color of the mixture.[5] Unmixed streaks or layers indicate poor mixing, and the area should be remixed until uniformity of color is achieved.

Compaction

Fly ash-soil mixtures involving coarse-grained soils can be compacted as indicated in Section III. However, lime-fly ash-soil and fly ash-soil mixtures incorporating

Table 21. Effect of lime-fly ash and fly ash alone on properties of some soils.(9,13,21)

SOIL TYPE	FLY ASH	ADMIXTURE %	RATIO LIME:FLY ASH	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	γ_dMAX-OPT. M.C.		UNCONFINED COMPRESSIVE STRENGTH			CBR		PERMEABILITY COEFFICIENT cm/sec
							pcf (kg/m^3)	%	Days	psi	(kPa)	Days	%	
North Dakota Clay	-	0	0	92.4	25.7	66.7	104.6 (1676)	21.4	-	57	(390)	-	-	$\sim 10^{-10}$
	Ottertail (lignite)	24	1:5	42.0	38.4	3.6	102.7 (1645)	19.9	0 7	241 464	(1660) (3200)	-	-	3.8×10^{-7}
	Basin (lignite)	20	1:4	52.2	36.7	15.5	104.9 (1680)	20.4	0 7	231 396	(1590) (2730)	-	-	4.3×10^{-8}
North Dakota Silty Clay	-	0	0	34.2	21.6	12.6	112.5 (1802)	6.7	0	87	(600)	-	-	1×10^{-6}
	Ottertail (lignite)	20	1:4	29.4	27.1	2.3	114.0 (1826)	15.4	0 7	112 532	(770) (3670)	-	-	5.3×10^{-8}
	Basin (lignite)	20	1:4	30.0	26.4	3.6	113.6 (1820)	15.2	0 7	124 444	(850) (3060)	-	-	4.7×10^{-8}
Arkansas Ralinite Clay		0	0	-	-	19	97.5 (1562)	20	7	190	(1310)	-	4	-
	(western)	10	0:10	-	-	18	101.2 (1621)	18	7	305	(2100)	-	5	-
		20	0:20	-	-	17	102.6 (1643)	17.5	7	410	(2830)	-	15	-
Missouri Clay	-	0	0	65	22	45	98 (1750)	24	0	28	(190)	3	3.5	-
	Hawthorne	15	0:15	45	24	15	95 (1522)	26.5	0 5 28	21 56 204	(140) (390) (1410)	3 5 8	7 9 12.5	-

cohesive soils are best compacted with pneumatic-tired or sheepsfoot rollers. Vibratory rollers are better suited to granular mixtures.[5]

Compaction of some stabilized soils may result in the formation of horizontal shear planes, resulting in a thin, platey structure in the upper part of the compacted layer. These compaction planes should be removed by light scarification with a spike-toothed harrow, weeder, or nail drag, and the loosened material then moistened and recompacted with a pneumatic-tired roller.[5]

Surfacing

Lime-fly ash-soil and fly ash-soil base courses require a protective wearing surface. A double bituminous surface treatment usually 3/4 inches (19 mm) thick is common. In areas of higher traffic volumes, wearing surfaces should be increased in thickness. Plant-mix asphaltic concrete 1-1/2 to 3 inches (38-76 mm) in thickness is often used. In areas where the pavement is subjected to freeze-thaw cycles, thicker surfacings can reduce the number of free-thaw cycles to which a stabilized soil base or subbase will be subjected.[5]

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VI. FLY ASH EMBANKMENTS

A. Factors Affecting Utilization

The utilization of fly ash as borrow material in embankments has resulted from two primary factors: the light unit weight of fly ash and its relative economy as compared to natural borrow materials. The property of light unit weight is particularly advantageous in situations where filling operations are necessary on relatively weak subsoils. The use of natural materials, which have dry unit weights on the order of 100-130 pcf (1600-2080 kg/m^3)[9] could produce excessive settlement in a compressible subsoil or, conceivably, complete failure of the weak, underlying strata. The average range of compacted dry unit weights for fly ash, 70-95 pcf (1120-1520 kg/m^3),[15] represents a considerable reduction in the surcharge placed on in-situ materials. Other methods are available in these situations, such as overexcavation of the weak soils and backfilling with more suitable material, vibroflotation, sandpiles, surcharging, and other means of minimizing or accelerating settlement in the subsoil. These methods, however, are very expensive; thus, the use of the lightweight fly ash can represent a more economical solution to the problem, depending, of course, on the availability of fly ash and its proximity to the construction site.

In circumstances where there is simply a lack of suitable borrow materials near the construction site, which often occurs in urban and other highly developed areas, or where they may not be available locally due to certain environmental constraints, fly ash from a nearby power plant or disposal area can represent an economic source of borrow material.

One of the most significant characteristics of fly ash in its use as a roadway embankment material is its strength. Well-compacted fly ash has been shown to exhibit strengths comparable to those for soils normally used in earthfill operations. Laboratory tests indicate that most fly ashes have shear strength parameters which would make them stable and strong construction materials for highway embankments and other light load-bearing fills.[3,8,21,22,27,35] In addition, many fly ashes possess self-hardening properties, as discussed in Section II, which can result in the development of shear strengths in excess of those encountered in many soils.[21] The addition of lime or cement can induce hardening in fly ashes which may not self-harden alone. Significant increases in shear strength parameters can be realized in relatively short periods of time and, if taken into consideration in the design of the embankment, can represent a distinct economy over other embankment materials.

It has been shown in the laboratory, in trial embankments, and on actual construction projects that those factors associated with fly ash which could be undesirable in embankment construction can be easily controlled or eliminated through relatively simple methods. Foremost among these would be the apparent frost susceptibility of fly ash based on its grain size distribution. Methods ranging from restriction of fly ash to depths below that of normal frost penetration to lightly stabilizing the fly ash used within frost penetration depths have been successful in preventing potentially damaging frost action. The nuisance of dusting of dry fly ash on a construction site is easily controlled through regulation of the material's moisture content. Erosion of fly ash slopes is eliminated through use of a minimal soil cover and seeding. The danger of chemical leaching is minimized through proper compaction, adequate drainage, and proper surface treatment.

Fly ash has been proven to be a suitable material for embankment construction which can be as easily handled as a natural soil and which presents no problem in its utilization which cannot be solved through proper design and good construction technique.

B. Case Histories

The first documented use of fly ash in an engineered fill in Great Britain occurred in the late 1950's.[31] A series of trial embankments were constructed at various locations in an attempt to determine proper design parameters and construction procedures and to convince local engineers of the suitability of fly ash for this particular application. The trial embankments led to the acceptance of imported fly ash fill on a number of roadway projects, and, ultimately, the incorporation of fly ash into Great Britain's massive Motorway System construction program, which is similar to the U. S. Interstate System. The use of fly ash permitted the construction of miles of high-speed divided highways over areas of poor subsoil where the use of heavier fill material would have been impractical.

About the same time, the French became interested in this application of fly ash. Within a few years, they also, were using this material in embankments on a number of major roadway projects. Other European countries followed suit, and West Germany, Finland, Poland, and the Ukrainian SSR were among the countries reporting the utilization of a significant portion of their 1974 fly ash production in roadway fills and embankments.[36]

In the United States, however, only five states reported any activity in this area: Illinois, West Virginia, Michigan, New York, and North Dakota.[13] The total volume of U. S. fly ash utilized in roadway fills and embankments has been negligible. Limited knowledge of the unique properties of fly ash which make it comparable, and even superior, to conventional fill materials, as well as a lack of construction experience, have resulted in the U. S. lagging far behind the European countries in this large-tonnage application of fly ash.

A number of case histories point up the success achieved with fly ash embankments in isolated situations in this country and in what have become routine cases in Great Britain. A few are presented below.

F.A. Route 437, Section 8
Lake County, Illinois

As a result of a demonstration project in Waukegan, Illinois, in 1965, the Illinois Department of Transportation agreed to permit the use of fly ash as an alternative embankment material along a 1.5-mile (2.4 km) section of the Amstutz Expressway between Grand and Greenwood Avenues in Waukegan.[2] The low bid was received for the fly ash alternative, which represented a savings of $62,500 over the use of earth fill.

The average height of the embankment beneath the proposed four-lane highway and 42-foot (12.8 m) median strip was 3.5 feet (1.1 m). The embankments for adjacent entrance and exit ramps were a maximum of 20 feet (6.1 m) in height.

Construction began in 1972 and was completed in 1974. A total of 267,000 yd^3 (204,000 m^3) of fly ash were placed. Two feet (0.6 m) of soil cover was placed in the median area and 8 feet (2.4 m) on the side slopes to support vegetation, prevent erosion and provide frost protection. Photographs of the construction are shown in Figures 42 through 45.

During the summer of 1974, frost depth gages, thermocouples, settlement plates, and observation wells were installed to monitor the behavior of the embankment during the winter months.[24] Readings and measurements taken

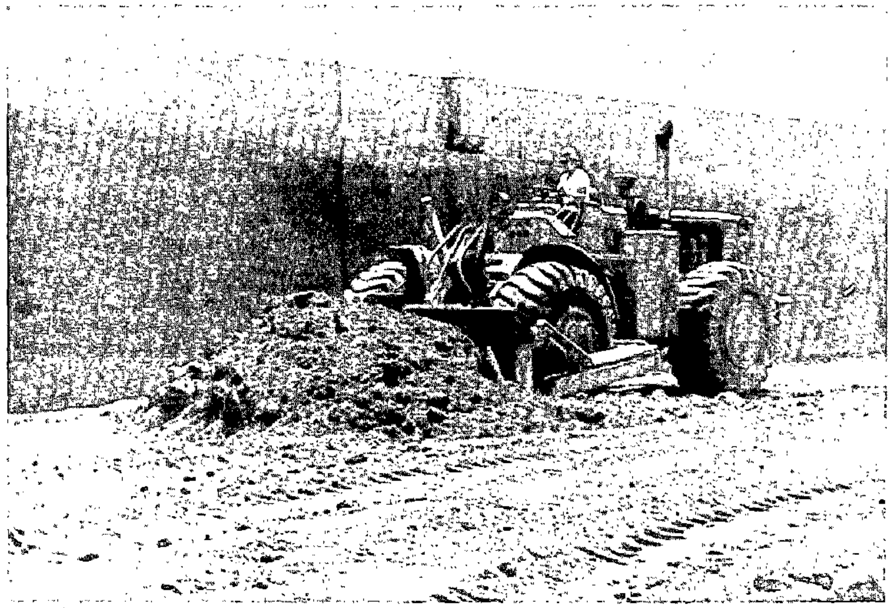

Figure 42. Spreading fly ash for roadway embankment construction in Waukegan, Illinois.

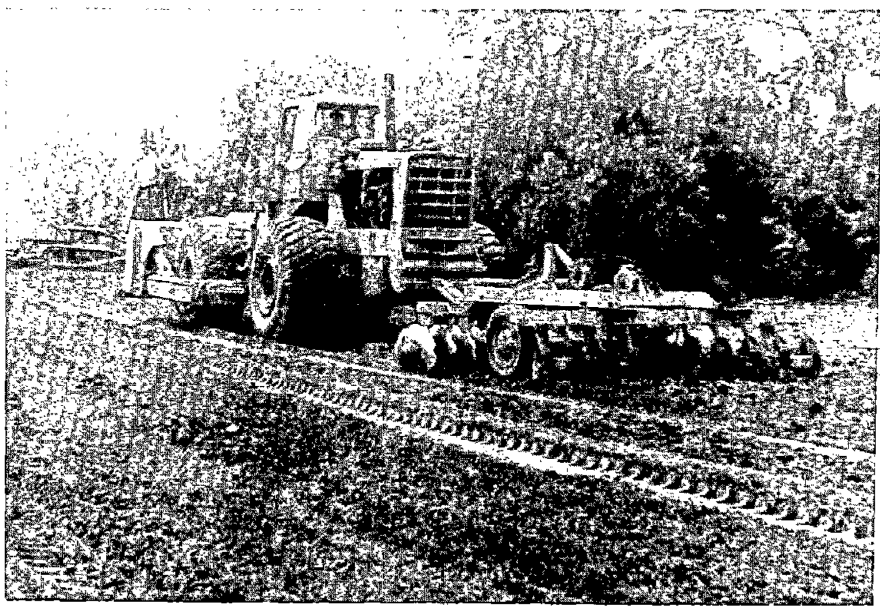

Figure 43. Discing stockpiled fly ash to break up lumps prior to compaction on F.A. Route 437.

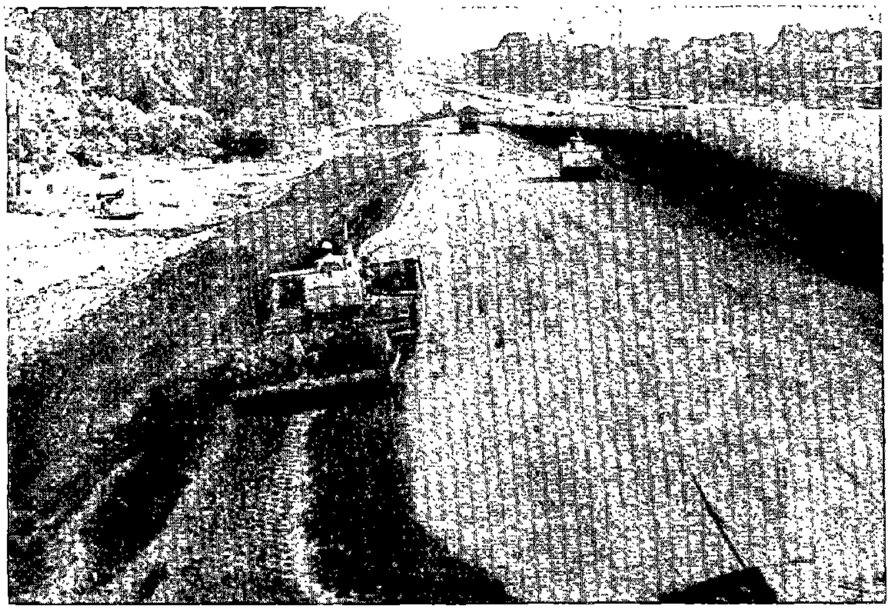

Figure 44. Compaction of fly ash embankment with vibratory roller and soil berm with sheepsfoot roller.

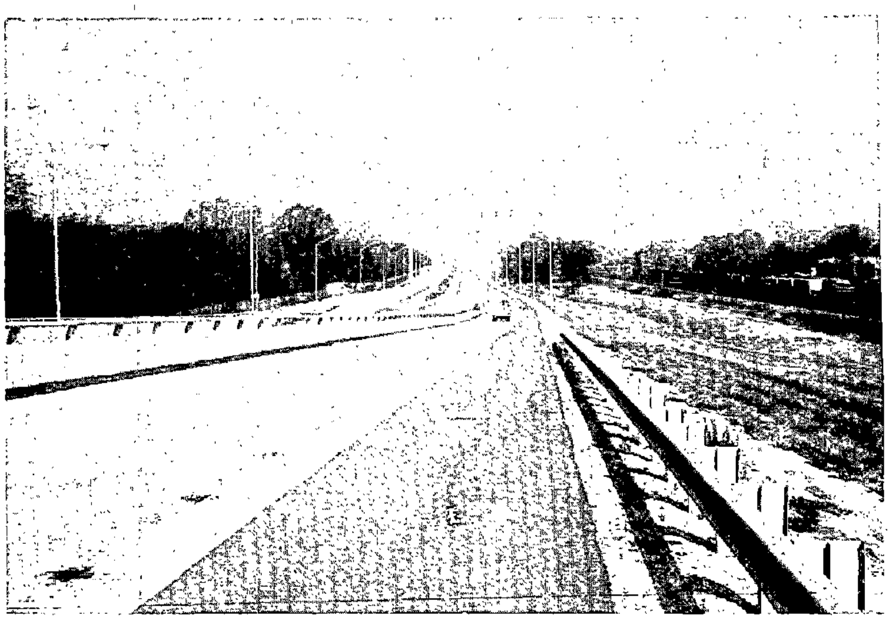

Figure 45. Completed fly ash embankment, Waukegan Illinois.

throughout the winter of 1974-75 indicated that the fly ash embankment was little affected by frost during the relatively mild winter. A visual inspection of the area in late 1975 revealed no pavement distress as might result from settlement of the underlying embankment material or frost heave.(5) Monitoring of the embankment will continue in the future.

Motorway M.5
Bristol and Somerset, England

A 2-mile (3.2 km) section of the M.5 between Bristol and Avonmouth was to be constructed on a highly compressible layer of alluvium as much as 40 feet (12.2 m) in thickness.(14) An embankment 7 feet (2.1 m) in height was required along the trunk road, with side road and interchange embankments 20 feet (6.1 m) in height. Fly ash was selected as the embankment material because of its relatively light unit weight in comparison to locally available borrow materials.

On another section of the M.5 between Clevedon Hills and Brent Knoll in Somerset, a similar situation existed where the proposed motorway was to cross an alluvium layer which was 90 feet (27.4 m) thick in areas.(7) This posed serious potential settlement problems at 14 bridges and two interchanges. Here again, fly ash was selected as a lightweight fill. When sufficient quantities of fly ash could not be obtained from a nearby power station, the fly ash was transported by rail from a power station 80 miles (129 km) away. A total of 1,025,000 tons (930,000 Mg) was brought to the site in this manner.

Alexandria By-pass
Dumbarton, Scotland

Stage I construction of the Alexandria By-pass included a bridge over the River Leven.(1) Because clearance had to be provided for navigational purposes, very high approach embankments were required. Poor subsoil, a saturated silt, necessitated the use of a lightweight fill material. Approximately 590,000 yd^3 (450,000 m^3) of fly ash were placed and compacted in the embankments. Grass was hydroseeded directly onto the side slopes of fly ash to provide a vegetative cover and protect against erosion.

Stage II construction included the extension of the Stage I embankment to a second bridge. This length of embankment spanned the same weak subsoils. Commencing in 1973, an additional 78,000 yd^3 (60,000 m^3) of fly ash were used in the embankment, which reached a maximum height of 39 feet (12 m). It was possible to place 1300 tons (1200 Mg) of fly ash daily. Two years after construction, the embankment had settled a total of only 10 inches (254 mm), which was considered to be quite satisfactory.

Clophill By-pass, A.6
Bedfordshire, England

Construction was begun in April of 1975 on an 8-foot (2.4 m) high roadway embankment along the A.6 Motorway.(16) The route of the motorway was underlain by a layer of highly compressible peat 16 feet (5 m) in thickness. In addition, the ground-water table was essentially at the level of the existing ground surface, making it almost impossible to operate construction equipment on the soggy surface. It was decided to use fly ash in the embankment in order to minimize settlement in the underlying peat layer. Hopper ash was brought to the site from three different power stations and placed at a rate of 400-800 yd^3 (300-600 m^3) per day. Approximately 20,000 yd^3 (15,000 m^3) of compacted fly ash were used to provide a stable embankment.

Side slopes of 2 horizontal to 1 vertical were constructed. Although these were to be eventually covered with a 4-inch (102 mm) layer of topsoil and seeded, no erosion of the bare slopes was visible during a site visit in October of 1975 even though the previous month had been very rainy. Total settlement as of October was 6 inches (152 mm), which was less than predicted. The embankment is shown in Figures 46 and 47.

U. S. Route 250
Fairmont, West Virginia

A slide caused by poor drainage occurred along U. S. Route 250 near Fairmont, West Virginia.(25) Engineers of the West Virginia Department of Highways decided to remove the slide mass, install subsurface drainage, and replace the slide material with compacted fly ash. Approximately 5,000 tons (4500 Mg) of fly ash were utilized in the embankment which had an average height of 25 feet (7.6 m) and side slopes of one and one-half horizontal to one vertical. An average of 97 percent of standard Proctor maximum dry density was obtained. The surface of the embankment was sealed with a coat of hand-sprayed road tar (RT-12).

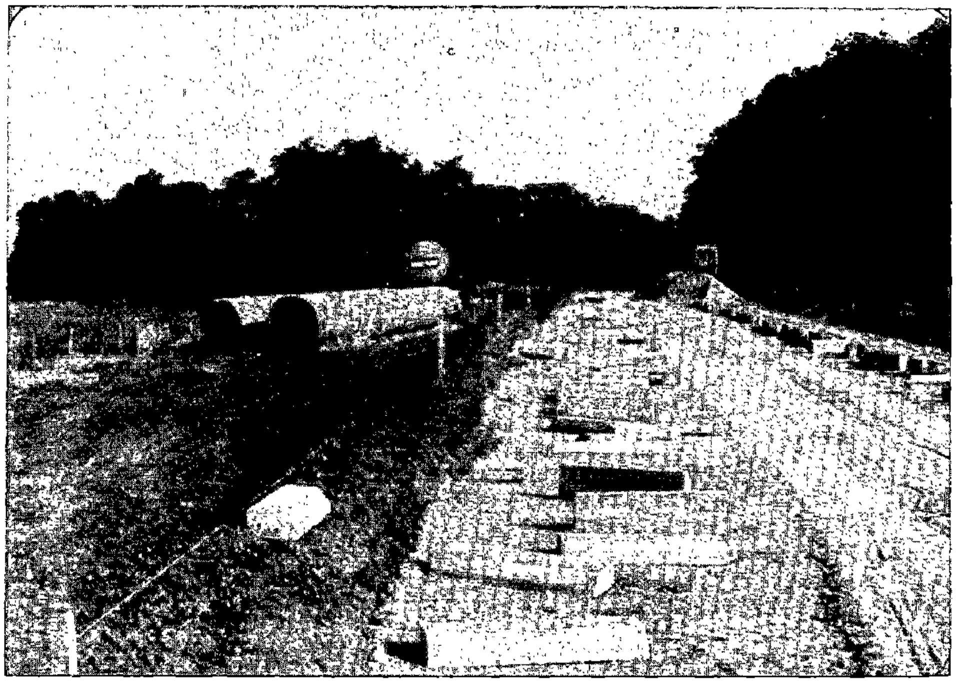

Figure 46. Completed fly ash embankment with hoggin subbase in place (provisions were made in width of embankment for additional lanes).

Figure 47. Sideslopes of fly ash embankment at Clophill By-pass prior to hydroseeding.

C. Design Considerations

A number of characteristics associated with fly ash affect design considerations and determine the suitability of any fly ash for embankment application. In the first category are those engineering parameters of concern in all fill materials, namely strength and compressibility. The second category is comprised of characteristics common to a smaller number of materials: frost susceptibility, capillary action, erodibility, and leaching. In some cases, specific tests can be performed to determine the magnitude of a particular parameter, which can then be used in accepted design methods for embankments. In other cases, design procedures are recommended to take into account certain unique characteristics of fly ash.

Shear Strength

For embankment design, it is necessary to determine the effective shear strength of the fly ash, usually by means of the undrained triaxial shear test performed in accordance with AASHTO T 234-74. As a result of the thin-layer placement process and the permeability of most fly ashes, the pore pressures in the fly ash are essentially dissipated prior to the placement of the next fly ash layer. Specimens are usually prepared at maximum dry density and optimum moisture content as determined by AASHTO T 99-74.

It may be noted that the cohesive strength, c_u, for fly ashes which possess no self-hardening properties differs from that for fly ashes which do self-harden. In the former case, there exists an apparent cohesion due to capillary forces produced by pore water.(11) This apparent cohesion can be destroyed by complete drying or saturation. The most significant portion of the shear strength of these fly ashes is derived from the angle of internal friction. Fly ashes which do self-harden exhibit real cohesion resulting from the cementation action which occurs between the fly ash particles. The cohesion of these ashes, in fact, increases with age.(9) Values of cohesion and angle of internal friction for a number of American and British fly ashes are listed in Table 22.

The phenomenon of age-hardening is apparent in many of the fly ashes listed in Table 22. The rate at which a fly ash increases in strength is a function of the individual ash and the ambient conditions, primarily temperature. Age-hardening has been observed quite frequently in the field as evidenced by the results of numerous California Bearing Ratio tests, several of which are presented in Table 23. In the situation where large loads will not be applied to

Table 22. Shear strength parameters of several British and American fly ashes.(17,18,28,32,33)

SOURCE	AGE AT TESTING, days									
	0		7		14		28		56	
	Cu psi (kPa)	ϕ deg	Cu psi (kPa)	ϕ deg	Cu psi (kPa)	ϕ deg	Cu psi (kPa)	ϕ deg	Cu psi (kPa)	ϕ deg
Agecroft, U.K.	6.5 (45)	33.6	-	-	30.0 (207)	34.6	-	-	30.0 (207)	37.0
Battersea, U.K.	7.0 (48)	38.0	-	-	12.0 (83)	36.5	-	-	16.0 (110)	36.5
Bold, U.K.	5.0 (34)	34.6	-	-	34.0 (234)	38.6	-	-	40.0 (280)	38.8
Dunston, U.K.	6.0 (41)	33.6	-	-	8.0 (55)	34.2	-	-	10.0 (690)	34.7
Skelton Grange, U.K.	4.0 (28)	34.3	-	-	16.0 (110)	40.0	-	-	26.0 (179)	40.0
Westwood, U.K.	5.0 (34)	31.8	-	-	7.0 (48)	31.8	-	-	12.0 (83)	36.5
Brighton, U.K.	-	-	4 (28)	25	4 (28)	28	-	-	-	-
Brimsdown, U.K.	5 (34)	25.5	6 (41)	29	-	-	-	-	-	-
Brunswick Wharf, U.K.	6 (41)	26	12 (83)	26	-	-	-	-	-	-
Cliff Quay, U.K.	4 (28)	26	14 (97)	28	-	-	20 (138)	32	-	-
Croydon, U.K.	5 (34)	26.5	9 (62)	30	-	-	-	-	-	-
Goldington, U.K.	-	-	-	-	40 (276)	42	56 (386)	41	-	-
Hackney, U.K.	10 (69)	29	25 (172)	23	-	-	-	-	-	-
Little Barford 'A', U.K.	4.5 (31)	26	6 (41)	27	-	-	17 (117)	32	-	-
Little Barford 'B', U.K.	-	-	-	-	16 (110)	27	16 (110)	30	-	-
Northfleet, U.K.	-	-	7.5 (52)	24	10 (69)	26	-	-	-	-
Rye House, U.K.	8 (55)	27	7 (48)	30	-	-	5 (34)	30	-	-
Tilbury, U.K.	16 (110)	48	25 (172)	45	60 (414)	41	70 (483)	38	-	-
West Thurrock, U.K.	-	-	12 (83)	31	11 (76)	29	-	-	-	-
Barony, U.K.	11 (76)	38	29 (200)	41	-	-	32 (221)	42	-	-
Braehead, U.K.	9 (62)	34	29 (200)	39	-	-	32 (221)	41	-	-
Kincardine, U.K.	14 (97)	33.5	14 (97)	34	-	-	12 (83)	35.5	-	-
Portobello, U.K.	13 (90)	35	17 (117)	41	-	-	20 (138)	43	-	-
Marysville, Mich.	13 (90)	43	-	-	-	-	14 (97)	43	-	-
St. Clair, Mich.	10 (69)	40	-	-	-	-	7 (48)	40	-	-
Trenton Channel, Mich.	10 (69)	38	-	-	-	-	15 (103)	38	-	-
Grand Avenue, Mo.	5.2 (36)	29	89 (614)	45	-	-	170 (1172)	45	-	-

the embankment or fill until sometime after construction is complete, one month for example, then 28-day values of shear strength can be used in design. For ashes in which self-hardening occurs, the 28-day strength should represent a large enough increase over the strength at zero days that some economy can be realized in foundation design. Thus, the strength gain with time can be taken into consideration in the design of the embankment provided that loading does not occur immediately after construction and that saturation of the embankment does not occur early in the life of the embankment (the self-hardening mechanism is inhibited by saturated conditions).[9]

It has been shown that shear strength is affected by sample density and moisture content. The undrained shear strength decreases significantly in fly ash samples compacted on the wet side of optimum.[19] In addition, shear strength decreases in samples compacted to less than maximum dry density.[33] This is an important consideration in design since it is not often practical to require 100 percent compaction in the field. Normally, a minimum of 90 percent compaction is specified. For design purposes then, a reduction in the laboratory values of apparent cohesion and angle of internal friction is recommended in accordance with the values shown in Table 24.

As a result of the shear strength of fly ash, most fly ash embankments can be constructed with side slopes of 2 or 3 horizontal to 1 vertical. Each case, however, should be determined through a standard stability analysis.

As previously mentioned, most fly ashes which have been tested have possessed shear strengths which make them suitable for light load-bearing fill. Obviously, many fly ashes with self-hardening properties are superior fill materials which can be used in heavier load-bearing applications, e.g., structural fills. If a fly ash is encountered, however, which possesses no self-hardening properties and has inadequate shear strengths for the application being considered, it is possible to mobilize the inherent pozzolanic property of the fly ash by the addition of small amounts of cement or lime. This form of stabilization increases the initial shear strength of the fly ash and, in addition, provides some long-term strength gain, much in the same manner as self-hardening. Cement stabilization results in a more rapid strength gain, but in certain geographic areas lime may be more economical. The amount of cement or lime required is determined by shear strength tests on trial mixes which have been cured under conditions (temperature and time) approximating field conditions.

Table 23. Increase in CBR values with time for some fly ashes with self-hardening characteristics.(26,28,30)

FLY ASH SOURCE	CBR, %		
	AGE AT TESTING, weeks		
	0	4	39
Agecroft	10-45	50-80	
Bold	50	150	
Willington Tip	20	80	
Portobello	16		38

Table 24. Reduction in shear strength parameters with densities less than maximum dry density.(9,33)

PERCENTAGE OF MAXIMUM DRY DENSITY	PERCENTAGE OF VALUE AT MAXIMUM DRY DENSITY		
	SHEAR STRENGTH	c_u	ϕ
85	60	70	80
90	75		
95	90-95		

Compressibility

Low compressibility is a desirable quality in a roadway embankment material from the standpoint of preserving a smooth roadway profile and minimizing differential settlement between highway structures, such as a bridge and its adjacent approach slabs. In all cases reported to date, settlements within fly ash embankments have been within acceptable limits and, in many cases, nearly negligible. Both fly ashes with and without self-hardening properties have given very satisfactory performance.[22,27,28]

The compressibility of fly ashes with no self-hardening properties is similar to that for a typical cohesive soil.[11] Consolidation occurs at a more rapid rate than in clays, however, since the permeability of fly ash is greater than clay. The overall compressibility in these cases is a function of the initial density of the fly ash. Figure 48 presents the consolidation curves for freshly compacted samples of several western Pennsylvania fly ashes having no self-hardening properties. The samples were compacted at varying densities. It can be seen from Figure 48 that loosely compacted samples were more highly compressible than the more densely compacted samples. Although the actual shape of the consolidation curves is determined by other factors as well, initial density is the predominant factor in the compressibility of fly ashes which exhibit no self-hardening.

For fly ashes which do self-harden, as well as stabilized fly ashes, the time-dependent component of age-hardening can reduce both the magnitude of compressibility and the rate of consolidation.[21] An extensive laboratory study on lime-stabilized fly ash produced several results which are of value in the design of roadway embankments utilizing self-hardening or stabilized fly ashes.[21] Results were based on samples which were allowed to cure 10 days:

1. Rate of settlement in the fly ash layer tends to be independent of the thickness of the layer;

2. The magnitude of the settlement is less than that which would occur in ordinary soils, and is a function of the hardening characteristics of the material and the age at which the load is applied;

3. Long-term loading or curing prior to loading results in lower compressibilities than short-term loading.

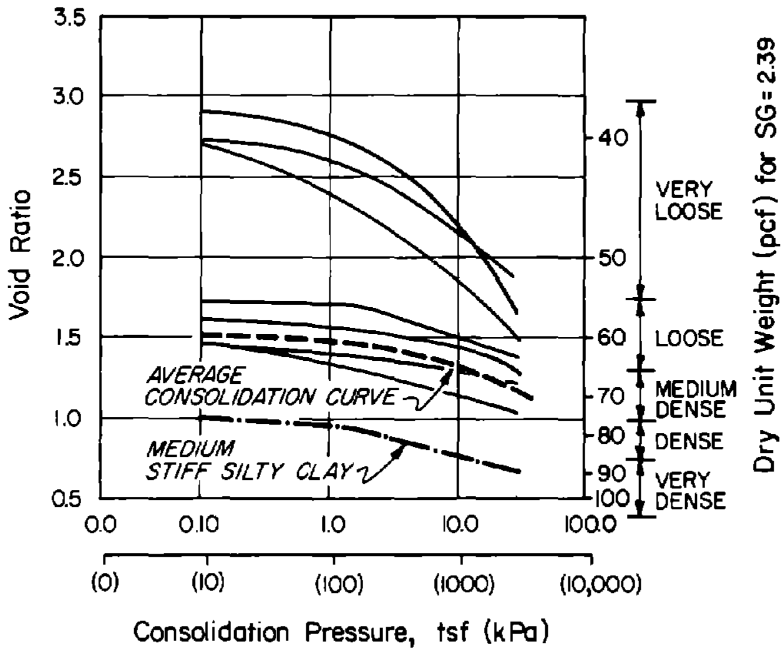

Figure 48. Consolidation curves for some western Pennsylvania fly ashes with no self-hardening characteristics.(11)

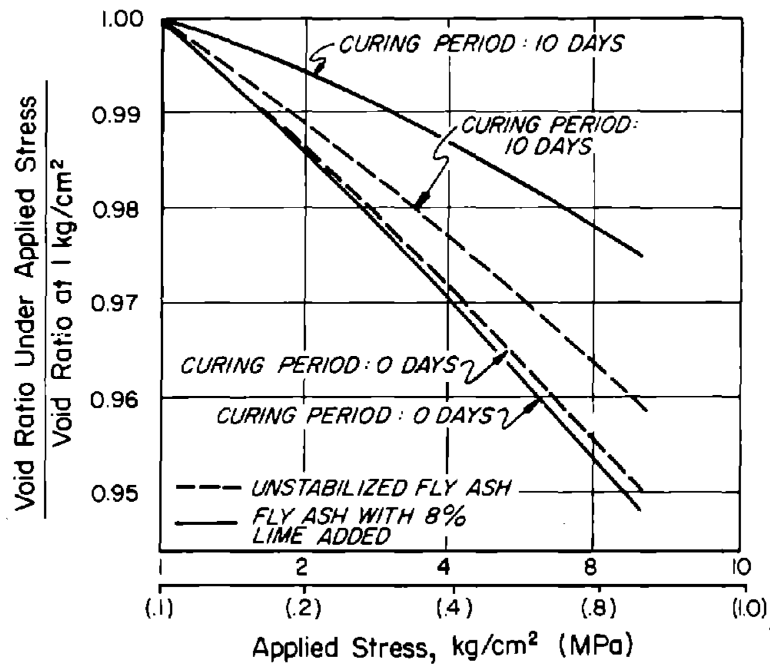

Figure 49. Compressibility characteristics of a stabilized and unstabilized Michigan fly ash.(21)

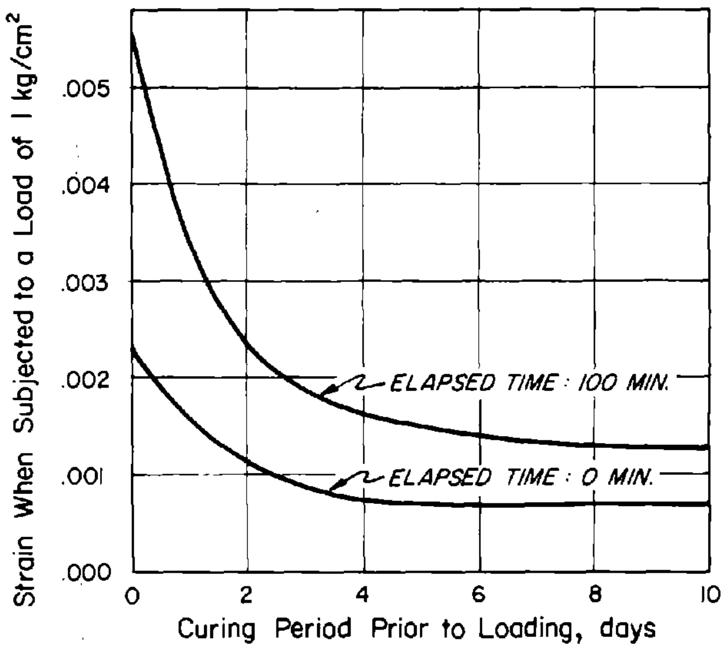

Figure 50. Effect of curing time on the deformation of a stabilized fly ash.(21)

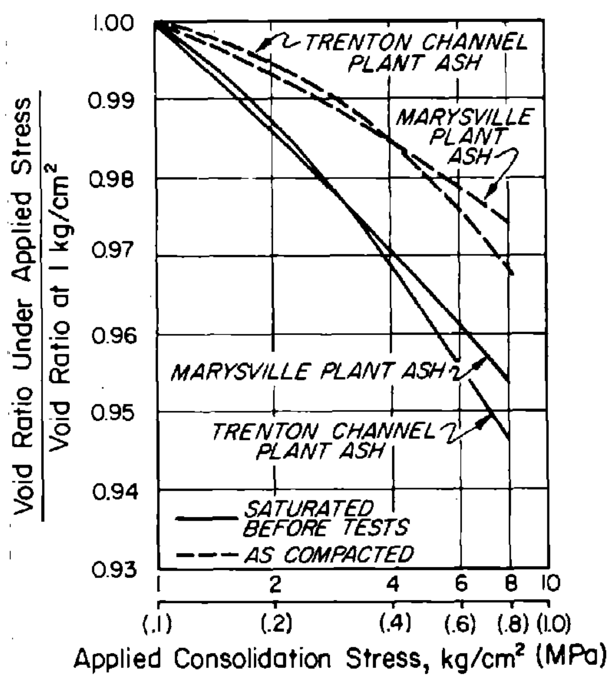

Figure 51. Compressibility behavior of saturated and partially saturated, compacted fly ash.(17)

Figure 49 illustrates the difference in compressibility of a Michigan fly ash, stabilized and unstabilized, when subjected to a load. The fly ash in question has no inherent self-hardening capabilities. Figure 50 illustrates the effect of curing on the deformation of a stabilized or self-hardening fly ash.

The study also indicates that partially saturated samples of fly ash, regardless of possession or lack of self-hardening properties, tend to be less compressible than fully saturated samples. This is illustrated in Figure 51. Thus, if a fly ash embankment is designed such that the fill remains in a partially saturated state, the compressibility characteristics are improved.

Based on the preceding conclusions, it is important that any laboratory testing program developed to provide engineering parameters for stability and settlement analyses attempt to correlate sample age, saturation conditions, and loading conditions with conditions anticipated in the field.

Regarding settlement analysis, it should be noted that the British have found difficulty in correlating predicted settlements and actual field settlements. It appears that the application of consolidation theory to oedometer test results yields exaggerated settlements, whereas settlements measured in dynamic tests are slightly on the conservative side.[28] A technique for developing predictions of long-term settlements in fly ash which age-hardens has been developed and is discussed in Reference 21, but is awaiting field verification.

Frost Susceptibility

The grain-size distribution of most fly ashes places them within the realm of frost susceptible soils. However, frost susceptibility is also influenced by pore size, mineralogical composition, strength and permeability.[21] Frost susceptibility criteria has not been developed in the United States for fly ash as embankment material; such criteria has been developed, however, at the Road Research Laboratory in England and are based on the amount of frost heave experienced by a compacted specimen when subjected to freezing conditions which simulate field conditions.[10] The following criteria was adopted for 6-inch (152 mm) high samples subjected to the 250-hour test:

1. Satisfactory materials heave 0.5 inches (13 mm) or less.

2. Marginally frost susceptible materials heave between 0.5 inches to 0.7 inches (13-18 mm).

3. Frost-susceptible materials heave 0.7 inches (18 mm) or more.

Details of the test can be found in Reference 10.

Frost heave tests performed on numerous fly ashes have shown that grain size distribution is not a reliable indication of frost susceptibility. A number of fine-grained fly ashes have been found to perform quite satisfactorily from the standpoint of frost-heave characteristics.[10]

For fly ashes which are found to be frost-susceptible, it is possible to increase their frost resistance through stabilization with cement or lime.[34] The amount of stabilizer required is a function of the pozzolanic properties of the fly ash and its permeability. The addition of cement or lime not only increases the tensile strength of the fly ash, thereby enhancing its ability to resist the heave pressures produced by the formation of ice lenses, but also reduces the permeability of the fly ash, thereby restricting the inflow of water and thus reducing the quantity of water available for frost formation.

Unfortunately, it is not possible to correlate satisfactory frost resistance with a single value of strength or permeability which would be applicable to all fly ashes. Testing is required for each individual fly ash if the fly ash is to be used in regions where it is exposed to freezing temperatures, i.e. above the normal frost penetration depths experienced in various parts of the country. The amount of cement or lime required to adequately stabilize a number of fly ashes in separate studies varied from 5 percent to 15 percent.[21,34] If stabilization of the fly ash is uneconomical, the fly ash should be restricted to use below the frost depth, with construction of a layer of non-frost-susceptible material above this level. The frost depth is a function, of course, of geographic location.

Capillary Action

Capillary action occurs in compacted fine fly ash, but to a lesser degree in fly ash where coarser particles are present, i.e., lagoon ash. This capillary rise can result in saturation and instability of fly ash less than 2 feet (0.6 m) in thickness.[9] As a result, it is recommended that unstabilized fly ash not generally be used in a total compacted thickness of less than 18 inches (46 mm).[30] In areas where the bottom of the fly ash is in contact with ground water, it is recommended that a protective filter of free-draining material be placed on the bottom of the embankment to a height of at least 18 inches (46 mm) above the ground water table to prevent capillary rise into the fly ash. The filter should be designed so as to prevent washing of the relatively fine-grained fly ash through the granular filter.

Erodibility

Unprotected, compacted slopes of fly ash are erodible when subjected to surface runoff or high winds. The erosion of the fly ash embankment shown in Figure 52 resulted from improperly controlled surface runoff. It is recommended, therefore, that permanent measures be taken to protect those surfaces which are not covered by pavement or the like. A thin layer of topsoil, followed by fertilization and seeding, provides a very satisfactory as well as aesthetically acceptable surface treatment. A minimum thickness of 6 inches (152 mm) of topsoil is recommended. Dense cover should be established within two years. If more rapid coverage is required, it is recommended that the topsoil thickness be increased.(4) The Scottish have had much success in hydroseeding bare slopes of fly ash, having low boron content, with a mixture of seed and fertilizer in a bituminous emulsion.(1)

If vegetative cover is not desired, or only provisions for short-term surface protection are necessary, a spray made from a bituminous emulsion can be applied directly to the fly ash immediately after compaction. The spray can be made by diluting a 50 percent bituminous emulsion at a ratio

Figure 52. Erosion of unprotected fly ash embankment.

of one part emulsion to four parts wetting agent and spraying at a rate of 2.5 gallons/100 ft^2 (0.001 m^3/m^2).[9] Stabilization of the surface with a few percent cement or covering with a blanket of coarse ash also provides short-term protection against erosion.

Leaching

While fly ash is basically an inert material when exposed to atmospheric conditions, it does contain certain compounds which are soluble in water. Numerous laboratory studies have determined that the percentage of fly ash soluble in water varies from less than 1 percent to 7 percent by weight.[3,6,17,29] The principal dissolved ions appear to be calcium and sulphate, with small quantities of magnesium, sodium, potassium, and silicate ions usually present. The leachate is generally alkaline, with a pH ranging from 6.2 to 11.5.[29] Leaching of trace metals has been investigated and is discussed in Reference 37.

Leaching of fly ash embankments can be produced by the following:

1. Surface runoff waters which flow across the top and sides of the embankment; and

2. Infiltration waters which pass through the fly ash mass.[23]

During runoff, chemicals can be leached from the surface layer of fly ash and the finer particles of fly ash eroded from the surface and carried off in suspension. The velocity of water percolating through the fly ash embankment is much less than that of runoff waters, thus retention time is greater and the opportunity for dissolving the soluble components of fly ash is greater.

Since the primary concern with leaching is the potential for pollution of ground and surface waters in the vicinity of fly ash embankments, the problem can be eliminated or minimized by controlling the amount of water which actually infiltrates or runs off a fly ash embankment.

Control of run-off waters, or surface drainage, is a relatively simple matter. All surface drainage from peripheral areas should be diverted around an embankment to minimize the volume of water actually passing over the surface of the fly ash. In addition, any of the properly constructed methods of erosion control prevent the washing away of fly ash particles. Runoff from pavement surfaces should be collected and discharged into a piped drainage system. Fly ash slopes of great length should be regularly benched in such a manner as to drain the runoff to the ends of the embankment,[11] rather than allowing the full volume of runoff to travel down the face of the embankment to the toe.

Infiltration can be controlled in three ways: restriction of the quantity of surface water coming in contact with the embankment, draining of springs and seeps away from the embankment, and control of the permeability of the fly ash.

The impermeable pavement surfaces along the top surface of the fly ash embankment prevent water in these areas from infiltrating the embankment, assuming, of course, that an adequate runoff collection system is provided. Topsoil and any additional soil cover which has been provided for frost protection aid in retaining a portion of the water which may infiltrate the unpaved surfaces of the embankment. Vegetative cover contributes greatly to moisture retention and lessened infiltration of water to the underlying layer. In the case of seeps and springs which could introduce water into the bottom or sides of the embankment, drains or a drainage blanket of properly sized and graded granular material should be provided to carry the water away from the embankment.

The volume of water which actually infiltrates the fly ash mass is a function of the fly ash's permeability. The permeability of fly ash compacted to maximum dry density in accordance with AASHTO T 99-74 has been shown to range from 5×10^{-7} cm/sec to 4×10^{-4} cm/sec, which represents very low to low permeability.[21] The permeability is, in turn, a function of grain size and compaction.

The field experience to date with fly ash embankments has indicated that very little water tends to percolate through the complete embankment. Fly ash embankments constructed on the flood plains of Great Britain and exposed to large amounts of precipitation have shown no evidence of pollution to the ground or surface waters.[6] This has been due, in part, to the low permeabilities of the in-place fly ash. In addition, where natural age-hardening proceeds, the soluble constituents of the fly ash are involved in the chemical process, thereby continuously reducing the quantity of materials which could be leached.[19]

A compacted embankment, properly drained and provided with adequate surface treatment, should represent no danger, either in the short-term or long-term, to the surrounding ground and surface waters.

D. Construction Procedures

Recommended construction procedures have been developed as a result of experience gained with trial embankments and actual construction projects in this country and abroad. General guidelines are presented herein; however, it is expected that field experience on each individual job will dictate final construction procedures.

Subgrade Preparation

Topsoil and soft material should be removed from the embankment area prior to embankment construction. Any springs or seeps in the area should be drained away from the embankments by the use of herringbone drains or a blanket of granular material. If ground water is expected to come in contact with the bottom of the embankment, a granular filter blanket of properly sized and graded material should be provided to a height of at least 18 inches (46 mm) above the ground-water table and should be constructed of material of the proper size and gradation to prevent the fly ash from washing through the filter.

It is impossible to compact fly ash which is dumped or spread into standing water. Therefore, any ponds, etc., should be either well-drained or filled with granular material as above prior to spreading the fly ash.

Fly Ash Delivery

Fly ash is normally delivered to the site in dump trucks which are covered to prevent moisture loss and subsequent dusting of the fly ash. Fly ash has been delivered by rail when large quantities are required and not available within a practical trucking distance.

Fly ash from hoppers or silos is conditioned with water at the power plant. The amount of water added, usually 20 to 40 percent, depends upon the compaction criteria established for the job and the type of fly ash. Stockpiled and lagoon fly ashes generally contain sufficient quantities of water and are thus not further conditioned at the plant.

Stockpiling

It is normally necessary to stockpile the fly ash for a short time on-site after delivery and prior to spreading. If any fly ash remains in the stockpile at the end of the day's work, its surface should either be thoroughly moistened or covered with a thin blanket of heavier material, such as bottom ash, to prevent drying and dusting of the fly ash.

Spreading

The fly ash is spread by means of a dozer into loose lifts of 9-inch (229 mm) thickness. On small jobs, a motor patrol can be outfitted with a blade and used in place of a dozer.(30) If the fly ash contains any lumps, as might be the case with stockpiled ash, it may be necessary to till the loose layer with a rotary tiller or similar equipment to break down the lumps.(2)

Compaction Criteria

The desired compaction on any project is usually specified as a percent of the maximum dry density of the fly ash as determined by AASHTO T 99-74 although AASHTO T 180-74 criteria has been used successfully on some projects. Compaction of less than 90 percent of maximum dry density is not recommended.

If the fly ash to be used is hopper or silo ash, the conditioned water content of the ash as delivered from the plant should be specified within a range of ±4 percent of optimum moisture content as determined from moisture-density tests on samples of the fly ash.(30) During dry or inclement weather, it may be necessary to adjust the amount of water being added at the power plant in order to compensate for moisture lost due to evaporation or moisture absorbed due to wet weather. Some means of adding water should be available on-site in the event that the moisture content of the fly ash falls below that necessary to produce the specified compaction.

The lagoon ash should be close to optimum moisture content upon excavation from the pond. Because lagoon ash has a very flat moisture-density curve, it is not meaningful to specify a moisture content other than to specify that it be such that the required density can be obtained.(8) It is important to perform moisture-density tests on samples collected from the same area of the lagoon from which the fly ash is to be excavated for construction.

Stockpiled fly ash can vary greatly in moisture content depending upon its location in the stockpile. Therefore, some adjustment of moisture content may be necessary prior to compaction. As with lagoon ash, care is required in sampling to determine the mositure-density relationship for the stockpiled fly ash.

If fly ash from more than one source is to be used on the project at the same time, it may be necessary to use a mean maximum dry density determined by taking the maximum dry density of each ash in proportion to the anticipated fraction of the total fly ash coming from each source. This mean maximum dry density is then used for determining the required compaction.[12]

Under certain circumstances, it may be more meaningful to develop a performance specification for compaction by means of a test section prior to full scale construction. The number of passes of a particular piece of compaction equipment required to achieve the desired density in the fly ash from a particular source is determined. The number of passes is normally applicable to fly ash within a certain moisture content range. If fly ash is arriving from more than one source, the number of passes required for each fly ash is taken in proportion to the fraction of the total fly ash coming from each source, and the resulting mean number of passes is applied to compaction of all the fly ashes.[30]

Compaction Equipment

On any construction project, it is recommended that the suitability of any proposed compaction equipment be verified on a test strip prior to actual construction. It may also be necessary to adjust the compaction criteria as a result of field trials. Figure 53 illustrates the increase in compaction with additional passes of a vibratory roller as determined in one field study.

The most satisfactory compaction results to date have been achieved with self-propelled pneumatic-tired rollers and self-propelled and towed vibratory rollers. Pneumatic-tired rollers with dead weights of 10 to 12 tons (9-11 Mg), including ballast, normally produce 90 percent compaction in 6 passes.[9] Tire pressure should not exceed 36 psi (250 kPa) as greater pressures tend to cut the fly ash surface.[30] An example of good surface finish achieved by a pneumatic-tired roller is shown in Figure 54. Small vibrating rollers with dead weights of 1 to 1-1/2 tons (0.9-1.4 Mg) have provided satisfactory results in about 8 passes. On jobs where greater production rates are required, vibratory rollers with dead weights of 6 to 10 tons (5.5-9 Mg) can be used. Compaction with a 10-ton (9 Mg) vibratory roller is shown in Figure 55. Heavier models tend to seriously overstress the fly ash surface or may tend to bog down. In confined areas, hand-held impact rammers with a large foot are satisfactory. To date, sheepsfoot rollers, smooth-wheeled rollers, and grid and vibrating plates have not been successful on fly ash.[25]

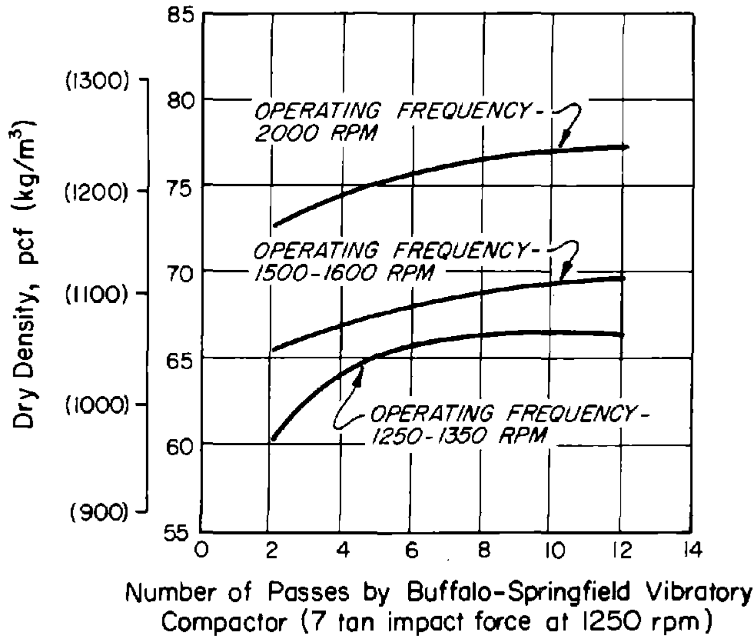

Figure 53. Increase in compaction of fly ash with additional passes of a vibratory roller.(11)

On projects where vibratory or pneumatic-tired rollers have been used, it has been found that tracking the loose layer with a dozer or dump truck provides initial compaction which permits the compaction equipment to operate much more efficently on the fly ash.

The fly ash should be compacted immediately after spreading. If the moisture content of the fly ash is too low to permit the specified compaction, the layer should be moistened with water. The recommended maximum thickness of the compacted layer is 6 inches (152 mm), although this may vary with the size and type of compaction equipment used.

Compaction During Inclement Weather

On many projects, it has been possible to continue compaction during wet weather.[16,20] In such cases it is necessary to adjust the moisture content of the incoming fly ash so that precipitation does not render the fly ash too wet for proper compaction. If the top layer of fly ash becomes too wet during rain, it may soften, with subsequent bogging down of compaction equipment. In these cases, it is recommended that equipment be kept off the fly ash surface until the weather has improved. Fly ash recovers very

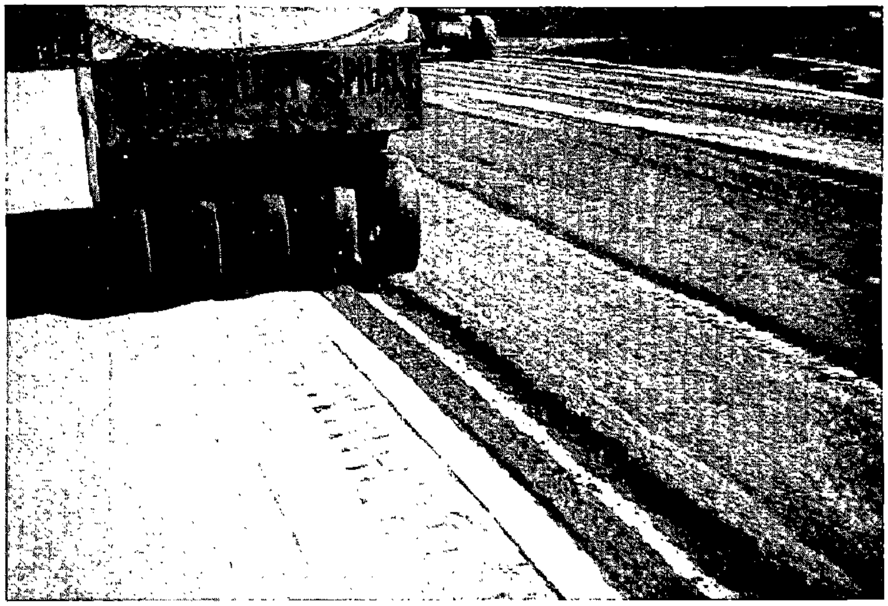

Figure 54. Compaction of fly ash with a pneumatic-tired roller.

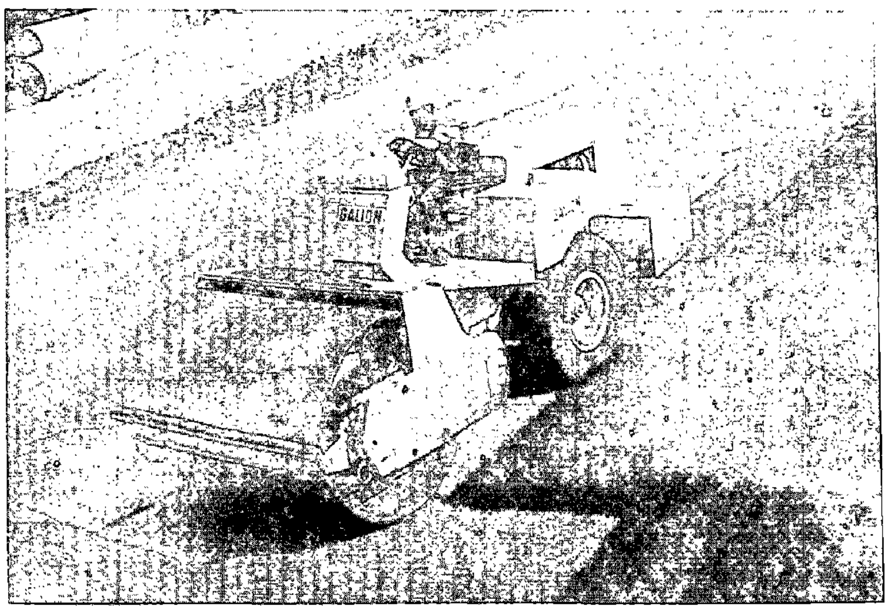

Figure 55. Compaction of fly ash with a vibratory roller.

rapidly from wet conditions, and a few passes of the compaction equipment should be all that is necessary to permit spreading to resume.

If construction is occurring during the winter months and frost sets in, equipment should be kept off the fly ash surface. Upon thawing, a few passes of the compaction equipment satisfactorily reestablishes the surface.

As a matter of practice, all fly ash surfaces should be sloped at the end of each day's work to provide for positive drainage and to prevent either ponding of water, which can result in a saturated condition which produces softness and instability, or runoff channels which can produce erosion of the slopes and potential sediment problems in the area surface waters. On large projects where substantial quantities of fly ash are used, the drainage of the embankment runoff into temporary holding ponds during construction is recommended to permit settling out of particulate matter in the pond before discharging of storm waters into the natural drainage system of the area.

Prevention of Dusting

If a compacted layer of fly ash is exposed to drying weather, high winds, or traffic for any length of time, the top surface can become dry and subject to dusting. Dusting on a construction site is not only an environmental hazard and nuisance, but can also be harmful to motorized equipment. It is therefore necessary to undertake certain precautions to prevent dusting.[8]

1. During dry conditions, high winds, or delays in construction of the next layer, the surface of the top compacted layer of fly ash should be kept continuously moist or, in lieu of this,

    a. Covered with a thin layer of soil, bottom ash, or other granular material not susceptible to dusting;

    b. Stabilized with a small amount of cement to form a thin crust; or

    c. Sprayed with tar or a bituminous emulsion.

2. In addition, all traffic should be restricted to one area or path which has been protected by any of the three aforementioned methods to prevent pickup or dispersion of the fly ash by the tires of passing vehicles.

It is desirable to apply the permanent surface treatment, i.e., topsoil and seeding, immediately after completion of the embankment. If there is any delay which results in the exposure of any fly ash surfaces, one of the aforementioned temporary surface treatments should be applied.

Chemical Stabilization

If chemical stabilization of the fly ash is required to either provide adequate strength or frost resistance, the cement or lime is normally added on a layer-by-layer basis and mixed in place. The fly ash is spread to the required loose lift thickness. Cement or lime is then applied by either distributing bags on a predetermined grid spacing, breaking the bags, and raking or discing the stabilizer into the fly ash; or distributing in bulk with mechanical spreaders. Mixing is usually more efficient if the fly ash is laid in a relatively dry condition, with only enough moisture to prevent dusting. Moisture is then added to the mix which is then compacted to the appropriate compacted layer thickness. Care must be taken in windy weather as the cement or lime dusts very easily.

Construction of Soil Cover

If a soil layer is to be constructed on the surface and slopes of the fly ash for frost protection or a landscaping treatment, it can be applied in two ways. The first method is to spread and compact the soil on the sides of the fly ash, lift by lift, as the fly ash layers themselves are constructed. This method works well for thick layers of soil. The second method, most suitable for topsoil application, is to overconstruct the fly ash slopes and trim back to the appropriate slopes upon completion of the last layer. Since polished surfaces can be created by the trimming process, keys are provided along the slope with a drag line bucket. The soil layer is then constructed on the slopes.

Construction During Freezing Weather

In this country and in Great Britain, construction of fly ash embankments has proceeded throughout the winter months.[2,16] Hopper ashes delivered to the site are often very warm. If these are spread immediately and compacted, the temperature of the embankment tends to stay above freezing. If freezing does occur, frost usually penetrates only the uppermost layer, which can be readily recompacted upon thawing.[20] In any case, care should be exercised in construction during freezing periods if frost-susceptible ash is being used.

Many lagoon ashes are not frost susceptible and would seem to be ideal for winter construction. However, the high moisture content of freshly excavated lagoon ash could result in freezing. Therefore, care should be exercised with these materials, and in no case should frozen material be placed or compacted. Covering on-site stockpiles of fly ash with soil or the like during cold periods can also prevent freezing of the material.

Protection of Concrete and Pipes

The sulphate content of many fly ashes has raised concern about possible sulphate attack on adjacent concrete structures. There have been no reported cases of such attack in the 15 years in which fly ash has been used as embankment or backfill material. If desired, certain precautions can be taken however. These normally are limited to the painting of adjacent concrete faces with tar or a bitumen paint or rubberized compounds which also offer moisture protection to the concrete. The use of sulphate-resisting cement is not necessary.[32]

If concrete is to be poured directly onto a fly ash surface, such as in slab or pavement construction, it is recommended that a moisture proof membrane, such as polythene sheeting, tar paper, or the like, be laid on the fly ash surface to prevent absorption of moisture from the fresh concrete by the fly ash.[32]

Certain chemical and electrical resistivity tests on some fly ashes have indicated that they may be corrosive to pipes imbedded in fly ash fill. Tests performed by British laboratories on the corrosive action of fly ash on cast-iron, lead, copper, P.V.C., and terra-cotta pipes have produced no evidence of harmful action.[32] Each fly ash, however, should be individually evaluated. If protection of pipes is deemed necessary, wrapping pipes in polythene sheeting or imbedding and backfilling with inert material are considered adequate precautionary measures.

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VII. FLY ASH AS STRUCTURAL BACKFILL

A. Factors Affecting Utilization

Fly ash has been successfully used as structural backfill material for bridge abutments, walls, and other types of retaining structures. Its success in this application has resulted from its light weight, good shear strength characteristics, and, in some cases, its self-hardening properties. Actual research and data collection pertaining to the behavior of fly ash in a structural backfill application is limited. Thus, design parameters are generally theoretical. Nonetheless, field performance to date has been highly satisfactory. The result in Great Britain, where fly ash has been used extensively behind bridge abutments, has been the achievement of smooth transitions from approach slabs to bridge decks as indicated by the reduction or elimination of the noticeable "bump" which occurs at the junction of the two as a result of differential settlement.

Because of its light weight, fly ash is an ideal backfill material over soils of low bearing capacity, since the resulting settlement in the poor subgrade is kept to a minimum. In addition, the settlement within the fly ash backfill takes place almost entirely during the construction period. The light weight and good shear strength characteristics of fly ash can also offer advantages in the design of retaining structures in terms of reduced lateral pressures. The self-hardening properties of some fly ashes further reduce actual settlement in the backfill. In addition, fly ash is relatively easy to handle and can be placed and compacted using small rollers in the confined areas behind bridge abutments or other retaining structures.

As in other applications of fly ash, dusting can be controlled by maintenance of an appropriate moisture content; erosion can be controlled with proper surface protection; and frost susceptibility can be eliminated through stabilization or provision of adequate cover. Each of these items is discussed in detail in Section VI as it relates to embankment construction.

B. Case Histories

The use of fly ash as structural backfill has occurred in Great Britain since the early 1960's. The material has been utilized quite extensively in the new British Motorway System (similar to the U. S. Interstate System). A number of case histories point up the successful application of fly ash as structural backfill. Photographs of fly ash backfills at two locations not reported below are shown in Figures 56 and 57.

Backfill Behind Retaining Wall, Perry Barr Expressway, Birmingham, England

Approximately 10,000 cubic yards (7,600m^3) of stockpiled fly ash were used as load bearing fill behind a 13-foot (4 m) high retaining wall along a section of the Perry Bar Expressway in Birmingham.(2) Fly ash was chosen for the backfill because of its relatively low density and its self-hardening properties. The low density was essential because of the low allowable bearing capacity of the subgrade, and advantage was taken of the self-hardening properties, with the resultant increases in shear strength, to reduce the active pressure in the design of the retaining wall. The material was spread in 9-inch (229 mm) loose lifts and compacted with a lightweight vibratory roller. The strength development of the fly ash backfill was determined from unconfined compression tests as indicated in Table 25.

Backfill for Bridge Abutments and Wing Walls, Staples Corner Interchange, North London, England

Fly ash was used as a backfill material for bridge abutments and wing walls at the Staples Corner Interchange in North London.(4) Fly ash was specified because of its light weight and self-hardening properties, thus reducing settlement behind the bridge abutments and lateral pressures on the precast concrete wing wall sections. The maximum height of fly ash backfill was 25 feet (7.6 m) and the total volume of fly ash used in this project was 35,000 cubic yards (27,000 m^3) compacted.

Backfill for Bridge Abutments, A.308, Staines By-Pass, near Staines, England

Fly ash was used as backfill at two bridge sites on the Staine By-pass on the A.308.(7) In order to study the problem of differential settlement of the road surface at bridge abutments, fly ash was used as backfill at one end of each bridge while a well-graded sandy gravel was used at

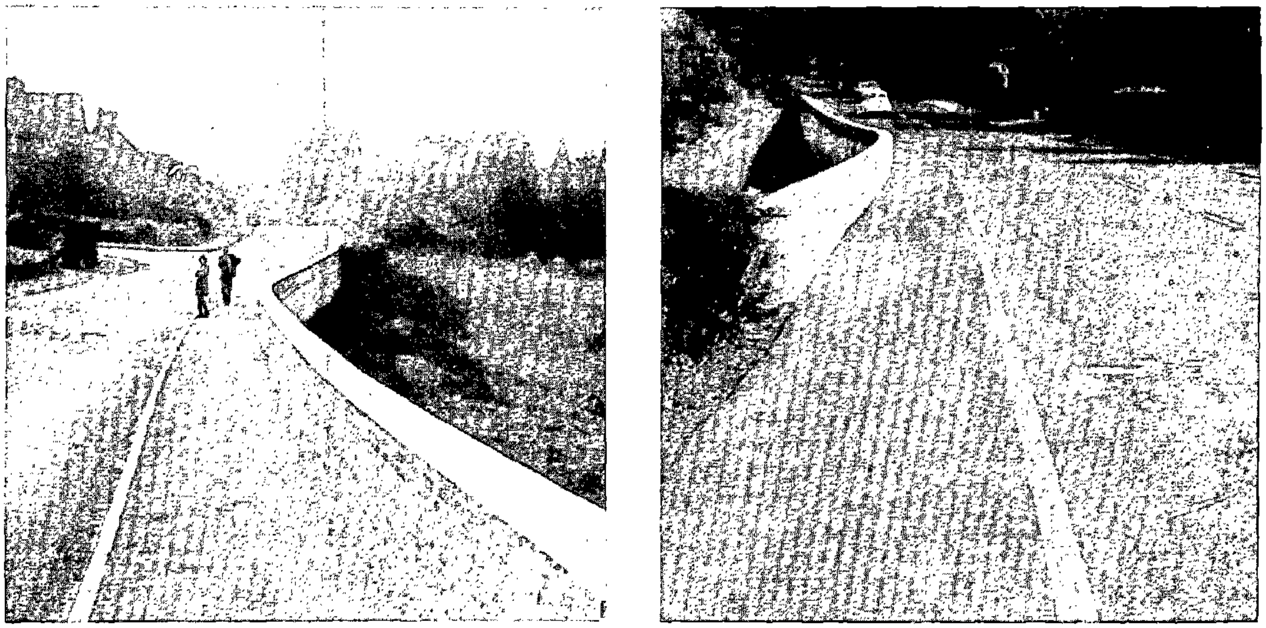

Figure 56. Completed fly ash backfill behind retaining wall at Bidborough, Kent, England.

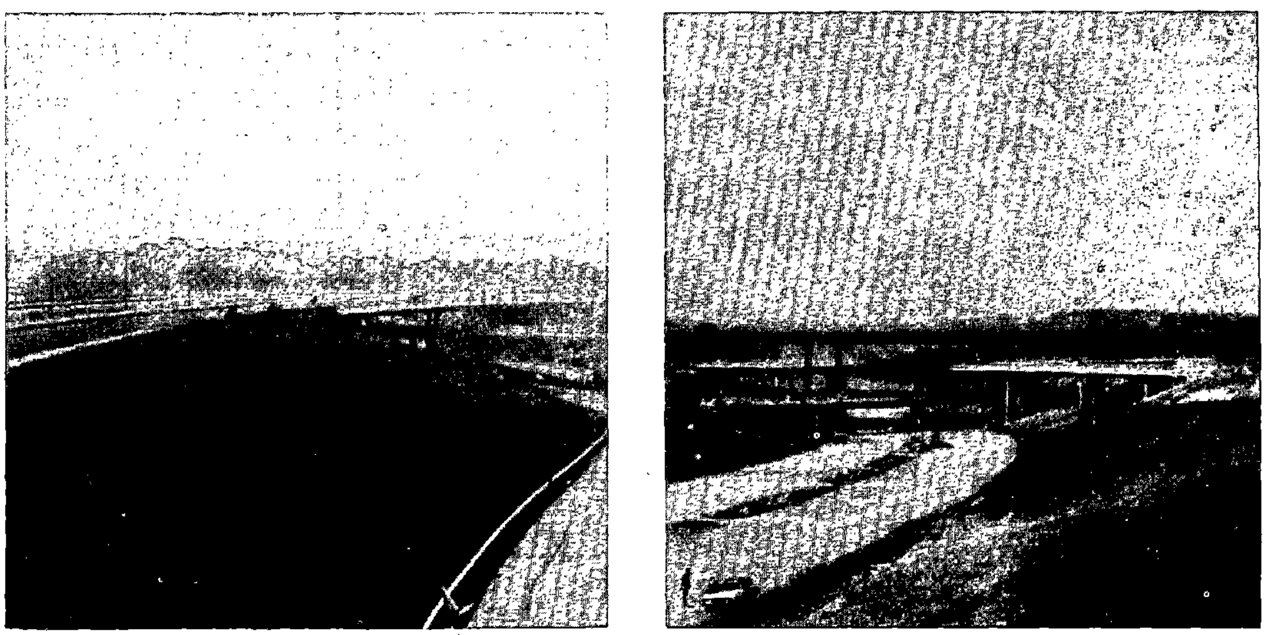

Figure 57. Construction of the M.23-M.25 interchange, England, where fly ash was used in the embankments and as structural backfill behind the bridge abutments.

Table 25. Strength development in structural backfill of fly ash with self-hardening characteristics.[2]

AGE OF BACKFILL months	UNCONFINED COMPRESSIVE STRENGTH psi (kPa)
3-1/2	9.3-34.0 (64-234)
5-1/2	12.9-35.3 (89-243)
8-1/2	17.7-36.4 (122-251)
14-1/2	23.3-41.0 (161-283)

the other end. The well-graded sandy gravel is considered to be a high quality backfill material.

Fresh hopper ash was used in this project. Relatively light compaction equipment was used for constructing the backfill.

The construction of the backfills was carefully observed and records of the settlement of the road surface and the subsoil were obtained. The backfilling operations were carried out between May and October of 1963.

After a monitoring period of about 1-1/2 years following completion of the backfills, it was found that the settlement within the fly ash backfills at both bridges was less than or equal to that within the granular backfills, and was limited to 0.10 inch. Settlement in the subgrade beneath the fly ash backfills was approximately half of that which occurred beneath the granular backfills.

In spite of some difficulty with the control of moisture content of fly ash, the fly ash backfills were completely satisfactory and have performed as well as, or better than, those built of well-graded sandy gravel.

C. Structural Design Considerations

In the past, conventional methods of determining lateral pressures exerted by granular backfill have been used in the design of bridge abutments and wing walls which were to be backfilled with fly ash.(3) No cases of failure or unsatisfactory performance by any retaining structures backfilled with fly ash have been reported to date. However, attempts have been made in the field to measure the actual lateral pressures produced by fly ash backfill in order to verify that fly ash behaves as a granular material in this application(6,9,11,12) particularly fly ash which experiences self-hardening with age.

Smith assumed that active pressures would be developed in some cases when typical British fly ashes were used as a backfill.(10) Using the example of a fly ash backfill compacted to 90 percent of maximum dry density and possessing an angle of internal friction of 31° and a cohesion of 10 psi (70 kPa) three days after compaction, he computed a negative value for the horizontal component of active pressure equal to -1094 psf (50 kPa) for a retaining wall 20 feet (6 m) in height. More favorable results would be expected if allowances were made for the increases in shear strength with time which occurred in that particular fly ash.

When the lateral pressures on the wall are small or negative in value, the Civil Engineering Code of Practice for Earth Retaining Structures followed in Great Britain suggests that design can be carried out on the assumption that the horizontal pressure at any depth is equal to that from a fluid having a density of 30 pcf (480 kg/m^3). This figure was considered by Smith to be unduly conservative when designing retaining walls having fly ash backfill. In the case of the design of a particular bridge retaining wall with the fly ash backfill, Smith has reported that a minimum horizontal pressure equivalent to that from a fluid with a density of 15 pcf (240 kg/m^3) was used with the approval of the British Ministry of Transport.(8)

In order to determine the probable distribution of lateral pressures exerted by fly ash backfill, field investigations have been carried out by engineers in Great Britain and the United States.(6,9,11,12) In a field investigation in Great Britain reported by Wilson, et al., pressure cells and thermo-couples were installed at four levels in the fly ash backfill behind a rigid retaining wall and surveys of the face of the wall carried out at intervals.(12) Measurements during the period of filling showed that significant lateral pressures were induced, apparently by the compaction process. The measured lateral pressures were generally greater than the active earth pressures assumed in the design. The very small wall movements observed are reported to be responsible for such high lateral pressures since full shearing strength of fly ash could not be mobilized to decrease the pressures to correspond to the "active" values.

Full scale field tests were carried out in the United States recently by Joshi, et al.,[6] to measure lateral and vertical pressures within fly ash backfill behind a rigid retaining wall. The fly ash used in the backfill possessed self-hardening properties. The measured pressures recorded by cells at three elevations in the backfill were compared to theoretical at-rest values computed for an average measured total unit weight of 86 pcf (1380 kg/m^3) and coefficient of lateral earth pressure at rest, k_o, of 0.5. On the average, recorded lateral pressures at all levels were less than calculated values.

Joshi, et al.,[6] suggested that the recorded lateral pressures were minimal and lower than the theoretical at-rest values as a result of light compaction of the fly ash with a vibrating plate, which prevented high locked-in lateral pressures normally produced during the compaction process. Furthermore, the fly ash was placed in 2 to 3-foot (0.6-0.9m) stages and allowed to "set" a sufficient period of time before construction of the next stage. Thus, hardening of each stage occurred before placement of the upper layers.

In summary, no conclusive results were produced in the various field investigations on fly ash backfill behind retaining structures. However, as indicated by the work of Casagrande[1] and Jones and Sims[5], even in conventional backfill the determination of lateral pressures is not a simple, straight-forward process. The engineer should be guided, however, by actual field conditions when determining the magnitude of lateral earth pressures produced by a fly ash backfill on any retaining structure. The field conditions which will influence the magnitude of the lateral pressures include:

1. Compaction of the fill near the wall;

2. The flexibility of the retaining structure; and

3. The properties of the particular fly ash used in a given situation.

D. Testing and Evaluation

In addition to the determination of the usual index properties, the shear strength characteristics of the particular fly ash proposed for use as backfill should be determined for designing the retaining structures. Tests for determining these properties have been described in Section VI. Compaction characteristics of the fly ash can be determined in accordance with AASHTO T 99-74.

Self-hardening properties of the fly ash are usually measured by CBR or unconfined compression tests. Consolidation characteristics of the fly ash can be determined by suitable laboratory tests as described in Section VI.

E. Construction Procedures

Most of the usual procedures for constructing a fly ash embankment are applicable to the structural backfill operations. These are discussed in detail in Section VI. These construction procedures are quite similar to those for conventional soils, except that certain precautions should be taken to prevent dusting or erosion of the fly ash. In addition, certain precautionary measures are taken to protect concrete surfaces from sulphate attack from adjacent fly ash. This protection is usually the application of a tar or bitumen paint. No cases of sulphate attack have ever been reported, however. Also, construction of a drain along the face of the wall or retaining structure is recommended since fly ash is not considered to be free-draining material. The drainage filter should be properly sized to prevent washout of the fine fly ash particles.

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VIII. FLY ASH IN GROUTING

A. Factors Affecting Utilization

The objectives in grouting are, generally, to:

1) increase the strength;

2) reduce the permeability;

3) reduce the compressibility; or

4) fill undesirable voids

in the formation being grouted. Traditionally, grout components have ranged from gravels and sands to cement and clays in dry or slurried form.

Several desirable properties of fly ash have led to its use in grouting. These include:

1) particle shape and size;

2) unit weight;

3) gradation; and

4) pozzolanic activity.

Each of these properties is briefly discussed below.

Particle Shape and Size

The fineness of particle size (some fly ashes are finer than cement)[9] and the predominantly spherical shape of fly ash particles enables grouts incorporating fly ash to be pumped more easily than those containing only cement or sand. The spherical shape results in a ball-bearing effect which enhances the flow properties of the grout.[10] Partial replacement of either cement or sand by fly ash improves pumpability and injection by keeping the grout in suspension and thus reducing sedimentation.[8]

Unit Weight

The unit weight of fly ash varies between 2.1 gm/cm^3 and 2.6 gm/cm^3 which is appreciably lower than that for cement (3.15 gm/cm^3) or sand (2.7 gm/cm^3).[9] This is often advantageous where weight is a critical factor; in addition, sedimentation is reduced because of the low unit weight.

Grading

Fly ash particles are mainly of silt size, but fly ash also contains a small but significant percentage of finer clay-sized material. Although the coefficient of uniformity is low, these clay-sized particles provide sufficient grading to reduce segregation during pumping and injection and result in lower voids and increased durability of the grout when placed.(3)

Pozzolanic Activity

The pozzolanic properties of fly ash, as defined in Section II, enable it to combine with lime to produce a stable cementitious material. Since the hydration of cement also produces lime, additional cementation results when fly ash is added to cement. This reaction provides a more effective bond than that between sand and cement in weak cement grouts.(3)

Fly ash does contain a certain amount of water-soluble sulphate. However, the pozzolanic reaction between fly ash and lime released during the hydration of cement has been shown to actually increase the sulphate-resistance of a cement grout.(9)

B. Case Histories

Fly ash has been used extensively for years in grouting operations for foundations, stabilization, subsidence prevention, void filling, and other uses related to building construction. It was natural, therefore, that such usage be extended to similar situations involving highway-associated structures. The most widely publicized uses of fly ash grout in highway construction have occurred in Great Britain, although such applications are numerous in this country as well. Several case histories are reported below.

Soil Grouting, M.6 Midland Link Motorway, England

The proposed route of the Midland Link Motorway crossed an industrial disposal area approximately 160 yards (146 m) long and 35 yards (32 m) wide.(5) The depth of the industrial fill varied from 45 feet (13.7 m) to 54 feet (16.5 m). The fill material was of inadequate bearing strength and represented a potential fire hazard. It was decided to eliminate both problems by injection grouting the voids in the fill. The grout used consisted of 10 parts fly ash to 1 part cement, with a water to cement ratio of 1:1. A series of holes 3-1/2 inches (89 mm) in diameter were drilled, lined with casings of 2-inch (51 mm) internal diameter, and grouted. Primary and secondary holes were drilled at 18-foot (5.5 m) and 9-foot (2.7 m) centers respectively. An estimated 3,000 tons (2,700 Mg) of fly ash was to be used before the grouting was complete.

Stabilization of Bridge Approaches Lewis and Baxton Counties, West Virginia

The West Virginia Department of Highways used a fly ash cement slurry to stabilize the bridge approach slabs across all four lanes on two structures of Interstate 79 in Lewis and Baxton counties in West Virginia.(7) Similar treatment was scheduled for sections of Interstate 77 and State Route 2 in the Parkersburg area.

The mudjacking procedure involved the preparation of a batch mix with two bags of fly ash to one bag of cement. In addition, the maintenance crews added two shovels of Bentonite clay or local shaley clay and one shovel of calcium to the mix. The addition of calcium was necessitated by the low temperatures.

Sufficient water was added to the mixture placed in a mixer to obtain a slurry. This material was pumped under a pressure of 290 psi (2000 kPa) into 2-1/2 inch (64 mm) diameter holes drilled into the abutment backfill. Grout holes were cut through the concrete slab, base course material, and the backfill to the solid ground below. The application of the fly ash-cement slurry was reported to be successful with less than 1 percent shrinkage, and good flows and better strengths were achieved with fly ash as compared with previously used bagged rock dust (ground limestone) or available clays.

Remedial Work to Railway Tunnel Stockton Brook, England

A 200-foot (61 m) railway tunnel was located along the Stoke to Leek Goods Line in the London Midland Region of the British Rail System.(8) Minimum cover existed between the tunnel and the roadway above. It was decided to strengthen the tunnel by erecting an Armco arch within the existing tunnel. A commercially-produced cement-fly ash grout, known as "Basegrout," was used to fill the void between the Armco arch and the existing tunnel. The grout mix consisted of 10 parts fly ash to 1 part cement by weight. The mix was delivered to the site in covered trucks and in a slightly dampened condition. An access shaft was drilled from the roadway down through the existing cover and into the void, and both ends of the void were bricked in. The grout mix was slurried from the trucks into a ground hopper at the rate of 15 tons (14 Mg) every 30 minutes using a high-pressure water hose. The slurry was discharged into the access shaft and placed by gravity. The carrying water dispersed through the existing tunnel masonry. Approximately 800 tons (730 Mg) of grout were placed. The work occurred during October 1973.

C. Design Considerations

Selection of Grout and Fly Ash Type

There are several types of grouts in which fly ash can be utilized:[3]

a) grouts containing only fly ash;

b) lime-fly ash grouts;

c) Portland cement-fly ash grouts;

d) Portland cement-fly ash-clay grouts; and

e) Portland cement-fly ash-sand grouts.

The choice of the type and technique of grouting will depend on the circumstances of the particular case. The choice of the fly ash for use in the grout will probably be based on the location of the nearest power plant. Since the chemical and physical properties of the fly ash will affect the characteristics of the grout, some consideration should be given to the fineness, unit weight, free lime content, and carbon content of the fly ash(es) being considered for use. As previously mentioned, high fineness, low unit weight, and high free lime content are all desirable properties in fly ash to be used for grouting purposes. High carbon content, however, can inhibit the pozzolanic activity of the fly ash as well as increase the viscosity of slurried fly ash. It is recommended that grout applications be restricted to fly ashes with carbon contents less than 10 percent.[9]

Mix Proportions and Strength Data

Mix proportions for various grouts are determined on the basis of desired strength, setting time, flow properties,

Table 26. Strength development in various fly ash grouts.[10]

GROUT PROPORTIONS by weight	AVERAGE COMPRESSIVE STRENGTH psi (kPa)		
	7 Days	28 Days	3 Months
1 cement : 1 fly ash : 2 water	400 (2760)	1000 (6890)	1750 (12,070)
1 cement : 2 fly ash : 7 sand : 14 coarse aggregate 1/2" diam. : 2 water	470 (3240)	780 (5380)	1100 (7580)
fly ash plus enough water to achieve a cream-like consistency	140 (970)	-	-

weight, and other factors which are a function of the particular application. Trial mixes are generally the most straightforward method of determining required mix proportions. Data is presented in Table 26 on the strength development in various grouts as an example of the effect which various components and proportions have on the properties of grouts.

1. Grouts Containing Only Fly Ash

These are cheap, low-strength grouts useful for filling large cavities in the ground, such as old mine workings, abandoned sewers and terminals, etc. Most fly ash-water mixtures will slurry at a moisture content of about 35 percent (except lagoon and some stockpiled ashes, which have inherently high optimum moisture contents). However, the viscosity will probably be too high for pumping. Such mixtures do, however, flow easily at a moisture content of about 50 percent,[3] and it is possible to obtain fills with densities comparable to those obtained by compacting the fly ash at its optimum moisture content provided that the excess water needed for pumping can be subsequently drained away.

Fly ash with self-hardening properties can be used for void filling where some strength is required. As indicated in Section II, the free lime content in some ashes is large enough to produce a strength gain with time in the fly ash. Actual strengths developed are a function of the particular fly ash. In these cases, it is only necessary that any slurry water drain quickly from the fly ash, since the free lime is water-soluble. Fly ashes produced from western coals or limestone injection processes have considerable potential in this application.

2. Lime-Fly Ash Grouts

Hydrated lime-fly ash grout mixtures have been used successfully in a number of applications, including oil well grouting and soil stabilization.(6)

Since lime is much finer than fly ash and forms a more colloidal suspension, the particles in the grout do not segregate so readily and the pumpability is improved.(3) Also, lime increases the pozzolanic activity and the final strength of the set grout. Optimum lime-fly ash ratio for maximum strength is reported to be about 1:2.5. Although the rate of strength increase with the lime-fly ash mixes is appreciably lower than that of cement-fly ash mixes discussed below, the lime-fly ash mixes are found to attain strengths in time comparable to that of the cement-fly ash grouts. Furthermore, in many areas of the United States, the cost of lime is less than that of Portland cement. Therefore, in cases where maximum penetration is required or a slow rate of strength gain is acceptable, the use of lime-fly ash grouts may provide the best solution.(3)

3. Portland Cement-Fly Ash Grouts

Since fly ash has both pozzolanic properties and approximately the same grain-size distribution as cement, it can be used as a partial replacement for cement in cement grouts. Fly ash improves the flow properties, thus pumpability and workability of cement grouts. Experience with cement-fly ash grouts has shown that grouts with a 25 percent replacement of cement with fly ash can develop strengths after 3 months equivalent to grouts containing cement alone.(9) The cement-fly ash ratio can be varied to produce a wide range of strengths thus enabling one to choose the most economical proportions giving the desired strength instead of using an expensive, very strong cement grout for all applications. The ratios of cement to fly ash in common use vary from 1:3 to 1:10 depending on the strength and elastic properties desired. The strength is a function, also, of the water-to-solids ratio. Grout strength usually increases as the water ratio decreases.(4) This is illustrated in Figure 58. In addition, cement-fly ash grouts generally require less water than cement grouts to obtain a workable mix.(1)

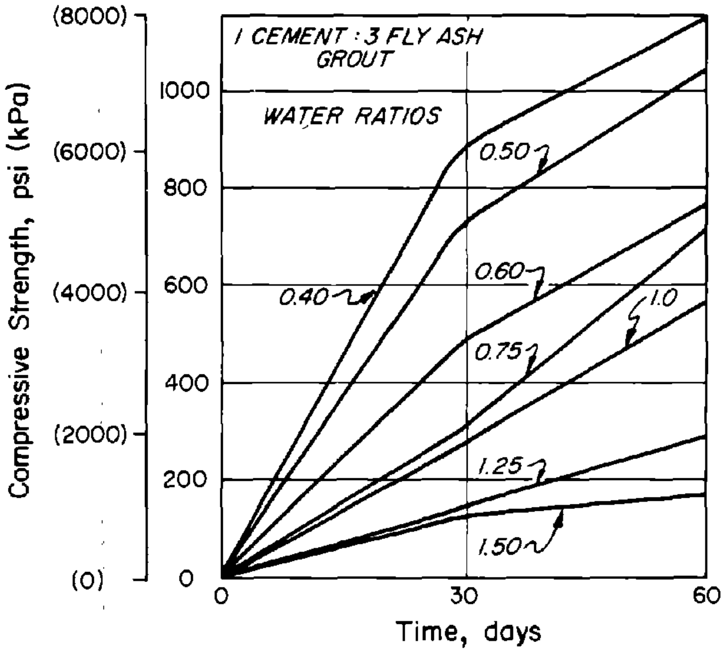

Figure 58. Effect of water-to-solids ratio on the time-strength development of cement-fly ash grout.(1)

In addition to its other advantages, cement-fly ash grout has a lower heat of hydration than cement grout. This can be of importance in situations where high temperatures are encountered, such as grouting of deep wells.(9)

Addition of certain accelerators such as soda ash (sodium carbonate) and calcium chloride has been reported to favorably affect the stability, setting time, fluidity, and consistency of fly ash-cement grouts.(2)

4. Portland Cement-Fly Ash-Clay Grouts

Bentonite is often added in quantities of up to 8 percent by weight to cement-fly ash grouts to aid in maintaining cement particles in suspension. Active bentonite, tends to make the grout more viscous, although pumpability and penetration characteristics are enhanced. The effectiveness of bentonite in cement-fly ash grouts should be verified by trial mixes, however, since the calcium ions in either the cement or the fly ash can reduce the suspending action of the bentonite.(3)

5. Portland Cement-Fly Ash-Sand Grouts

Sand and even gravel can be added to cement-fly ash grouts in situations where the grain-size is not a severe limitation. The wider ranges of particle sizes enable denser grouts to be produced which can have properties similar to those of good quality concrete. The lubricating action of the rounded particles of fly ash improves the pumpability of dense grouts.[3]

E. Construction Procedures

Generally speaking, grouting procedures for non-fly ash grouts are applicable to grouts containing fly ash. Suggested construction procedures can be found in References R-1 and 4.

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6. McLaren, Robert J. "Test Results, Lime-Fly Ash Grout for Subsurface Stabilization," unpublished memo, GAI Consultants, Inc., November 17, 1972.

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8. PFA Basemix, Ltd., "Project Case History - Remedial Works to Railway Tunnel at Stockton Brook on Stoke to Leed Goods Line," Holme, England.

9. Smith, P. H. "Fly Ash in Grout," Civil Engineering and Public Works Review, November-December, 1972.

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Suggested Additional References

R-1. American Society of Civil Engineers, "Symposium on Cement and Clay Grouting of Foundations," Journal of the Soil Mechanics and Foundation Division, Vol. 84, No. SM1, February 1958.

R-2. American Society of Civil Engineers, "Symposium on Grouting," Journal of the Soil Mechanics and Foundation Division, Vol. 87, No. SM2, April 1961.

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APPENDIX A

PROPOSED CUTOFF DATE PROCEDURES FOR CONSTRUCTION WITH LFA MIXES

General

Strength development of LFA 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 ($\overline{DD}$), is required to develop a desired cured strength ($\overline{CS}$). For typical conditions in many northern states, little beneficial curing can be achieved after November 30 on a predictable basis.

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, $\overline{RS}$) is generally less than the $\overline{CS}$.

A certain minimum strength called the Minimum Tolerable Strength ($\overline{MTS}$) is required for LFA mixes to insure adequate performance in a pavement system. $\overline{MTS}$ will vary depending on whether the LFA is utilized 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 $\overline{MTS}$.

Procedure

The procedure outlined herein describes the steps required to determine, in a systematic and rational manner, the appropriate cutoff date for construction involving LFA mixes utilized in a specific pavement system and having specified cured strength characteristics. The procedure is based on the "Residual Strength" concept developed in IHR-401,[3] and published in Highway Research Board Bulletin No. 442.[1] Figure A-1 illustrates the "Residual Strength" ($\overline{RS}$) and "Minimum Tolerable Strength" ($\overline{MTS}$) concept discussed above.

1. Establish a cured strength-degree day ($\overline{CS}$-$\overline{DD}$) relationship for the LFA mix. An example of this relationship for an LFA mix is shown in Figure A-2. Calculate the $\overline{DD}$ using a 40° F (4° C) base temperature.[2]

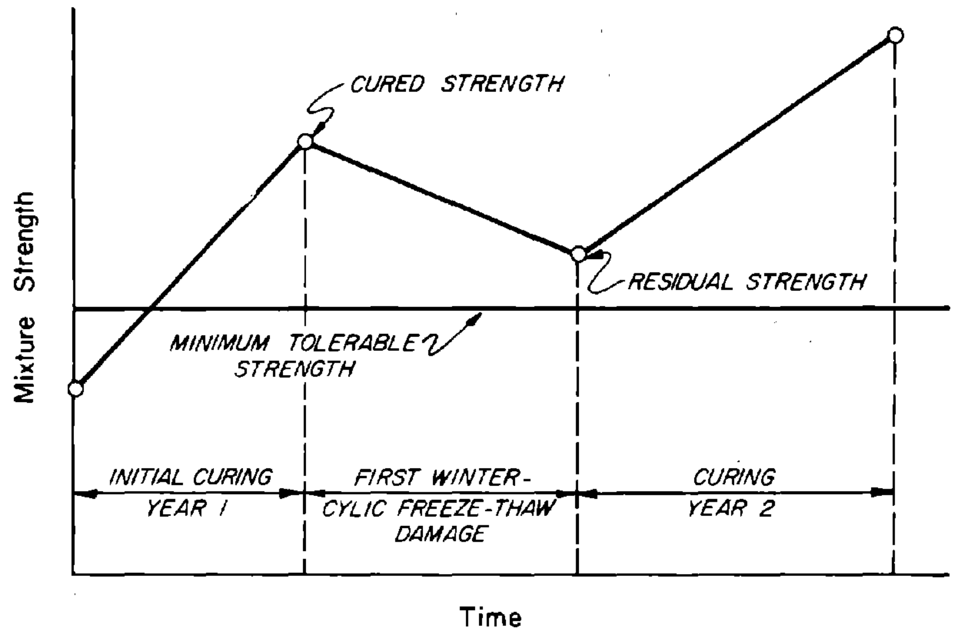

Figure A-1. Residual strength and minimum tolerable strength concept for LFA and LCFA mixtures.

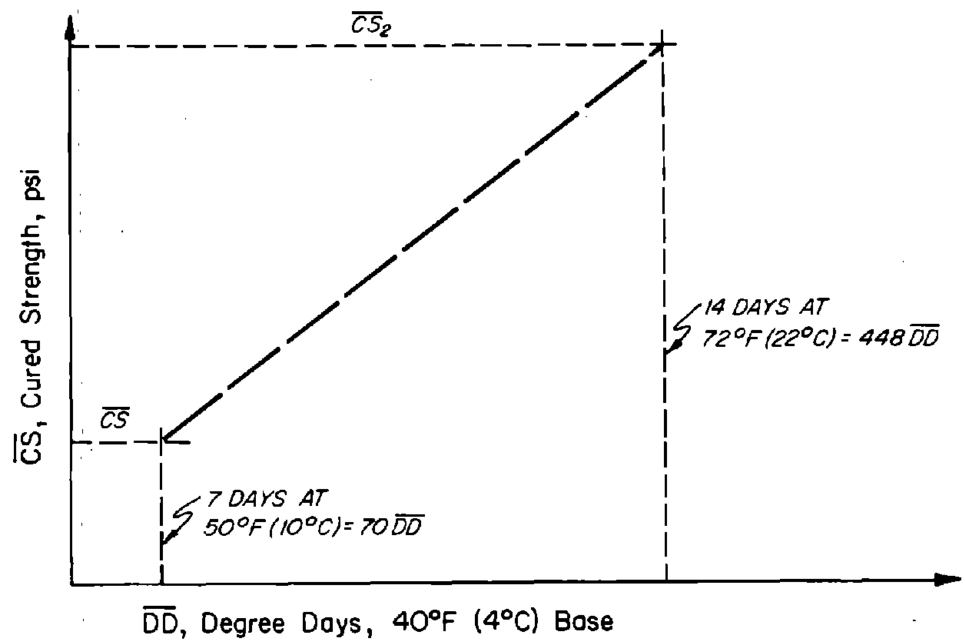

Figure A-2. Typical cured strength-degree day relationship for an LFA mixture.(2)

2. Minimum Tolerable Strength ($\overline{MTS}$) requirements for LFA mixes are given in the specifications or can be determined from pavement design criteria.

3. Cured Strength ($\overline{CS}$) requirements must be consistent with minimum $\overline{MTS}$ values selected to provide a $\overline{RS}$ greater than the $\overline{MTS}$ as illustrated in Figure A-1.

4. From Step 1 data, determine the DD required to achieve the $\overline{CS}$ requirement selected in Step 3 or given in the specifications.

5. From Figure A-3, select the appropriate CUTOFF DATE to provide the accumulation of an adequate number of $\overline{DD}$ for curing the LFA mix.

6. Adjust the CUTOFF DATE determined in Step 5 for construction and curing variability. As an example, the suggested adjustment for the Chicago, Illinois, area is to set the CUTOFF DATE 7 days earlier than the data obtained from Step 5. The 7 days adjustment is equivalent to approximately 175 $\overline{DD}$ during mid-October in Chicago.

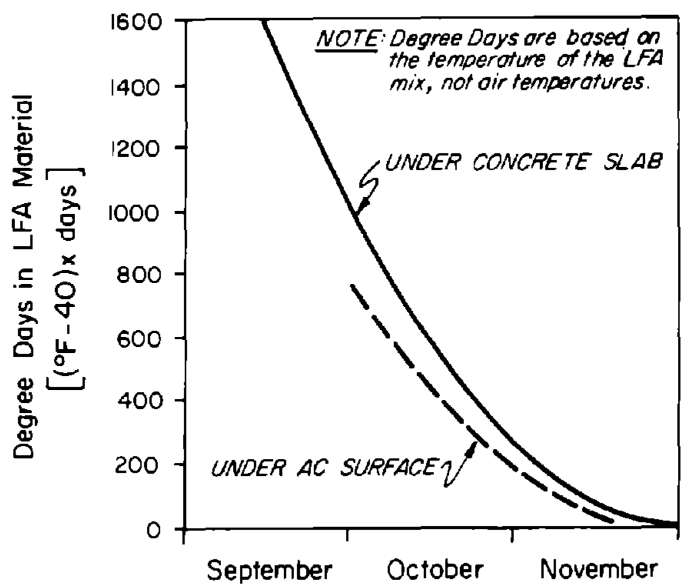

Figure A-3. Typical degree day-cutoff date relationship for a given geographic location.(2)

References

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2. MacMurdo, F. D., and Barenberg, E. J., "Determination of Realistic Cutoff Dates for Late Season Construction with Lime-Fly Ash and Lime-Cement-Fly Ash Mixtures," Record No. 442, Highway Research Board, 1973, pp. 92-101.

3. Unpublished Data, Project IHR-401, Illinois Cooperative Highway Research Program, University of Illinois, Urbana, Illinois, November 1971.

APPENDIX B

THE EQUAL-COST-LINE METHOD OF TRIAL MIX SELECTION FOR LIME-FLY ASH-SOIL MIXTURES(1)

Several methods are available for selecting trial-batch mix proportions of lime, fly ash, and soil for further testing as base course or subbase course material. However, one way to short-cut the testing program is to eliminate all uneconomical trial mixes. First, a maximum allowable lime percentage is selected which is economically competitive with other types of construction, such as soil-cement, crushed stone, etc. As a hypothetical example, this is plotted as Point A in Figure B-1. Next, the cost of handling an additional material (i.e., fly ash) is estimated and expressed as its equivalent in percentage of lime. This is subtracted from Point A to give Point B. Starting at Point B, an "equal-cost line" is drawn with a negative slope equal to the cost of fly ash divided by the cost of lime, both on a dry, delivered, per-ton basis.* Trial mixes are then selected from the area below this line, since proportions above the line are uneconomical. A second limitation which can be imposed is to require a minimum of three percent lime, since lower lime contents may lead to lean areas in the field due to imperfect mixing. This limit is represented by line CD. A lower minimum content may be permissable if lime is applied in a slurry.

Selection of trial mixes from within triangle BCD is partly a matter of judgment. If maximum strength is desired, equal-cost points are selected at Point A and along line BD. For example, in Figure B-1 one might select 10 percent lime, 90 percent soil (Point A), then 7 percent lime plus 8 percent fly ash, abbreviated as 85:7:8 soil:lime:fly ash (Point E), then 79:5:16 (Point F), and 73:3:24 (Point D). Intermediate points can be filled in if desired. Ordinarily, one of these mixes will give the highest strength and durability. If the resulting strength is excessive for the proposed use, costs can be cut by using less fly ash or less lime or less of both. For example, if Points F and D in Figure B-1 are found to be about equally overdesigned, the more economical ratio 82:3:15 (Point G) could be tried. Or if Point F is the best mix, trials could be made at points intermediate to Points F and H, thus maintaining the same lime: fly ash ratio. If none of the trial mixes gives satisfactory strength and durability, the lime or fly ash or soil type may be at fault, or chemical accelerators can be tried.

*In this method mix percentages must be expressed on a total-dry-mix basis.

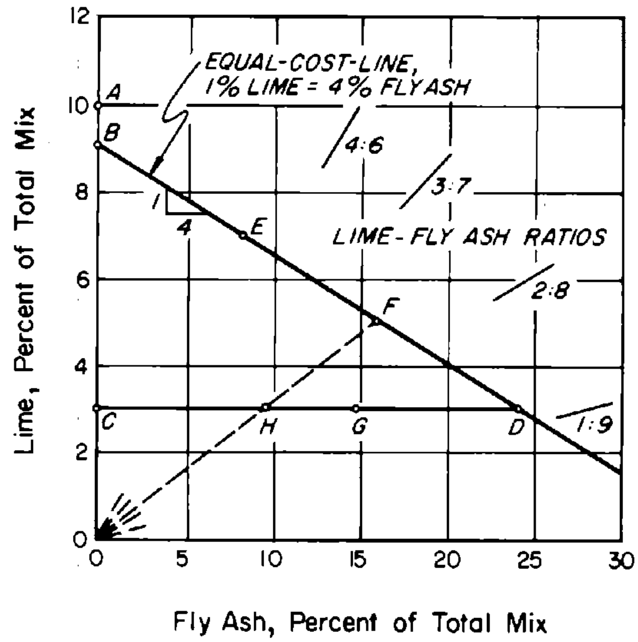

Figure B-1. Example of Iowa equal-cost-line method of trial mix selection for lime-fly ash-soil mixtures.(1)

Reference

1. Davidson, D. T., and Handy, R. L., "Soil Stabilization," Highway Engineering Handbook, K. B. Woods (Ed.), McGraw-Hill Publishing Company, Inc., 1960, pp. 21-102 to 21-103.

PHOTO CREDITS

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