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16. Abstract  The objective of this study was to exploit the beneficiation and utilization of coal combustion by-products (CCBPs) as low-cost alternate aggregates and stabilizers in roadway and airfield construction. A literature review and laboratory investigation were performed to investigate the use of CCBPs in three highway construction applications: (1) fly ash for subgrade stabilization, (2) flue gas desulfurized (FGD) gypsum and Gypsite for base and subbase construction materials, and (3) bottom ash (with sulfur modification) in asphaltic concrete mixtures.  Results from field trials utilizing CCBPs in roadbases indicate good to excellent performance characteristics and negligible environmental impact. Class C and lime-enhanced Class F fly ashes were shown to be effective stabilizers for both clayey and sandy soils. The field trials showed that sulfate-resistant cements and Class C fly ashes provide a wide range of strengths in stabilized sulfate and sulfite roadbases.  The use of sulfur improved the crushing strength and reduced the asphalt demand associated with bottom ash in asphaltic concrete mixtures. Sulfur modification required asphalt contents equal to and often less than for mixtures using conventional aggregates.  Based on its availability, lower cost, safe use, and demonstrated performance, CCBP materials should find increasing use especially in those regions of Texas experiencing dwindling or non-existent aggregate sources.					
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# **EVALUATION OF THE USE OF COAL COMBUSTION BY-PRODUCTS IN HIGHWAY AND AIRFIELD PAVEMENT CONSTRUCTION**

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## IMPLEMENTATION RECOMMENDATIONS

The following are recommendations for implementing the use of coal combustion by-products in highway and airfield pavement construction:

1. This project was proposed as Phase I of a two-phase study. It is recommended that Phase II (a field testing research program) be implemented at this time. It is recommended that a research project (either in-house or university-based) be established to design, monitor construction and performance of field test sections incorporating coal combustion by-products into highway and airfield pavement construction as discussed in this report. It is anticipated that all test sections could be constructed in one year. Performance monitoring should be continued for a minimum of two years.
2. It is recommended that TxDOT construct the following test sections to evaluate the constructability and performance of coal combustion by-products in highway/airfield construction:

### **CCBPs as Subgrade Stabilizers**

#### ***A-3 Soil:***

- |                        |   |
|------------------------|---|
| <i>Test Section 1.</i> | <i>15% Class 'C' fly ash.</i>                 |
| <i>Test Section 2.</i> | <i>20% Class 'C' fly ash.</i>                 |
| <i>Test Section 3.</i> | <i>3% Quicklime and 8% Class 'F' fly ash.</i> |

#### ***A-7-6 Soil:***

- |                        |   |
|------------------------|---|
| <i>Test Section 1.</i> | <i>Control - 4% Quicklime.</i>                |
| <i>Test Section 2.</i> | <i>2% Quicklime and 5% Class 'F' fly ash.</i> |

*Test Section 3. 10% Class 'C' fly ash.*

*Test Section 4. 15% Class 'C' fly ash.*

### **CCBPs as Road Base Materials**

*Test Section 1. Cement Stabilized or HVFA Cement Stabilized FGD.*

*Test Section 2. Fly Ash (Class 'C') Stabilized FGD.*

### **Sulfur-Modified Bottom Ash (SMBA) Asphaltic Mixtures**

Researchers believe that SMBA asphaltic mixtures have potential for use on general aviation runways and that this would be an excellent field test; however, it is recommended that an even smaller scale field experiment be developed first. It is recommended that a field experiment (100 meters in length) be placed at a location such as Texas A&M's Riverside Campus. The aggregate fraction of this mixture could be either 100% bottom ash or 50% bottom ash as determined by the Project Director. Any of the mixture designs presented in Section IV would be a good starting point for development of a field trial.

3. Preliminary specifications are presented in Section V of this report. These should be evaluated and modified as needed during the field testing portion of a Phase II research program.

## **DISCLAIMER**

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation (TxDOT) or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes.





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## SUMMARY

The objective of this study is to exploit the beneficiation and utilization of coal combustion by-products (CCBPs) as low-cost alternate aggregates and stabilizers in roadway and airfield construction. Specific objectives include the following:

1. Identify existing and developmental applications incorporating CCBPs in roadway construction.
2. Generate new and exploit existing utilization concepts that tend to enhance the potential for CCBPs as alternate aggregates, binders, and stabilizers.
3. Prepare preliminary specifications for any mixture design rationale incorporating CCBP materials generated in this study.
4. Assess environmental impact of CCBP materials and mixtures utilized in this study.
5. Recommend most promising concepts for field testing.

The report is divided into five sections:

- Section I: Introduction,
- Section II: The use of fly ash for soil stabilization,
- Section III: The use of by-product calcium sulfates and sulfites and hydrated fly ash in roadway construction,
- Section IV: The use of sulfur-modified bottom ash (SMBA) in asphalt concrete mixtures,
- Section V: Conclusions and recommendations, and
- Appendixes.

Each section contains its own respective literature review and relevant list of references. The researchers solicited the materials used in this study based on their availability in the area serviced by the Bryan District of TxDOT. However, the materials selection criteria, mix design rationale,

and construction procedures should apply to virtually all coal combustion by-products generated in Texas.

Results from field trials utilizing CCBP in roadbases indicate good to excellent performance characteristics and negligible environmental impact. Class 'C' and lime-enhanced Class 'F' fly ashes were shown to be effective stabilizers for both clay and sandy soils. Sulfate-resistant cements and Class 'C' fly ashes were shown to provide a wide range of strengths in stabilized sulfate and sulfite roadbases. When Class 'F' ashes are used, acceptable strengths can only be achieved with the addition of cement or lime. When the FGD materials and stabilizers are preblended prior to stockpiling, they must be utilized within 1 to 3 days so as not to be affected by any degradation of strength.

The use of sulfur improved crushing strength and reduced asphalt demand associated with bottom ash in asphaltic concrete mixtures. Sulfur modification permitted increased bottom ash utilization which, in turn, resulted in lower unit weight. In addition, sulfur modification required asphalt contents equal to and often less than for mixtures using conventional aggregates.

The successful use of hydrated fly ash as a base material in certain sectors of Texas is prompting other utilities to consider producing this product.

Based on its availability, lower cost, safe use and demonstrated performance, CCBP materials should find increasing use especially in those regions of Texas experiencing dwindling or non-existent aggregate sources.

# SECTION I

## INTRODUCTION

### BACKGROUND

#### Coal Combustion By-Product (CCBP) Production

Coal combustion by-products (CCBPs) are generated by energy production processes that use coal-fired boilers and various types of air pollution control equipment. Seventy-five million megagrams (Mg) of CCBPs are produced annually in the U.S. with about 12 million Mg generated in Texas. This makes Texas the largest producer of coal ash in the United States. Texas has seen a dramatic rise in the amount of coal ash produced in the last decade because of fuel supply diversification where low sulfur coals from outside the State or indigenous Texas lignite resources have been employed. The large quantities of coal ash production cited above rank it with municipal solid waste (MSW) as one of the largest by-product or "waste" streams on the national and state levels. For instance, CCBP production is about half as large as the national MSW stream (about 160 million Mg annually) and almost three-quarters the size of the Texas MSW stream (about 16 million Mg annually).

Coal combustion by-products exist in three primary forms: fly ash, bottom ash (which includes boiler slag), and flue gas desulfurization (FGD) material. Industry can use fly ash and bottom ash to make a variety of useful products, such as concrete for roads and buildings, bricks and other building products for housing, and fill materials for site development (1.1). The third by-product, FGD material, is produced by scrubbing SO<sub>x</sub>-bearing components from the flue gas. The FGD material is a synthetic gypsum, a calcium sulfate dihydrate (CaSO<sub>4</sub>•2H<sub>2</sub>O), which has been used in building construction for the production of wallboard, in agriculture for soil enhancement, in the production of cement, and for alternative road base materials (1.2).

Coal ash has many advantages. It is non-hazardous (1.3). It is abundant. It is economical. Its use saves energy, and there is a continuing, consistent supply. Figure 1-1 shows the locations of 18 coal-burning electric power generating plants in Texas. It is interesting to note that these locations coincide with regions in which the availability of quality aggregates (e.g., limestone, iron ore gravel, etc.) is either scarce, diminishing, or non-existent.

Unfortunately, relatively few CCBPs are finding their way into the marketplace. Although Texas is the largest generator of coal ash in the United States, less than 15 percent of the estimated 12 million Mg of coal ash generated annually in the state is currently being recycled. This compares with a 25 percent utilization rate nationally. The rest is disposed of in landfills. Table 1-1 details 1991 production and utilization of the three principal CCBPs (fly ash, bottom ash, and FGD material) and compares Texas and U.S. utilization rates.

**Table 1-1. 1991 Production and Utilization in Texas: Fly Ash, Bottom Ash, and FGD Materials.**

Coal Combustion By-Product	Produced (Mg x 1,000)	Disposed (Mg x 1,000)	Utilized (Mg x 1,000)	Texas Utilization Rate (%)	U.S. Utilization Rate (%)
Fly Ash	5,930	5,199	732	12.3%	25.8%
Bottom Ash	3,414	2,990	424	12.4%	44.4%
FGD Material	2,696	2,646	50	1.9%	1.9%
Total	12,040	10,835	1,206	10.0%	25.1%

Using coal ash saves enormous amounts of energy because it can replace many energy-intensive products such as cement, lime, and crushed stone. Each megagram of fly ash that replaces a megagram of cement, for example, saves the equivalent of nearly one barrel of imported oil. Using coal ash also saves the energy costs of mining and transporting materials such as clay, sand, limestone, and gravel. In addition, CCBP utilization conserves these virgin materials since coal ash can be directly substituted for conventional aggregates and/or cementation materials in many construction applications (1.4, 1.5, 1.6).

# COAL COMBUSTION BY-PRODUCT GENERATORS IN TEXAS

1-3

	UTILITY	(IN 1,000 TONS) FLY ASH PRODUCED	BOTTOM ASH PRODUCED	FGD MATERIAL PRODUCED	TOTAL PRODUCED	
1	COLETO CREEK	Central Power & Light *	96	24	0	120
2	LIMESTONE	Houston Lighting & Power	16	456	1,496	1,968
3	W. A. PARISH	Houston Lighting & Power	307	115	126	548
4	FAYETTE	Lower Colorado River Authority	207	104	42	354
5	J. K. SPRUCE	City Public Service of San Antonio	32	6	11	59
6	J. T. DEELY	City Public Service of San Antonio	114	41	0	156
7	SAN MIGUEL	San Miguel Electric	584	195	253	1,032
8	PIRKEY	Southwestern Electric Power Co.*	300	73	224	596
9	WELSH	Southwestern Electric Power Co.*	179	63	0	242
10	HARRINGTON	Southwestern Public Service	162	45	0	207
11	TOLK	Southwestern Public Service	154	62	0	216
12	GIBBONS CREEK	Texas Municipal Power Authority	455	419	172	1,045
13	BIG BROWN	Texas Utilities Electric	559	285	0	844
14	MARTIN LAKE	Texas Utilities Electric	1,099	567	480	2,146
15	MONITICELLO	Texas Utilities Electric	1,405	592	66	2,063
16	SANDOW	Texas Utilities Electric/Alcoa	524	191	140	856
17	TNP	Texas-New Mexico Power	291	114	0	405
18	OKLAUNION	West Texas Utilities*	105	38	24	166
	<b>TOTAL</b>	<b>All</b>	<b>6,590</b>	<b>3,390</b>	<b>3,033</b>	<b>13,013</b>

\* Subsidiaries of Central & South West Corporation

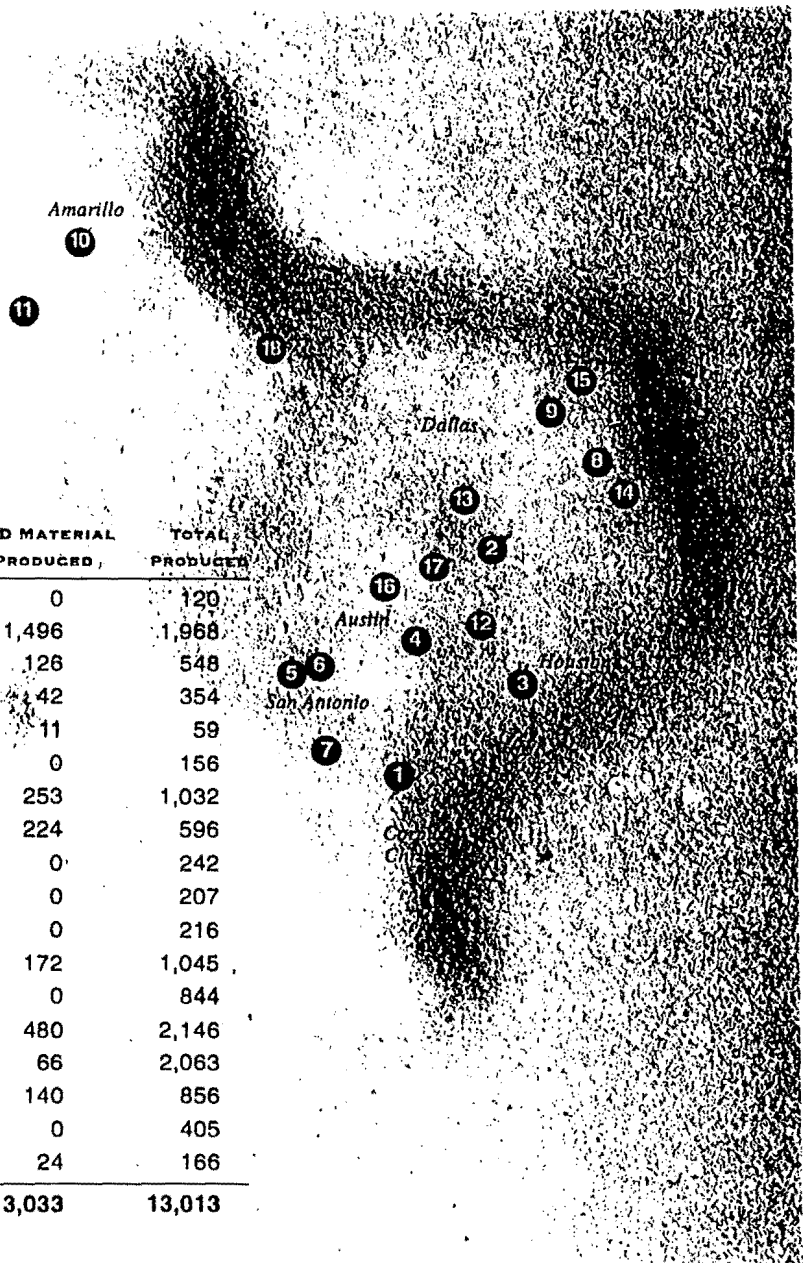


Figure 1-1. Coal Combustion Electric Power Generating Plants in Texas. (Courtesy of Texas Coal Ash Utilization Group)

Research and experience have shown CCBPs to be excellent, oftentimes superior, alternatives to traditional virgin construction materials at about half the cost. Additionally, CCBPs are non-hazardous. However, if these by-products are not utilized, they must be placed in a landfill. In the case of electric utility coal-fired plants in Texas, the utilities usually own these landfills, and they are typically adjacent to the plant. Since only 30 percent of all coal ash is recycled nationally and less than 15 percent is recycled in Texas, the increasing number of CCBP landfills are a major concern. The costs associated with constructing and operating these large landfills are included in the cost of electricity and are paid by utility customers. Increasing the utilization of CCBPs can minimize and, in some cases, eliminate the necessity for large, expensive CCBP landfills.

The amount of CCBP materials utilized in Texas remains small for several reasons, including regulatory constraints, utilization habits, and regulated trucking. In some cases, there may be a reluctance to use coal ash because it is relatively new or has a "waste" stigma. The principal constraints to increased utilization, until recently, were regulatory barriers, including state rules and regulations, that restricted or discouraged the recycling of materials otherwise regulated as waste. These rules resulted in coal combustion by-products retaining a "waste" stigma, even though they were being utilized in a variety of commercial applications.

### **Legislative Developments**

Several promising legislative developments have occurred in the last 5 years, which should improve the potential for CCBP utilization. A chronological summary of some of these legislative developments include:

- **SB 1340**

Texas SB 1340 was passed in 1991 to encourage recycling and the use of recycled/recyclable products, with the objective of minimizing the landfilling or incineration of solid wastes. One of the waste streams cited for minimization in this bill is "fossil fuel combustion recycled material" (Section 15) or, as it is more commonly known, "coal ash" or "coal combustion by-products." The bill required that state, county, and municipal entities amend their specifications (by January 1, 1992) to allow CCBP use in road and bridge construction if technically appropriate and economically cost-justified.

- **SB 1051**

Texas SB 1051 was passed in 1993 with the goal of reducing solid waste by creating markets for recycled materials and otherwise promoting recycling and the use of recycled materials. The bill established the Recycling Market Development Board, which consists of the heads of the General Land Office (GLO), Texas Natural Resources Conservation Commission (TNRCC), the General Services Commission, and the Texas Department of Commerce. The board is charged with pursuing an economic development strategy that focuses on development of recycling industries and markets. The board will identify existing economic and regulatory incentives and disincentives to recycling, analyze existing and potential markets for various recyclable materials including CCBPs, examine and make policy recommendations (e.g., transportation rates, tax incentives, grants), provide technical assistance to organizations regarding market opportunities in recycling, and assist in identifying state and federal grants pertaining to recycling and solid waste management.

- **EPA's Final Regulatory Determination for CCBPs**

The Environmental Protection Agency (EPA) issued a Final Regulatory Determination for CCBPs on August 2, 1993. The EPA concluded that these by-products should be permanently exempted from regulation as hazardous wastes under Resource Conservation and Recovery Act (RCRA) Subtitle C and be regulated as a non-hazardous material under Subtitle D. The EPA made this determination after more than a decade of study, culminating with the agency's 1988 *Report of Congress: Wastes from the Combustion of Coal by Electric Utility Power Plants*. In this report, the EPA officially "encourages utilization of coal combustion by-products and supports state efforts to promote utilization in an environmentally beneficial manner."

- **Texas Natural Resource Conservation Commission Regulations on CCBP's**

In July 1995, the TNRCC established coal combustion by-products, fly ash, bottom ash, and FGD materials approved for the specific uses given in Table 1-2. (1.3) TNRCC further stipulated that in the above applications, CCBPs can be considered co-products and not wastes provided their use is carried out "in a manner consistent with recognized local, state, and federal standards or general industry practice or standards."

**Table 1-2. TNRCC Approved Uses for CCBP Materials. (Reference 1.3)**

<b>CCBP USES</b>			
<b>APPLICATIONS</b>	<b>FLY ASH</b>	<b>BOTTOM ASH</b>	<b>FGD MATERIAL</b>
Concrete and Concrete Products	X	X	
Cement/Fly Ash Blends	X	X	
Raw Feed for Cement Manufacture	X	X	X
Precast Concrete Products	X	X	
Lightweight and Concrete Aggregate	X	X	
Roller Compacted Concrete	X	X	
Soil Cement	X	X	
Flowable Fill	X	X	
Oil Well Cementing	X		
Roadbase, Subbase and Subgrade Material When Covered By A Wear Surface	X	X	X
Road Construction Material (Unsurfaced)		X	
Masonry	X	X	X
Blasting Grit		X	
Roofing Material	X	X	
Insulation Material	X	X	
Wallboard/Sheetrock			X
Artificial Reefs	X	X	
Road Surface Traction Material		X	
Mineral Filler (e.g., plastics, paint, rubber matting, carpet backing, bricks, and asphalt)	X	X	
Waste Stabilization and Solidification	X		



## CCBP MATERIALS

CCBPs currently being generated by coal-burning electric power plants include:

1. Fly ash (Class 'C' and Class 'F'),
2. Bottom ash,
3. Boiler slag,
4. FGD sulfates and sulfites,
5. Elemental sulfur, and
6. Hydrated fly ash.

### **Fly Ash, Bottom Ash, and Boiler Slag**

Coal ash is a by-product of the coal burning, power-generation process. It constitutes the residual matter remaining after the coal combustion process. Coal ash can be divided into three classes: bottom ash, boiler slag, and fly ash. Bottom ash and slag are the coarser materials that fall to the bottom of the combustion chamber and are comprised primarily of porous, angular particles with a low specific gravity. Fly ash is the fine, lightweight residue that escapes the combustion chamber with the flue gases. Dust collection devices extract the fly ash from the gases and stockpile it until disposal. Fly ash can vary from 10 percent to 85 percent of the total coal ash residue depending on burner and boiler bottom types as well as the coal used during the process (1.1).

The American Society for Testing and Materials (ASTM) further classifies fly ash into two classes: 'F' and 'C' (1.7). Two-thirds of all fly ash produced is classified as Class 'F' fly ash. Class 'F' or *low-lime* fly ash results from the combustion of bituminous or anthracite coal. Class 'C' or *high-lime* fly ash is produced largely in the western United States and results from the combustion of lignite or sub-bituminous coal, which is prevalent in the West (1.4). A high percentage of calcium oxide contained in Class 'C' fly ash is responsible for the natural cementitious properties that this ash exhibits.

Bottom ash, boiler slag, and fly ash have together been described as the "fourth most plentiful mineral resource in the United States" (1.5). They are produced in 45 states and marketed in all states except Hawaii. The coal-burning electric power generating industry

has been producing coal by-products at an ever-increasing rate. As petroleum prices have soared in past years, utilities are being forced to convert power plants from gas and oil to coal because it is cheaper and more readily available. Although coal ash utilization has increased substantially in recent years, the production of coal has also risen. Coal ash (i.e., fly ash, bottom ash, and boiler slag) production in the United States reached 64 million Mg in 1991, from which 69 percent (or 44 million Mg) had to be disposed of in solid waste landfills (1.6). With increasing coal ash production, rising disposal costs and shrinking landfill space, new methods of ash utilization are needed to solve this problem.

### **Flue Gas Desulfurized Sulfates (Gypsum) and Sulfites (Gypsites)**

FGD gypsum is generally produced from coal-fired power plants by burning lignite or sulfur coals. Sulfur is a natural contaminant in some coals and is almost completely converted to sulfur oxides ( $\text{SO}_x$ ) when the coal is burned. Consequently, sulfur oxides ( $\text{SO}_x$ ), being released primarily in the form of sulfur dioxide, are converted by atmospheric processes to sulfates or sulfites and have been shown to contribute to the formation of acid rain. For this reason, the EPA has established stringent regulations for discharging sulfur dioxide emissions into the air.

In accordance with the 1977 Clean Air Act Amendments, the EPA established regulations that require coal-burning electric power companies to take steps to reduce  $\text{SO}_x$  emissions and particulates. The most prevalent means of  $\text{SO}_x$  control is FGD, which removes sulfur dioxide from exhaust gases before they are emitted into the air. The Clean Air Act Amendments of 1990 require coal burning plants to further reduce  $\text{SO}_x$  emissions by either installing costly scrubber systems or switching to low-sulfur coal. In the former, sulfur dioxide gases are washed with a limestone ( $\text{CaCO}_3$ ), calcium hydroxide ( $\text{Ca(OH)}_2$ ), or calcium oxide ( $\text{CaO}$ ), which acts as a chemical absorbent to produce calcium sulfate, usually of the dihydrate form, which is subsequently pumped to large settling basins.

Due to scaling problems in the scrubber and auxiliary piping, some FGD processes have been altered to use an additive like sodium thiosulfate to inhibit oxidation of sulfite to sulfate inside the scrubber. The FGD by-product of this process consists of 76 percent sulfite

and will be referred to in this report as Gypsite. This complex by-product consists of calcium sulfate dihydrate, calcium carbonate, and a solid solution of calcium sulfate and calcium sulfite hemihydrate. The Gypsite slurry is difficult to handle because of its thixotropic properties. In its by-product state, until recently, Gypsite has had little or no commercial value. Gypsite also requires a large storage capacity because it is difficult to dewater. Hence, as more utilities convert to thiosulfate FGD processes, the disposal of Gypsite in landfill sites is becoming increasingly expensive and unacceptable.

As the volume of FGD by-products increases, the cost of dedicating more and more land to accommodate these materials grows accordingly. Managing these vast storage areas to assure minimal environmental impact adds a cost that the electric power consumer ultimately must bear. Without suitable high-volume outlets for these by-products, these conditions can only be expected to worsen. One such outlet is in roadway construction, primarily as bases and subbases. Based on the field trials conducted by Texas Transportation Institute (TTI) from 1991 to 1993, cement and fly ash stabilized gypsum and Gypsite materials were used to construct a series of road base test sections. These sections are still performing well, and investigators noted little difference in their performance ([1.8](#), [1.9](#), [1.10](#)).

### **Elemental Sulfur**

Coal contains sulfur in two forms: mineral sulfur in the form of pyrite and organic sulfur that is chemically bound in the coal. Mechanical coal cleaning processes can remove most of the mineral sulfur. However, removing organic sulfur requires chemical processing by either microwave desulfurization or hydrothermal desulfurization.

During the energy crisis of the 1970s, sulfur was exploited for use in three types of roadbuilding applications: (a) as a means for upgrading marginal aggregates in base course construction, (b) as a partial substitute for asphalt cement in hot mix applications, and (c) as a binder for recycling age-hardened asphalt paving mixtures ([1.11](#)). Geopolitical events caused the price of sulfur to soar, and, hence, many of these developments were temporarily shelved. More recently, the price of sulfur has dropped to near 1970 levels, and its use in roadway construction is once again under consideration. In as much as their chemistries,

mineralogies, and geometrics put CCBP coal ashes in the category of marginal aggregates, the use of sulfur is being extended to upgrade their characteristics to maximize their use in both roadway and airport paving systems.

### **Hydrated Fly Ash**

Hydrated fly ash is produced by subjecting a Class 'C' ash to water in a ponded field. Upon hydration, the hardened material is dug up, crushed, and used as an alternative to crushed limestone in many types of construction applications.

A 1987 American Coal Ash Association (ACAA) symposium contained a number of papers describing traditional, as well as developing, uses of coal combustion by-products (1.6). One developing market is the manufacture of pelletized lightweight aggregate from hydrated coal fly ash by incorporating chemical admixtures and either lime (1.12) or portland cement (1.13) as activators, and from a third process using fly ash and coal cleaning mixtures (1.14). These lightweight aggregates are produced without the need for heat energy for sintering. However, another process uses heat to produce a sintered fly ash aggregate (1.15). Most of the current sources in Texas for hydrated fly ash are in the Panhandle area. However, new sources are now being developed in Central and South Texas.

The lightweight aggregates produced without sintering are used primarily in concrete block; however, some may be used in non-structural and structural concretes. A sub-committee of American Concrete Institute (ACI) (Committee 213) on lightweight aggregate and lightweight aggregate concrete was formed in February 1989 to collect and organize information for publication on these and various other lightweight by-product aggregates.

### **RESEARCH PROGRAM**

The purpose of this research project is to investigate the potential for using fly ash, bottom ash, and/or flue gas desulfurized sulfates and sulfites, hydrated fly ash, and sulfur as base or subbase layers in roadway and airfield pavement construction. If these materials prove to be effective in producing good base or subbase, they can reduce demand for conventional higher priced aggregates. These materials are lighter and cheaper than

conventional aggregates and tend to produce paving mixtures of low unit weight. Their use could save the state money and create a reliable, cost-effective way to utilize increasing inventories of CCBP materials.

### **Objectives**

The objective of this study is to exploit the beneficiation and utilization of CCBPs as low-cost alternate aggregates and stabilizers in roadway and airfield construction.

Specific objectives include the following:

1. Identify existing and developmental applications incorporating CCBPs in roadway construction.
2. Exploit existing technology and generate new utilization concepts to enhance the potential for CCBPs as alternate aggregates, binders and stabilizing agents. Some of the CCBP concepts included in this study are
  - a. Cement-stabilized FGD gypsum,
  - b. Fly ash-stabilized FGD gypsum,
  - c. Cement-stabilized FGD Gypsite,
  - d. Fly ash-stabilized FGD Gypsite,
  - e. Cement-stabilized bottom ash/gypsum,
  - f. Fly ash-stabilized bottom ash/Gypsite, and
  - g. Hydrated fly ash and lime-fly ash CCBP mixtures.
3. Prepare preliminary specifications for any mixture design rationale incorporating CCBP materials that this study generates.
4. Assess environmental impact of CCBP materials and mixtures utilized in this study.
5. Recommend most promising concepts for field testing.

## **Materials**

Researchers obtained the following CCBP materials from the sources listed below and used in the laboratory tests for this study:

<b>By-Product Material</b>	<b>Source</b>
Gypsum	TU-Electric/ALCOA (Rockdale Plant)
Gypsite	HL&P (Limestone Plant)
Fly Ash (Class 'C')	HL&P (Parish Plant)
Fly Ash (Class 'F')	HL&P (Limestone Plant)
Bottom Ash	TU-Electric/ALCOA (Rockdale) HL&P (Parish) HL&P (Big Brown)
Boiler Slag	TU-Electric/ALCOA (Rockdale)
Sulfur	Stauffer Chemical, Washington, D.C.

Appendix A lists company-furnished specifications for each of these materials.

## **Report Organization**

This report has been prepared in five sections.

Section I:	Introduction,
Section II:	The use of fly ash for soil stabilization,
Section III:	The use of by-product calcium sulfates and sulfites and hydrated fly ash in roadway construction,
Section IV:	The use of sulfur-modified bottom ash (SMBA) in asphalt concrete mixtures,
Section V:	Conclusions and recommendations, and
Appendixes.	

To minimize redundancy, each of the above sections contains its own list of references, figures, and tables numbered according to their respective sections. In addition, each section contains its own review of the literature and conclusions as it relates to the research activity contained therein.

Section V provides a more general set of findings along with recommendations for future work.





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## SECTION II

### THE USE OF FLY ASH FOR SOIL STABILIZATION

#### BACKGROUND

The concept of soil improvement or modification through stabilization with additives has been around for several thousand years. With the concern regarding shortage of conventional aggregates and the practice to consider energy demands of a project, transportation agencies are focusing attention on using substitute materials such as stabilized soils for highway construction (2.1). One of the most common methods of stabilizing soils is by the addition of lime. However, with the ever increasing demand for coal in the U.S. resulting in enormous amounts of fly ash being produced, the increasing cost and regulation of coal ash disposal, and the implementation of the "Beneficial Use of Coal Ash Act" by the Federal Highway Administration in 1987, greater interest has developed to find useful applications for fly ash.

Several studies have been conducted on stabilization of soils with fly ash and lime-fly ash (2.2, 2.3, 2.4). However, it is not possible to directly apply the results of one study to an area not included in the study. This is because the effectiveness of the stabilization depends on, among other factors, the characteristics of the soil (site specific) and the fly ash (type of coal, power plant, etc.). The subgrade soils in the Bryan District of the Texas Department of Transportation (TxDOT) are typically stabilized with type C quick lime of grade DS (2.5). With the availability of large quantities of fly ash close to the district, the focus of this study is on the effectiveness of fly ash as an alternative stabilizer for subgrade soils.

Two frequently encountered subgrade soils, chosen with the consent of the project director, Tom Parker, P.E., and two different fly ashes from power plants close to the district were included in this study. A description of the materials used is included in this report. Controlled, standard laboratory tests were performed on the soils stabilized with varying percentages of fly ash and lime-fly ash mixes. The control tests included unstabilized and soil samples stabilized with type C quick lime of grade DS. Standard tests performed included

sieve analysis, specific gravity, pH measurement, Atterberg Limits liquid limit and plastic limit, moisture-density relationship by the modified Proctor method, unconfined compressive strength, and one-dimensional expansion. This report includes details and the results of the laboratory tests. Based on the suggested design requirements for subgrade, mix recommendations have been made for pilot field sections. These field sections need to be monitored to evaluate long-term performance. A limited study of the influence of delays between mixing of the stabilizer and compaction has been done under controlled laboratory conditions. This study was done only for the fine sand since a minimum curing period of 2 days is required after the addition of stabilizers to clays. However, the actual effect of such delays could be more severe under field conditions. It is recommended that field trials include sections with such delays to obtain a better understanding of their influence.

## **LITERATURE REVIEW**

Because the focus of this portion of the study is on evaluating the effectiveness of fly ash as an alternative stabilizer for stabilizing subgrade soils in the Bryan District of TxDOT, it is imperative to understand the characteristics of the soil that needs to be stabilized, the stabilizer that is being used, and the mechanisms that produce stabilization. The following is a general overview of the characteristics of fly ash and soils, the mechanisms commonly involved in fly ash stabilization and some of the work done by other researchers.

### **Fly Ash**

Fly ash, produced from the combustion of coal, consists of the inorganic matter present in the coal that has been fused during coal combustion, solidified while suspended in the exhaust gases, and collected by electrostatic precipitators (2.6). The fly ash collected is typically stored in stockpiles until it is either used or disposed.

As discussed in Section I, the chemical composition of a fly ash depends on the coal source. The American Society for Testing and Materials (ASTM) (2.7) classifies fly ash into two classes: 'F' and 'C'. Class 'F' or "low lime" fly ash results from the combustion of bituminous or anthracite coal and has low calcium contents. Class 'C' or "high lime" fly ash

results from the combustion of sub-bituminous or lignite coal, which is prevalent in the West (2.8). Class F fly ash does not exhibit self-cementing characteristics. However, the addition of lime causes pozzolanic (long-term) reactions producing desirable cementitious products (2.6).

The hydration properties of a fly ash depend on a number of factors including coal source, boiler design and operation, and ash collection system. The hydration chemistry of the ash is governed by the amount and type of calcium compounds in the ash and the extent to which the calcium exists in crystalline form (2.6). The ash hydration chemistry is very complex and does not lend itself as a useful analytical tool for engineering applications. Hence, it is better to recognize the fact that ashes from different sources can have different hydration characteristics and rely primarily on the manifestations of these properties on the properties of the materials that are being stabilized (2.6).

## Soils

Grain-size distribution and particle shape significantly influence the engineering behavior of coarse-grained soils, which is not the case for fine-grained soils. Soil mineralogy and the presence of water play a significant role in the behavior of fine-grained soils. The method of stabilization adopted to stabilize a soil depends on the soil type and characteristics. Typically, mechanical stabilization is effective for cohesionless, coarse-grained soils such as sand and gravel. This may not be effective for cohesive, fine-grained soils such as clay, in which admixture stabilization may be necessary. During mechanical stabilization, the soil particles are rearranged such that the smaller particles fill in the voids created by the larger particles, and engineering properties are thereby enhanced. However, this is not effective in the case of poorly graded soils in which a significant proportion of the particles are of almost the same size. In such situations, admixture stabilization is more effective than mechanical stabilization.

Clay minerals are very small, colloidal-sized crystals (diameter less than  $1\mu\text{m}$ ) that are very active electrochemically. Chemically, clays are *hydrous aluminosilicates* plus other metallic ions. The different types of clay minerals are formed from only two fundamental

crystal sheets the *tetrahedral*, or silica, and the *octahedral*, or alumina sheets. The particular way in which these sheets are stacked, together with different bonding and different metallic ions in the crystal lattice, constitute the different clay minerals (2.8). Surface chemistry and the presence of water influence the behavior of clay minerals. This can be altered with the help of adding admixtures.

### **Fly Ash and Lime-Fly Ash Stabilization**

Fly ash in soil stabilization has been mainly used as a supplement or replacement for lime and cement in soils showing poor pozzolanic properties to enhance the lime-silica reaction (2.4). The stabilization of soils with the addition of either fly ash or lime-fly ash mixes is due to two basic sets of reactions: 1) Short term or “immediate” reactions and 2) long-term “pozzolanic” reactions (2.9). The immediate effect of the introduction of fly ash or lime-fly ash mix to the soil is to cause flocculation and agglomeration of the clay particles due to ion exchange at the surface of the soil particles (in coarse-grained soils, the fly ash plays the role of micro aggregate, and cation exchange at the surface of the soil particles is of no significance). These short-term reactions enhance workability and reduce the plasticity, swell, and shrinkage. The long-term reactions occur over a period of time (weeks, months, or even years), depending on the rate of breakdown and hydration of the alumino-silicates (pozzolans), and result in the formation of cementitious products. These cementitious products bind the soil particles together. Soils that are deficient in pozzolans will not react with the lime, and the fly ash, in such situations, provides a source of pozzolans. The extent and reaction rate are affected by the fineness of the soil, the chemical composition of both the fly ash and the soil, the temperature, the moisture content, and the amount of stabilizer used (2.11).

Ferguson and Zey (2.12) found that stabilization of clay soils for pavement subgrades with Class C fly ash could increase the shear strength, subgrade support capacity, and the swell potential. The California Bearing Ratio (CBR) increased from 2 or 3 for the untreated soil to 25 to 35 for the fly ash-stabilized soil (2.12). Lime stabilization increased the shear strength of clay soils but not to the extent achieved with the fly ash (CBRs of 4 to 8). Also,

the reduction in Atterberg Limits was observed to be generally less than that for lime stabilization due to the lower calcium ion concentrations. However, the swell potential was generally reduced to levels comparable to lime treatment (2.12).

Preliminary studies by Nicholson and Kashyap (2.9) and Nicholson et al. (2.10) have demonstrated the potential for stabilization and improvement of “poor to marginal” quality tropical (Hawaiian) soils with locally available fly ash. These studies reported improvements such as increase of unconfined compressive strength, more than a tenfold increase in CBR values, reduction in plasticity, reduction in swell, greatly increased workability, lower maximum dry densities, higher optimum moisture content, and less variation of dry density from the maximum over a much wider range of water contents. The addition of small percentages of lime to the fly ash is observed to have dramatic results due to the source of pozzolans provided by the fly ash.

Keshawarz and Dutta (2.4) studied the stabilization of two expansive South Texas Soils with a Class F fly ash and compared it with lime and portland cement stabilization. They found that fly ash, lime, and cement stabilization were, in general, effective in reducing the fraction of clay-size particles and the plasticity index. Also, after stabilization, the unconfined compressive strengths of all soil samples increased substantially. The stabilization with fly ash was more effective with one soil than the other, indicating that the degree of stabilization achieved depends on the soil.

Chu and Kao (2.13) performed studies on the stabilization of a low-plasticity clay with a low calcium oxide (7.5%) fly ash and a slag. The results indicated that with the addition of only the fly ash, the increase in strength is marginal. However, with the addition of the slag (39.3% CaO) to the fly ash, the increase in strength was significant due to the pozzolanic reactions between the fly ash and the lime provided by the slag.

Bredenkamp and Lytton (2.14) investigated the reduction of sulfate swell in expansive clay subgrades in the Dallas District of TxDOT. The addition of hydrated lime to stabilize high sulfate soils resulted in the formation of ettringite, a mineral that can expand up to 200% of its original size. Laboratory tests indicated that low-calcium fly ash stabilizers have the potential to reduce swell in high-sulfate soils and hence are proposed as alternative stabilizers.

Pachowski et al. (2.15) investigated the physical and mechanical characteristics of mixtures of cohesionless soils, sand or sand-gravel aggregate, and brown coal fly ash (with and without lime). The test results indicated that with the addition of fly ash, the compaction becomes more effective and there is a significant increase in the CBR values. The most favorable conditions for mechanical and chemical stabilization were observed for a mixture having 20% to 30% fly ash content at optimum water content as specified by the Modified Proctor Test and 30% to 40% for the Standard Proctor Test. The strength of soil-cement-fly ash mixtures was found to be significantly greater than solely soil-cement mixtures. However, the soil-cement-fly ash mixes required longer time to gain comparable strengths. Observations of experimental pavement sections constructed with fly ash stabilized soils and fly ash concrete base course indicated that all of them were in good condition and performed satisfactorily under traffic conditions.

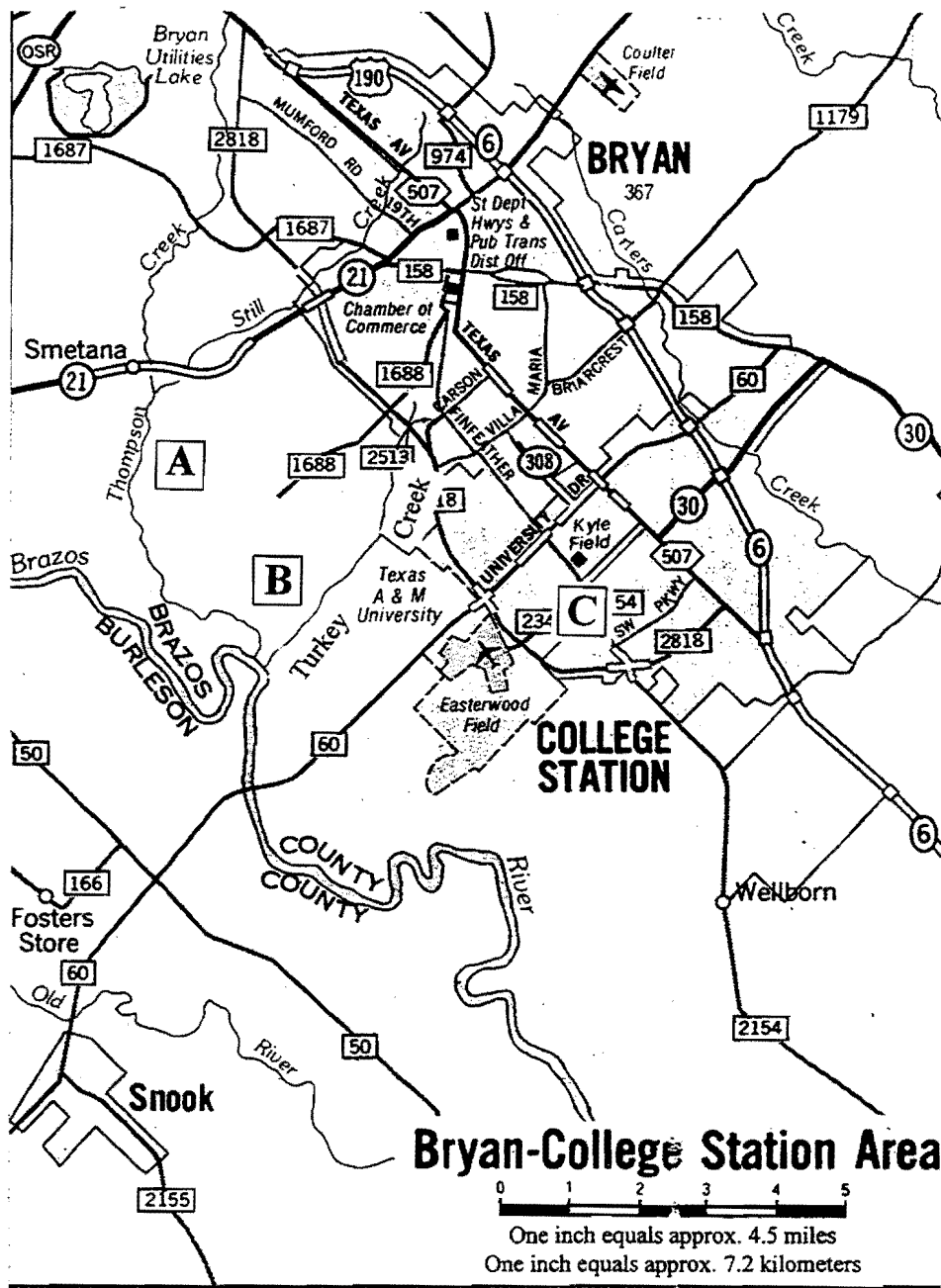
## **MATERIALS**

The following is a description of the materials used in the study and the tests performed on them. To study the effectiveness of fly ash in stabilizing subgrade soils commonly encountered in the Bryan District of TxDOT, the researchers made a decision to use two very different soils and fly ashes. The decision on the soils to be included in the study was made with the help of Mr. Tom Parker, P.E., the director of this research project. The fly ashes used were selected from two power plants located close to the Bryan District. Since the tests performed are standard, the descriptions are brief, and appropriate references and any deviations in procedure are included.

### **Soils**

The soils included in this study were obtained from sites near SH47 and the southwest corner of the intersection of Wellborn Road and George Bush Drive (Fig. 2-1.). Initially, two soils, both from sites on the SH 47 route one near Turkey Creek and the other near Thompson Creek were selected based on field investigation and experience. Preliminary field evaluation indicated that the soil near Turkey Creek was a highly plastic fine-grained soil, and the soil





**A - Thompson Creek**

**B - Turkey Creek**

**C - Southwest Corner of  
George Bush Drive and  
Wellborn Road**

Figure 2-1. Location of Soil Samples.

near Thompson Creek was a fine, cohesionless soil. Investigators transported soil samples to the laboratory, where they were dried and prepared for further analysis.

The cohesionless soil was reddish-brown in color, and the amount of material in the soil finer than the 75- $\mu\text{m}$  (No. 200) sieve (ASTM D 1140-92) was estimated to be 7.9% (2.16). The particle-size distribution was determined by subjecting 1.5 kg of the soil to a mechanical sieve analysis (ASTM D 422-63). Figure 2-2 shows the grain-size distribution curve for the soil. Based on the grain-size distribution and the percent fines, the soil was classified as a poorly graded sand. The soil classification is A-3 as per the American Association of State Highway and Transportation Officials (AASHTO) classification and SP as per the Unified Soil Classification System (USCS). The specific gravity of the soil (ASTM D 854-92) was estimated to be 2.638.

The cohesive soil was of reddish color and had streaks of white, calcareous material interspersed. The soil was broken down, and the fraction passing the 425- $\mu\text{m}$  (No.40) sieve was used to determine the Atterberg Limits (Liquid Limit and Plastic Limit) and calculate the Plasticity Index (PI) of the soil. The Atterberg Limits and PI were determined using ASTM D 4318-93. Figure 2-3 shows the flow curve for the soil from which the Liquid Limit is found to be 87.2%. The Plastic Limit (based on an average of three trials) was found to be 25.2%, and the PI was calculated to be 62%. Based on the Liquid Limit and the PI, the soil was classified as a highly plastic clay A-7-6 (as  $PI > LL - 30$ ) according to AASHTO and CH according to the USCS classification. The specific gravity of the soil (ASTM D 854-92) was estimated to be 2.766.

To determine the amount of lime necessary to reduce the PI of the A-7-6 soil from 62% to an acceptable value of 10-12% (communication with Tom Parker) and to raise the pH of the soil-lime system to 12.4 (necessary for pozzolanic reactions to occur), different percentages of the lime (see lime description below) provided by TxDOT were added, and the PIs and pHs were determined. The pH of the soil-lime system was determined by Tex-128-E (2.17). Figure 2-4 shows the PI and pH for lime contents varying from 6% (typically used) to 12%. It can be seen that at 12% lime the pH is close to 12.4 but the PI is 21%, which is unacceptable. Hence, based on the reason that a 12% lime requirement is

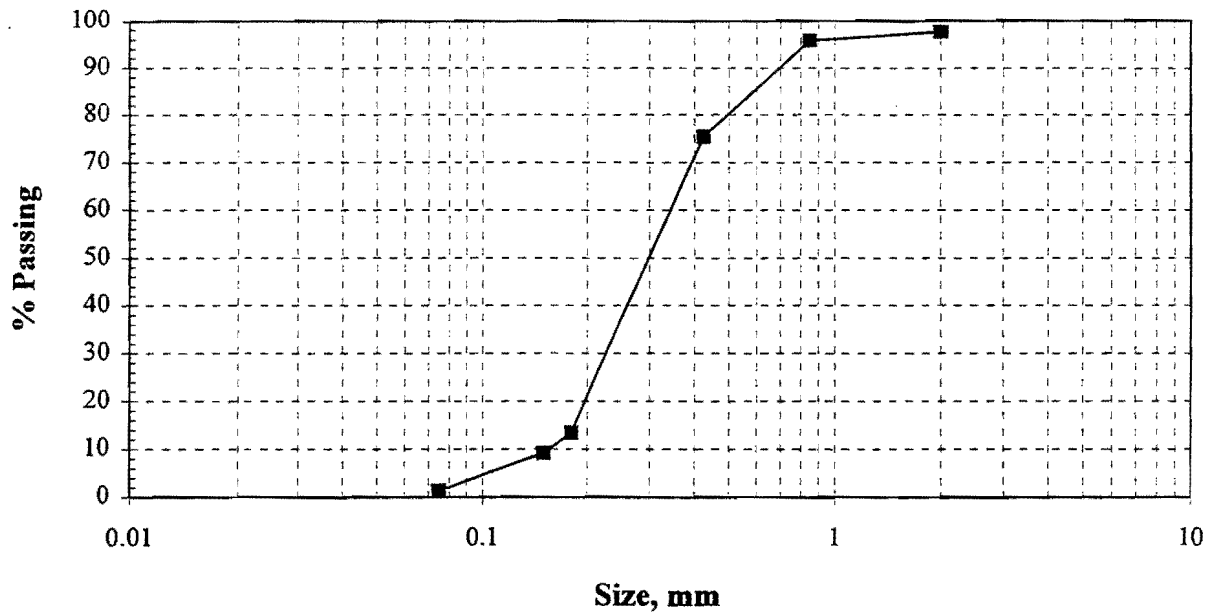


Figure 2-2. Grain-Size Distribution for Soil from Thompson Creek.

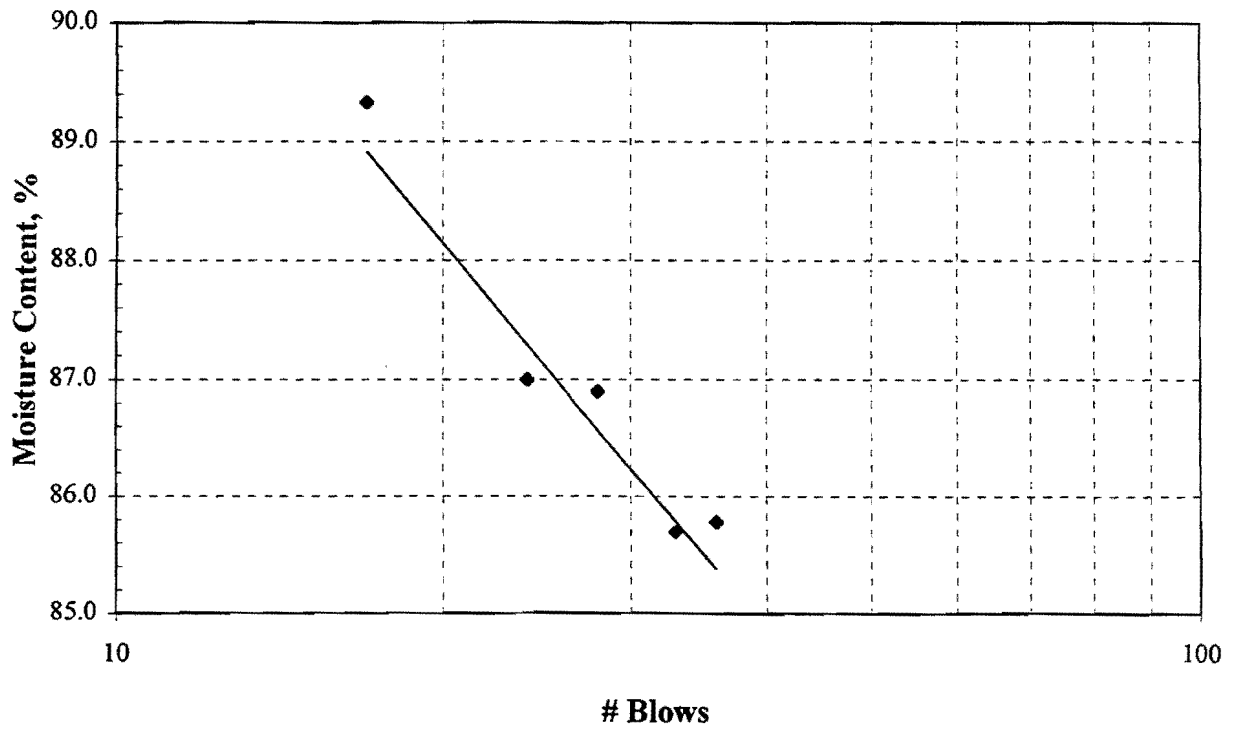


Figure 2-3. Flow Curve for Soil from Turkey Creek.

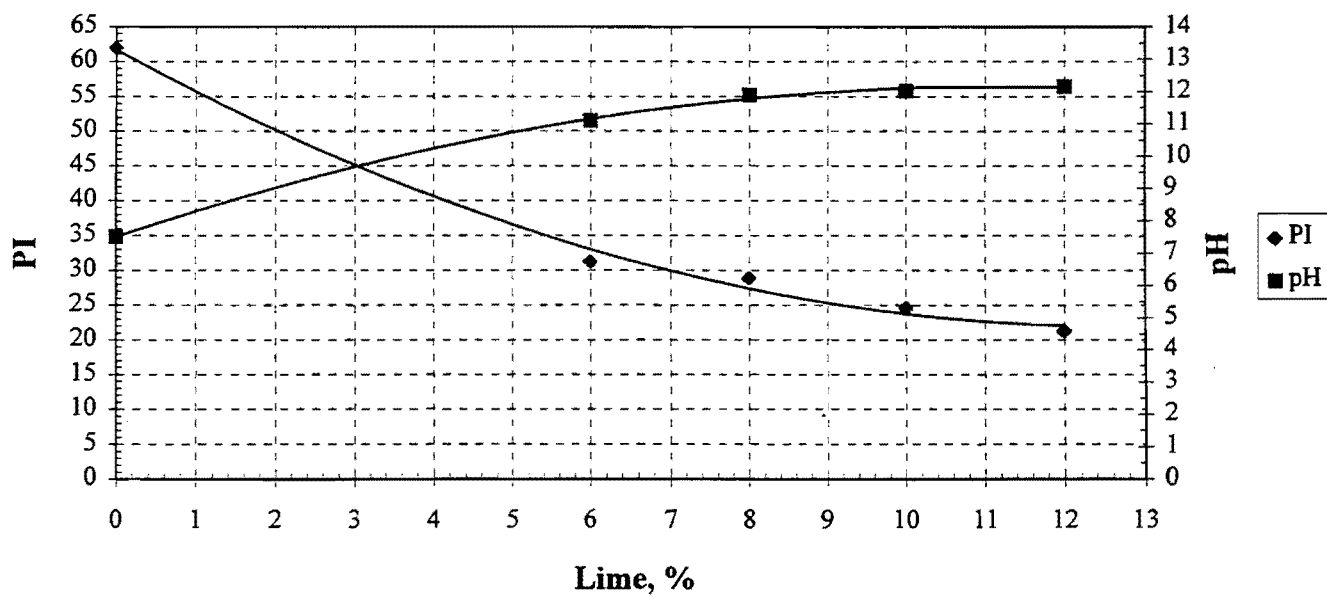


Figure 2-4. Influence of Lime on PI and pH of Turkey Creek Soil.

economically infeasible (2.5), the research project director and the members decided to stop further testing on this soil and select, in its place, another cohesive soil having a PI in the range of 25-30%.

The second cohesive soil was selected based on samples provided by TxDOT field engineers. The soil, located at the southwest corner of the intersection of Wellborn Road and George Bush Drive (Fig. 2-1), was grey in color and was found to have a Liquid Limit of 44.5% (Fig. 2-5), a Plastic Limit of 16.4 %, and a PI of 28.1%. This soil is classified as A-7-6 (as  $PI > LL-30$ ) according to AASHTO and CL (low plastic clay) according to the USCS classification.

### **Fly Ash**

The fly ashes used in this study were from two power plants of Houston Lighting & Power. One was a Class 'C' (high lime) fly ash from the W.A. Parish Plant in Fort Bend County, southwest of Houston and the other a Class 'F' (low lime) fly ash from the Limestone Plant located in Limestone County near Jewett, Texas. Table 2.1 shows the chemical and physical analyses of the fly ashes as provided by their manufacturers and the pertinent ASTM material specifications. The fly ashes were stored in air-tight containers until their usage.

### **Lime**

The lime used in this study was a Type C quick lime of Grade DS, which is currently being used by TxDOT for soil stabilization (2.5). A TxDOT contractor provided the lime in bags meeting the requirements of Item 264 (2.18). The lime was transferred to the laboratory and stored in dry form in air tight containers until immediately before use.

Table 2-1. Chemical and Physical Analyses of Fly Ashes.

<b>Chemical Analysis</b>				
	<b>W.A. Parish Class 'C' Fly Ash</b>	<b>Limestone Class 'F' Fly Ash</b>	<b>ASTM C-618-85 Specifications</b>	
			<b>Class 'C'</b>	<b>Class 'F'</b>
SiO <sub>2</sub>	33.63 %	57.03 %		
Al <sub>2</sub> O <sub>3</sub>	19.03 %	19.08 %		
Fe <sub>2</sub> O <sub>3</sub>	6.73 %	10.03 %		
Sum of SiO <sub>2</sub> + Al <sub>2</sub> O <sub>3</sub> + Fe <sub>2</sub> O <sub>3</sub>	59.39 %	86.14 %	50 % Min.	70 % Min.
MgO	4.82 %	1.89 %		
SO <sub>3</sub>	2.45 %	0.67 %	5.0 % Max.	5.0 % Max.
CaO	27.10 %	7.66 %		
Moisture Content	< 0.01 %	0.01 %	3.0 % Max.	3.0 % Max.
Loss on Ignition	0.25 %	0.01 %	6.0 % Max.	6.0 % Max.
Available Alkalis as Na <sub>2</sub> O	1.20 %	0.19 %	1.5 % Max.	1.5 % Max.
<b>Physical Analysis</b>				
Fineness (% Retained on #325 Sieve)	17.71 %	34.17 %	34 % Max.	34 % Max.
Water Requirement, % Control	94 %	95 %	105 % Max.	105 % Max.
Specific Gravity	2.63	2.38		
Autoclave Expansion, %	0.03 %	-0.03	0.8 % Max.	0.8 % Max.
Strength Activity Index w/ Portland Cement, 28 day	96 %	80 %	75 % Min.	75 % Min.

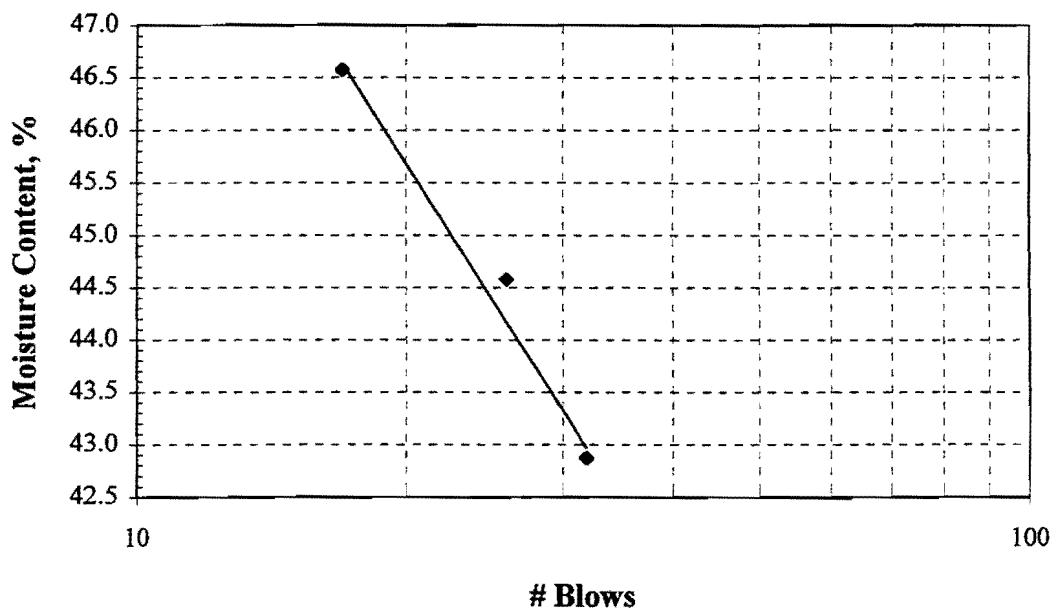


Figure 2-5. Flow Curve for Soil from Southwest Corner - George Bush Drive and Wellborn Road Intersection.

## TEST DESCRIPTIONS

The following tests, as appropriate, were run on the soil and soil-stabilizer samples:

1. Particle size analysis (ASTM D 422-63),
2. Material in soil finer than 75- $\mu\text{m}$  (No. 200) sieve (ASTM D 1140-92),
3. Specific gravity (ASTM D 854-92),
4. Liquid Limit, Plastic Limit, and Plasticity Index (ASTM D 4318-93),
5. Moisture content (ASTM D 2216-92),
6. Measurement of soil pH (Tex-128-E),
7. Moisture-density relationship (ASTM D 1557-91),
8. Unconfined compressive strength (ASTM D 2166-91), and
9. One-dimensional swell (ASTM D 4546-90).

The mechanical sieve analysis was performed on 1.5 kg of the oven-dried A-3 soil using a set of No. 10, 20, 40, 80, 100, and 200 sieves. The sieves were shaken, using a mechanical vibratory sieve shaker, for a total of 15 minutes prior to weighing mass of soil retained on each sieve. Other details of the test procedure are in ASTM D 422-63. The amount of soil finer than No. 200 sieve was determined by washing 500 grams of the soil with distilled water on the No. 200 sieve. The washings were collected, dried in a conventional oven, and finally weighed. Details of the test procedure are in ASTM D 1140-92. The specific gravity of the soil was determined using a specific gravity flask and 150 grams of oven-dried soil. The deairing was done using vacuum until air bubbles ceased to appear. Other details of the test procedure are in ASTM D 854-92.

The A-3 soil samples were molded immediately after the addition and mixing of the stabilizer and the required amount of moisture. A Humboldt mechanical mixer aided the mixing. A 3-hour delay between mixing the stabilizer and molding was induced only when the influence of such delays was being studied. Investigators molded the samples in a circular mold with an average inside diameter of 101.3 mm, height of 115.9 mm, and volume of 934 cm<sup>3</sup>. The molding was done using a mechanical sector face rammer with an



approximate mass of 4.54 kg falling through a height of 457 mm. The samples were molded in five approximately equal layers with each layer being subject to 25 blows. The details of the test procedure are in ASTM D 1557-91. Approximately 25-30 grams of the soil was used to determine the moisture content using a conventional oven. ASTM D 2216-92 outlines the details of the test procedure. After the samples were molded, they were trimmed, weighed, and extruded using a hydraulic jack. After determining the Optimum Moisture Content (OMC) from the moisture-density relationship curve, samples were molded at the OMC, extruded, and bagged immediately using two plastic bags and stored at room temperature during the entire period of curing. At the end of the curing period, the samples were removed from their bags, air dried for about 6 hours to remove any excess molding moisture, and finally subjected to an unconfined compression test. The unconfined compression test was done using a Tinius Olsen machine, and the maximum load per unit area was taken to be the unconfined compressive strength. Other details of the test procedure are in ASTM D 2166-91.

A hydrometer analysis (ASTM D 422-63) was attempted on the soil passing 75- $\mu\text{m}$  (No. 200) sieve. However, even after 36 hours there was no discernable difference in the hydrometer readings, indicating that most of the soil particles were still in suspension. Investigators discontinued further testing. For the highly plastic clay from Turkey Creek, the specific gravity (ASTM D 854-92) was determined using 50 grams of oven-dried soil. The fraction of soil passing the No. 40 (425- $\mu\text{m}$ ) sieve was used to determine the Liquid Limit, Plastic Limit and Plasticity Index. 250 grams of the soil was mixed with distilled water and admixtures (if any) into a creamy paste. When either lime or fly ash was added, a curing time of 48 hours was allowed for cation exchange to occur. For lime-fly ash mixes, the lime was added first and cured, and no additional curing was allowed after the addition of fly ash. This was done to simulate the construction specifications of TxDOT (Items 260.4 and 265.4). A minimum of three data points on the flow curve was used to determine the Liquid Limit, and the Plastic Limit was based on the average of three trials. The moisture contents were determined using about 10-15 grams of the soil and a conventional oven (ASTM D 2216-92).

For the plastic clay from Turkey Creek, the pH of the soil-lime system was determined using Tex-128-E. To 10 grams of the soil-lime mix in a test tube, 50 grams of deionized water was added, and the system was shaken for a period of 24 hours. At the end of 24 hours, the pH was determined by inserting the electrode of a pH meter that was calibrated using standard buffer solutions.

The cohesive soil from the southwest corner at the intersection of George Bush Drive and Wellborn Road had pebbles and concrete and asphalt chunks in it. These were removed, and the soil was broken down such that all of the soil passed through 4.75-mm (No. 4) sieve, and 75-80% passed the 425- $\mu$ m (No.40) sieve. The soil samples were mixed by hand after the addition of the admixture and the moisture. When either lime or fly ash was used, a curing time of 48 hours was allowed prior to molding to simulate TxDOT construction specifications. When lime-fly ash mix was used, the lime was added first and allowed to cure for 48 hours. Thereafter, fly ash was added and mixed, and the samples were molded without additional curing time. The samples were molded in the same molds used for the A-3 soil with the same mechanical rammer and molding procedure (ASTM D 1557-91). After determining the OMC, samples were molded at the OMC and double bagged for curing. At the end of the curing period, the samples were subjected to an unconfined compression test as described before.

The cohesive soil samples were also subjected to a one-dimensional expansion test using a consolidometer. The soil samples were molded at OMC as described before and then cored and trimmed to get samples of 50 mm diameter and 13.3 mm height. The samples were transferred to the consolidometer apparatus and mounted on the test frame. The samples were inundated with distilled water and the displacements with time were recorded using a transducer connected to a channel reader and a printer interface/printer. The first 30-minute transducer readings were recorded by hand; thereafter, transducer readings were recorded at 1 hour intervals by the printer. The test continued until very little change in transducer readings was recorded. Other details of the test can be found in ASTM D 4546-90.

## MOISTURE-DENSITY RELATIONSHIP FOR A-3 SOIL

Appendix B shows the moisture-density curves for the A-3 soil without admixtures and the A-3 soil with 10%, 15%, 20%, and 25% Class 'C' fly ash. Figure 2-6 shows the influence of different percentages of Class 'C' fly ash on the maximum dry density achieved. It can be seen that the maximum dry density achieved increases with increasing percent of fly ash. However, the magnitude of increase decreases with increasing fly ash content. Since the soil is a poorly graded sand, the sand particles are of almost the same size, and the addition of fly ash causes the voids between sand particles to fill up. This leads to an increase in the density. Thus, fly ash plays the role of a micro aggregate. However, with increasing fly ash content, the amount of fly ash can be in excess of that required to fill the voids and thereby cause a reduction in the density increase.

Figure 2-7 shows the influence of Class 'C' fly ash on the OMC. With increasing fly ash content, the OMC initially decreases up to a fly ash content of 20% and then increases.

Currently, given that the cost of fly ash is about one-quarter that of lime and that the existing practice is to add 5% quick lime (communication with Tom Parker), the researchers decided to limit the fly ash content to 20%.

Figure 2-8 shows the influence of lime and Class 'F' fly ash on the maximum dry density achieved. It can be seen that the maximum dry density achieved increases with the amount of fly ash, which can once again be attributed to the filling up of voids in the sand by the fly ash.

Figure 2-9 shows the influence of lime and Class 'F' fly ash on the OMC. The optimum moisture content is seen to decrease with the addition of lime and/or fly ash with the lowest OMC being at 20% fly ash content.

From Figures 2-6 and 2-8, it can be seen that there is very little (< 2%) difference in the maximum dry densities achieved by the addition of only lime to the native soil (i.e., no admixtures). However, the OMC has been reduced by about 3% due to the addition of lime.

Figures 2-10 and 2-11 show the influence of a 3-hour delay between mixing the stabilizer and molding on the moisture-density relationships of 15% and 20% fly ash contents. It can be seen that the maximum dry density achieved is not affected, but the OMC

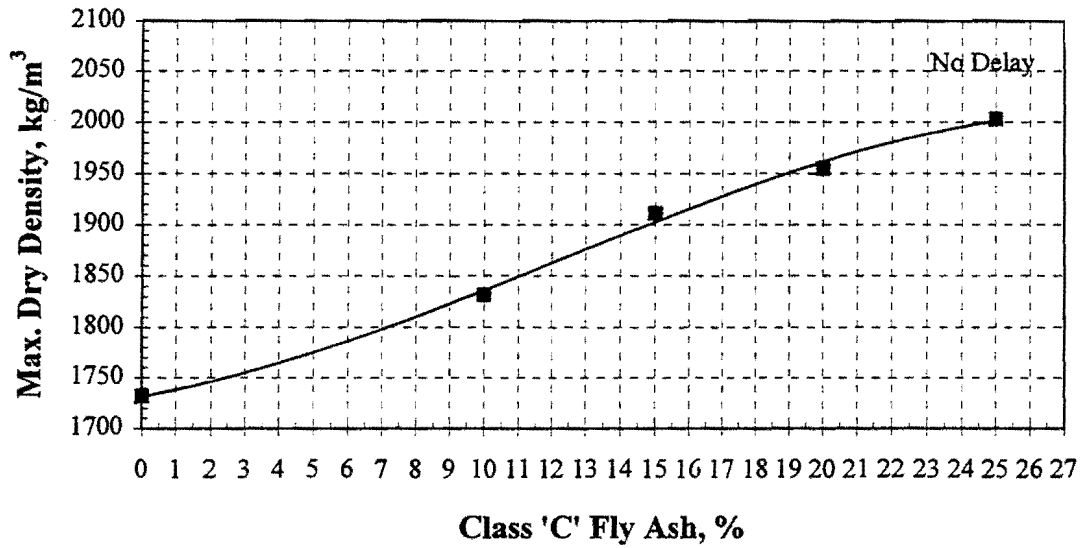


Figure 2-6. Influence of Class 'C' Fly Ash on Maximum Dry Density of A-3 Soil.

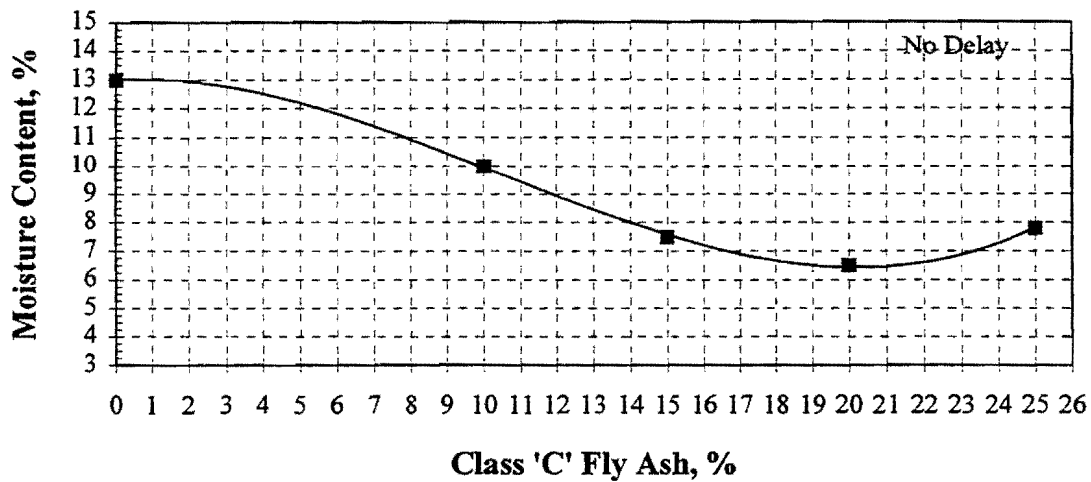


Figure 2-7. Influence of Class 'C' Fly Ash on Optimum Moisture Content of A-3 Soil.

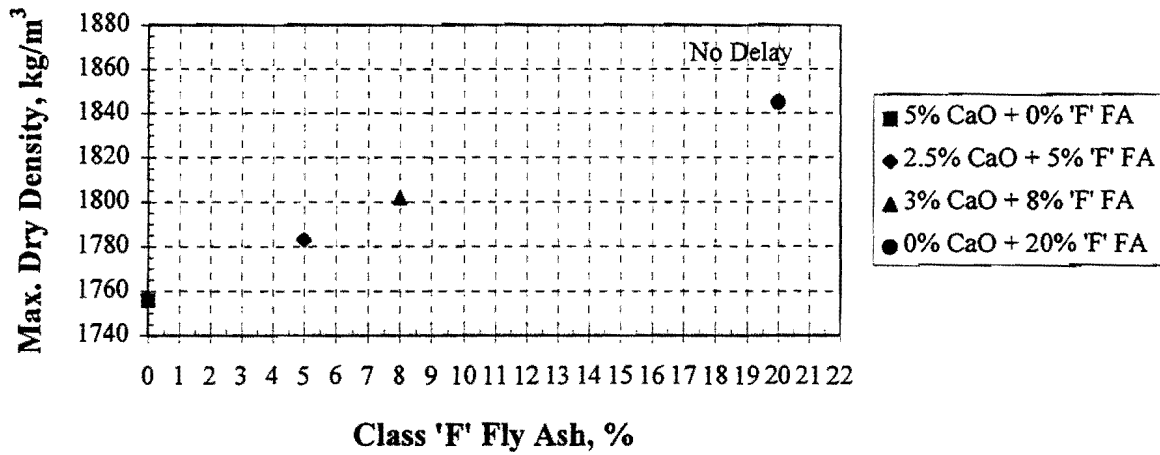


Figure 2-8. Influence of Lime and Class 'F' Fly Ash on Maximum Dry Density of A-3 Soil.

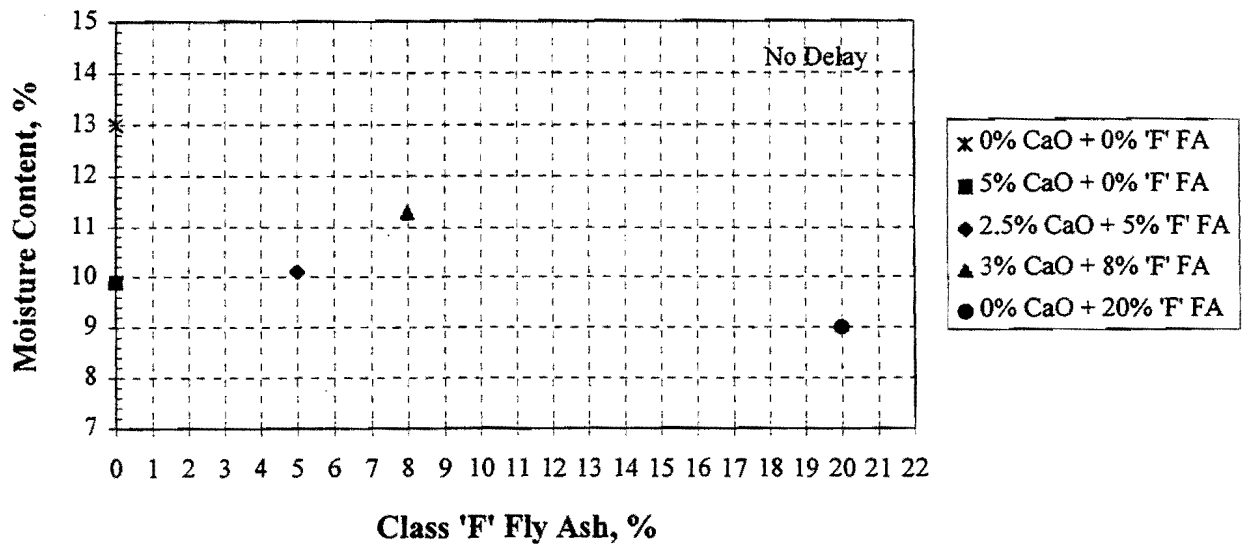


Figure 2-9. Influence of Lime and Class 'F' Fly Ash on Optimum Moisture Content of A-3 Soil.

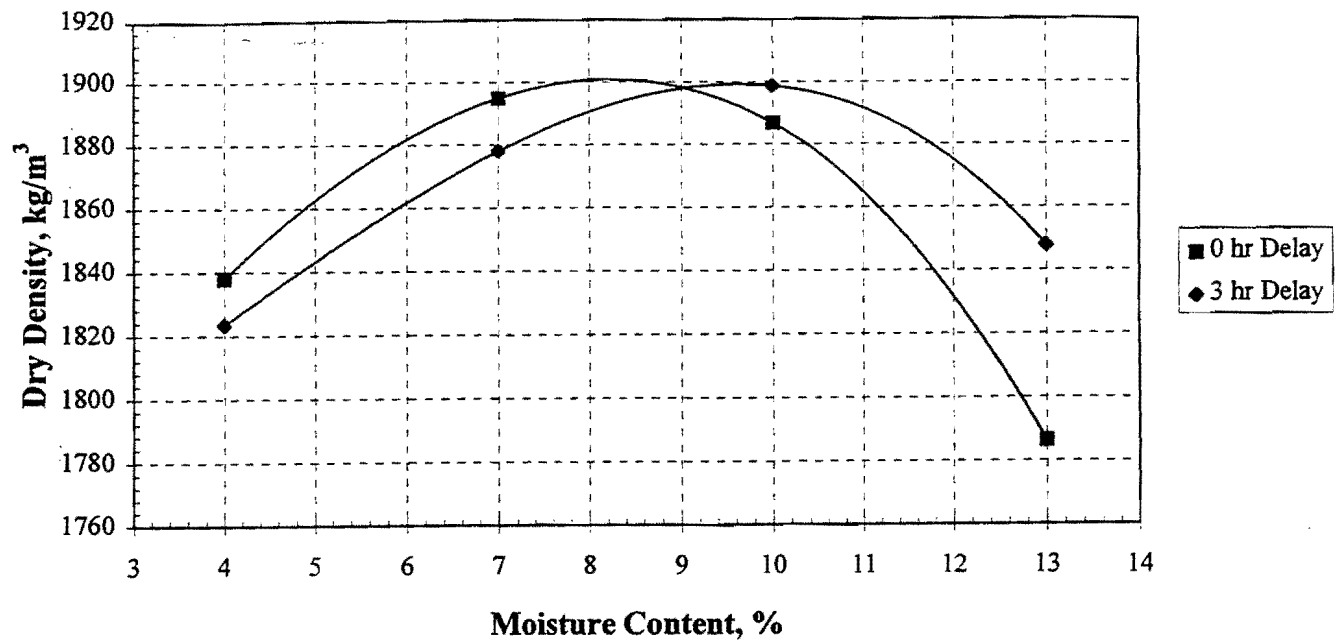


Figure 2-10. Influence of Delay on Moisture-Density Relationship of A-3 Soil with 15% Class 'C' Fly Ash.

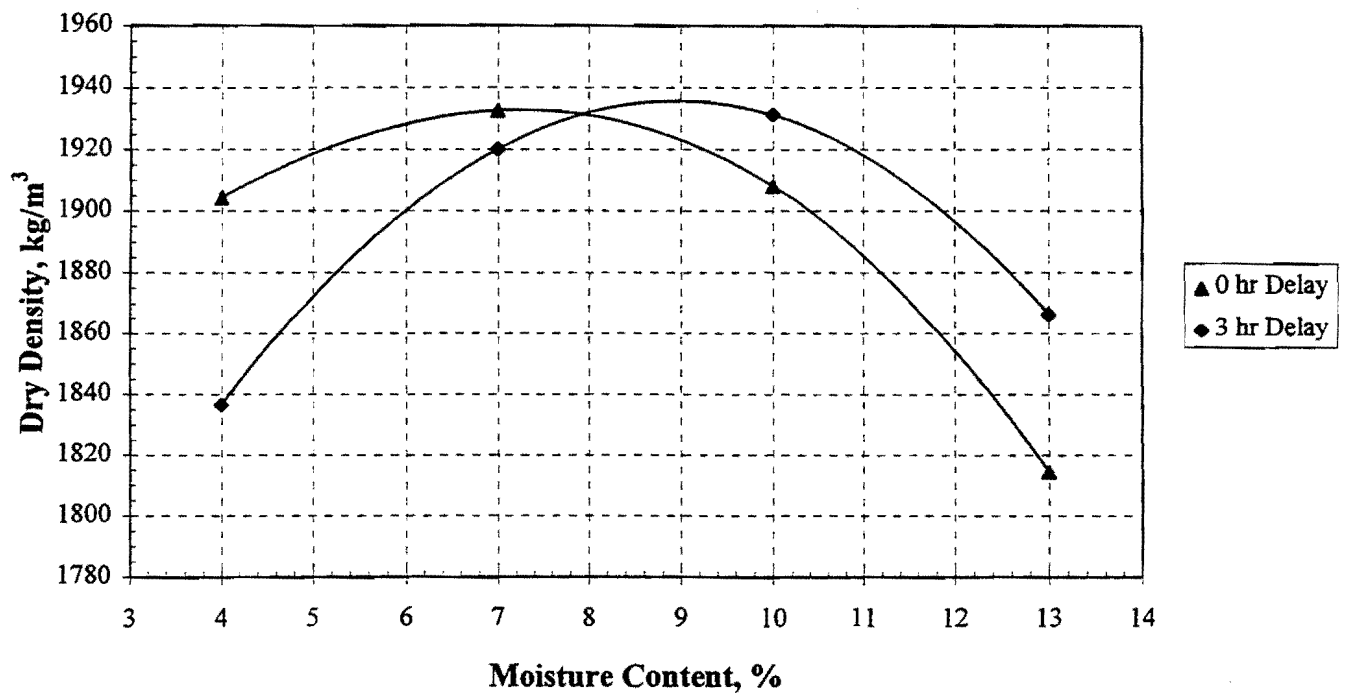


Figure 2-11. Influence of Delay on Moisture-Density Relationship of A-3 Soil with 20% Class 'C' Fly Ash.

increases by about 2% due to the delay. The lack of any significant influence on the maximum dry density can be attributed to the slow hydration rate of fly ash.

Based on the requirement that the maximum dry density achieved should at least be 1800 kg/m<sup>3</sup> and that a fly ash requirement of 25% was economically infeasible (2.5), further testing was limited to mixes that met these requirements (i.e., 10%, 15%, and 20% Class 'C' fly ash, 3% lime + 8% Class 'F' fly ash, and 20% Class 'F' fly ash).

### **UNCONFINED COMPRESSIVE STRENGTH FOR A-3 SOIL**

The investigators could not determine the unconfined compressive strength of the native A-3 soil because the demolded samples fell apart.

Figure 2-12 shows the influence of Class 'C' fly ash on the unconfined compressive strength of the A-3 soil. It can be seen that the unconfined compressive strength increases with the increase in Class 'C' fly ash content and the gain in strength decreases with time.

Figure 2-13 shows the 7-day unconfined compressive strength for lime-fly ash mixes. It can be seen that adding lime to low lime fly ash significantly enhances the strength. Merely adding 5% lime, which is the state-of-practice (2.5), does not significantly improve either the density or the strength. The addition of only low-lime fly ash is seen to increase the maximum dry density but not the unconfined compressive strength.

In Figure 2-12 it can be seen that the compressive strength of 3% lime + 8% Class 'F' fly ash mix is higher than that achieved by adding only 10% Class 'C' fly ash. This highlights the benefit of adding lime to low lime fly ash to enhance the pozzolanic reactions.

Figures 2-14 and 2-15 show the influence of a delay between mixing the stabilizer and molding of the samples on the unconfined compressive strength. The unconfined compressive strength is seen to drop by about 15% to 20% due to the delay, and the effect is greater for the larger fly ash content. However, the delays in the laboratory were at room temperature and under controlled exposure conditions. This is not the case for delays in the field, where conditions are uncontrolled, and therefore the influence can be significant.

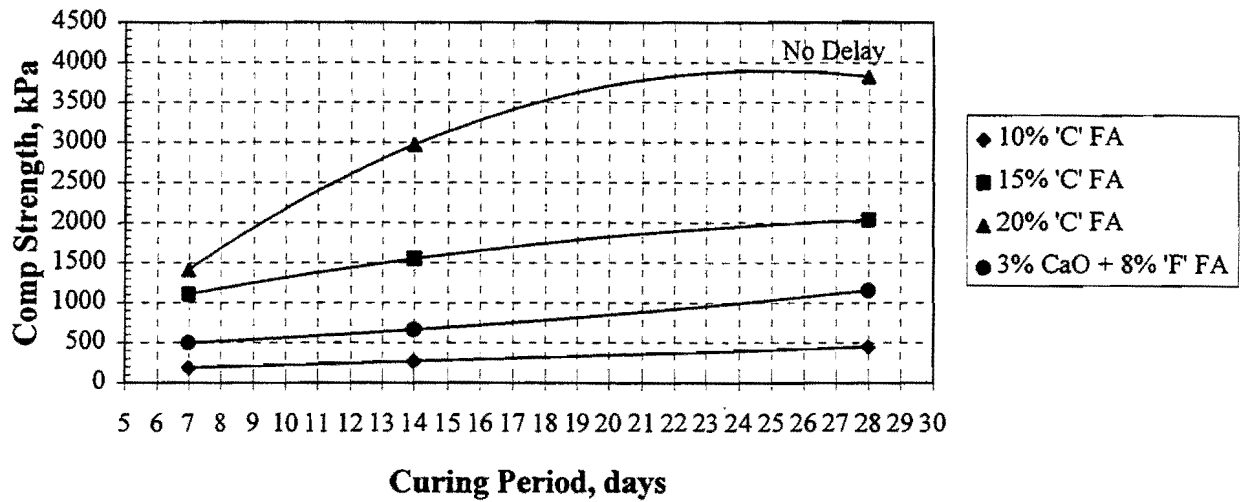


Figure 2-12. Unconfined Compressive Strength at Optimum Moisture Content for A-3 Soil.

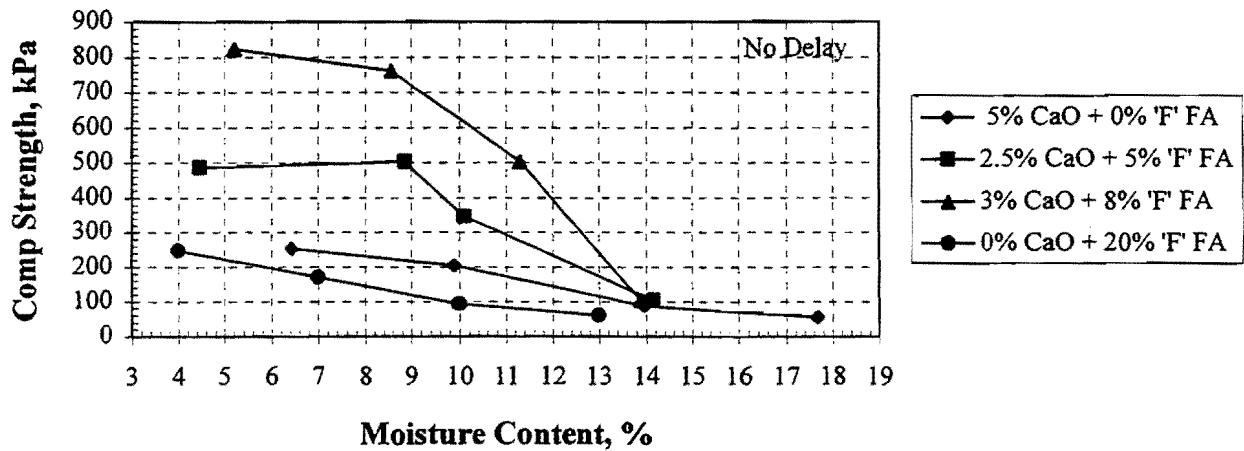


Figure 2-13. 7-Day Unconfined Compressive Strength for A-3 Soil with Lime and Class 'F' Fly Ash.



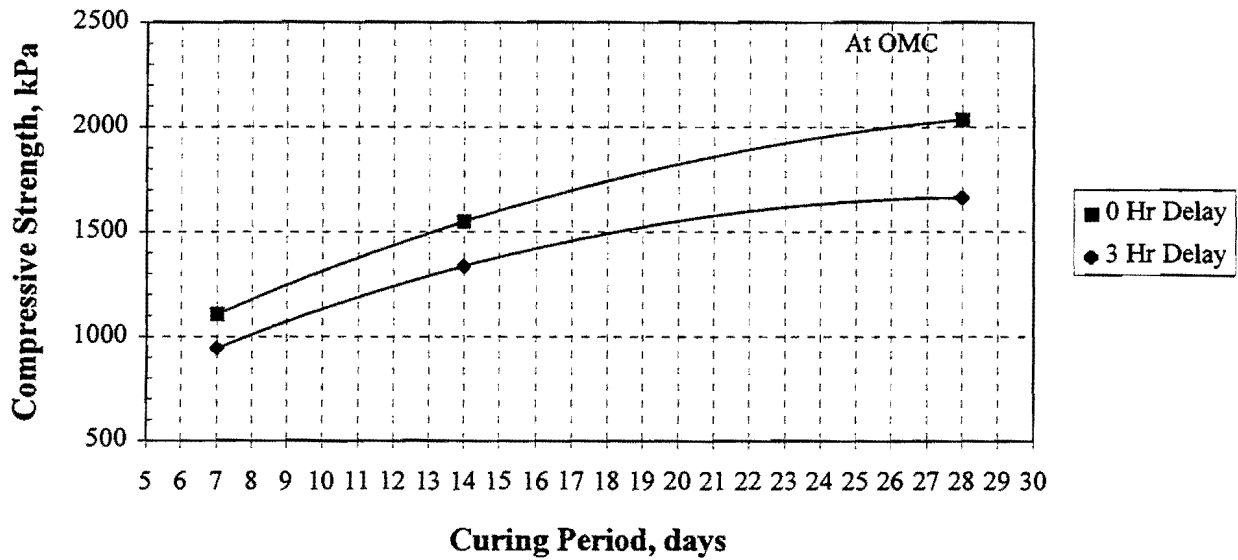


Figure 2-14. Influence of Delay on Unconfined Compressive Strength of A-3 Soil with 15% Class 'C' Fly Ash.

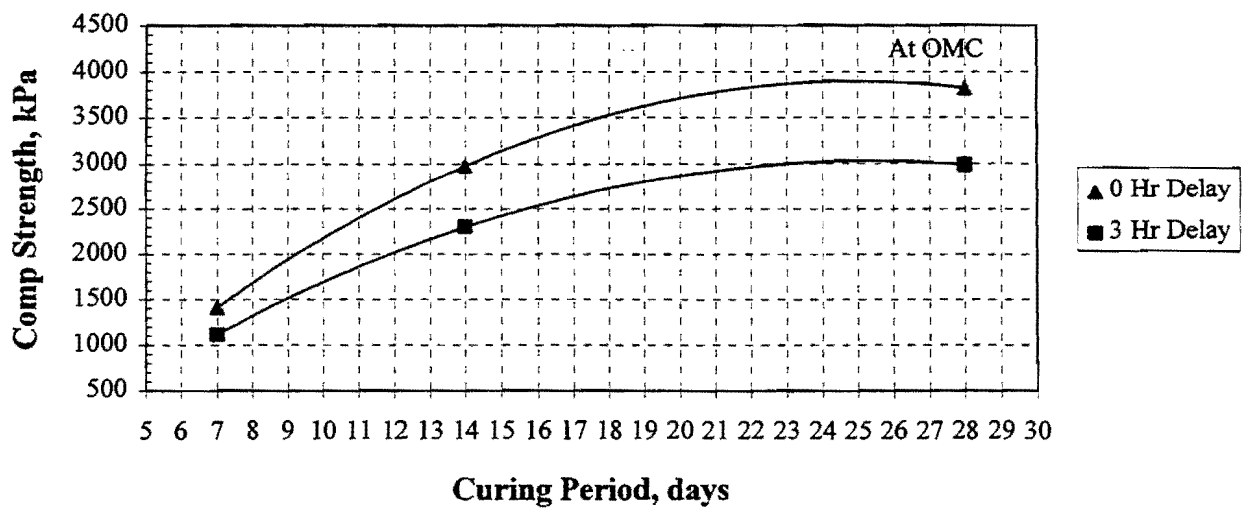


Figure 2-15. Influence of Delay on Unconfined Compressive Strength of A-3 Soil with 20% Class 'C' Fly Ash.

### **ATTERBERG LIMITS FOR A-7-6 SOIL**

Figure 2-16 shows the influence of lime and/or fly ash on the PI of the A-7-6 soil. The addition of 4% lime (2.5) is found to be the most effective in reducing the PI to 11.4%. A comparable reduction (PI of 12.31%) is observed with the addition of 20% Class 'C' fly ash. The PIs for addition of 15% Class 'C' fly ash and 2% lime + 5% Class 'F' fly ash are comparable (i.e., 15.43% and 15.41%, respectively).

### **MOISTURE-DENSITY RELATIONSHIP FOR A-7-6 SOIL**

Figure 2-17 shows the influence of lime and/or fly ash on the maximum dry density. It can be seen that with the addition of lime and/or fly ash, the maximum dry density decreases even though the workability was observed to increase. The least dry density is achieved with the addition of 4% quick lime. With the addition of 10% Class 'C' fly ash, the maximum dry density is the highest and is less for greater fly ash contents. The workability of the soil was observed to increase with increasing fly ash content.

Figure 2-18 shows the influence of lime and/or fly ash on the OMC. The addition of quick lime is seen to increase the OMC by 3 to 4%. With the addition of 10% Class 'C' fly ash, the OMC is seen to increase by only a small percentage. In general, the OMC increased with the addition of lime and/or fly ash, with the increase being larger when quick lime is added.

### **UNCONFINED COMPRESSIVE STRENGTH FOR A-7-6 SOIL**

Figure 2-19 shows the influence of lime and/or fly ash on the unconfined compressive strength. The maximum strength is seen to be obtained with the addition of 20% Class 'C' fly ash that is followed by the addition of 4% quick lime. The strength is seen to increase with increasing Class 'C' fly ash content. In general, the addition of lime and/or fly ash has increased the strength, with the smallest increase being for the 2% quick lime and 5% Class 'F' fly ash mix.

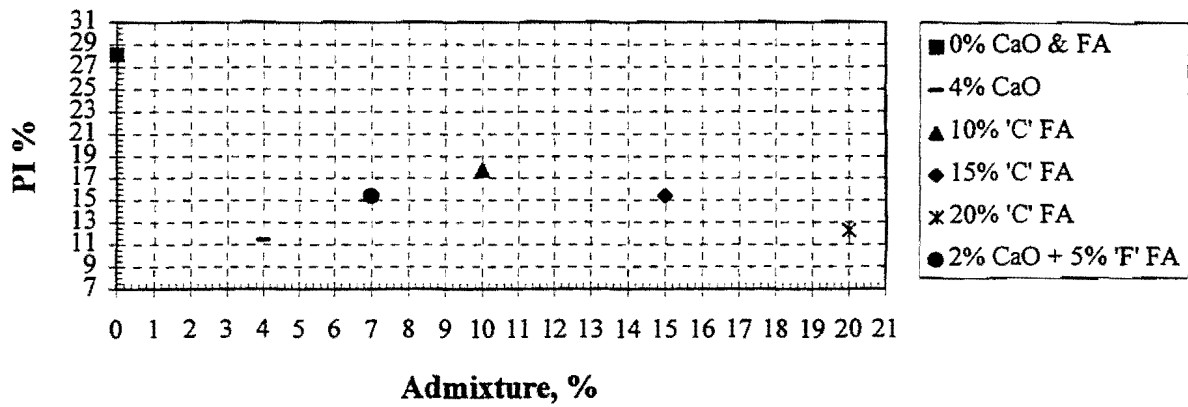


Figure 2-16. Influence of Lime and Fly Ash on the Plasticity Index of A-7-6 Soil.

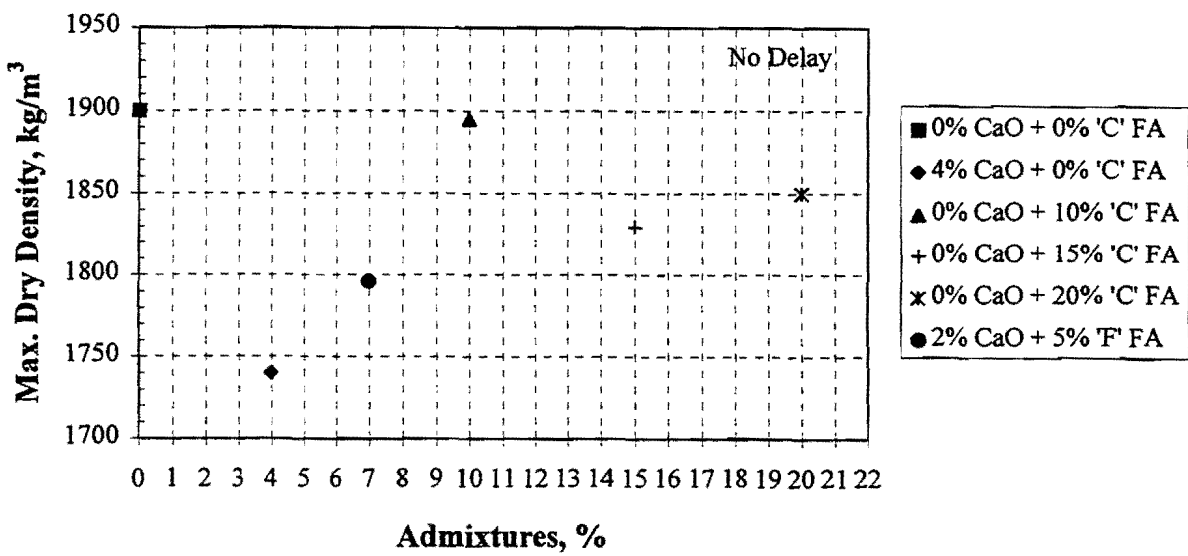


Figure 2-17. Influence of Lime and Fly Ash on Maximum Dry Density of A-7-6 Soil.

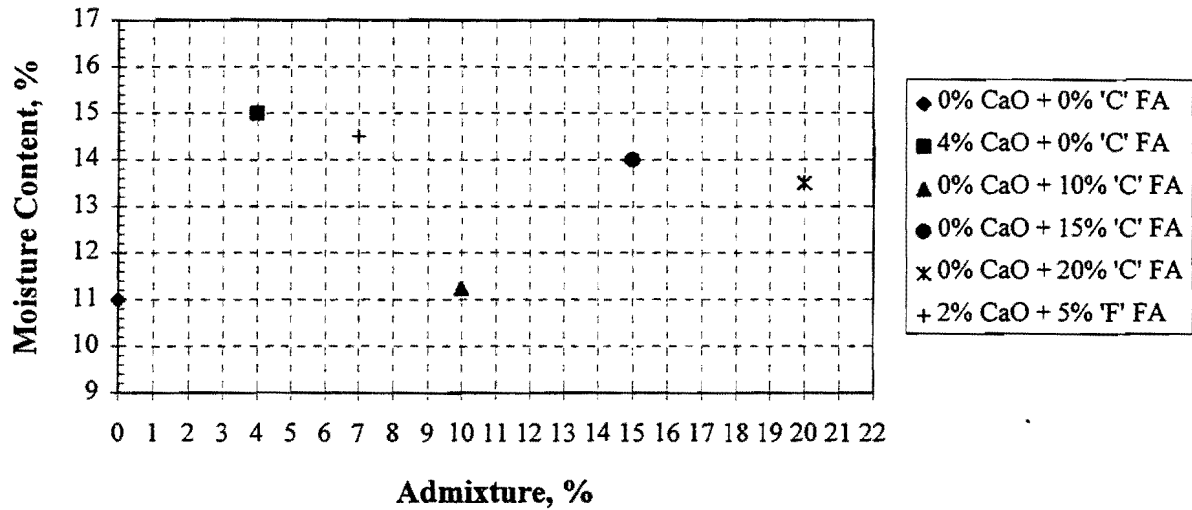


Figure 2-18. Influence of Lime and Fly Ash on Optimum Moisture Content of A-7-6 Soil.

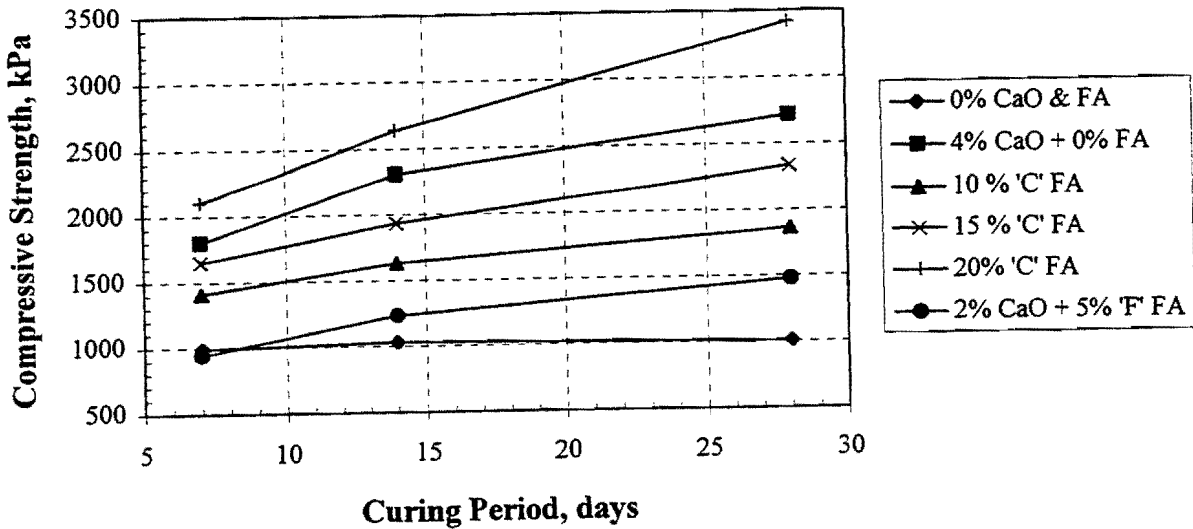


Figure 2-19. Influence of Lime and Fly Ash on Unconfined Compressive Strength of A-7-6 Soil.

## **ONE-DIMENSIONAL EXPANSION FOR A-7-6 SOIL**

Figure 2-20 shows the influence of lime and/or fly ash on the swell of the A-7-6 soil. The swell was calculated based on the change in sample height. In general the swell is seen to be reduced by 70% to almost 100% with the addition of lime and/or fly ash. It can be seen that the maximum reduction in swell is achieved by the addition of either 2% quick lime and 5% Class 'F' fly ash, 4% quick lime, or 10% Class 'C' fly ash. The reduction in swell with the addition of fly ash can be attributed to the formation of cementitious bonds due to pozzolanic reactions. For Class 'C' fly ash, the reduction in swell is seen to decrease with increasing fly ash content. Figures B-20 through B-25 of Appendix B show the individual swell curves.

## **SUMMARY OF FINDINGS AND RECOMMENDATIONS**

The overall objective of this study was to evaluate the effectiveness of Coal Combustion By Products in stabilizing pavement subgrade soils encountered in the Bryan District of TxDOT. Several studies have been carried out in the past to indicate that fly ash, in general, can be an effective stabilizer. However, the extent of stabilization achieved depends on the source and type of fly ash and the characteristics of the soil that needs to be stabilized. To evaluate the effectiveness of fly ash as a stabilizer, two very different subgrade soils that are commonly encountered in the Bryan District A-3 and A-7-6 were chosen for the study. The two fly ashes Class 'C' and Class 'F' were chosen from power plants close to the District. The over all findings of this study are as follows:

- The Class 'C' fly ash is a more effective stabilizer than the Class 'F' fly ash.
- Class 'F' lime- fly ash mixes are more effective stabilizers than only Class 'F' fly ash.
- The moisture requirements of lime and lime-fly ash mixes are greater than that of fly ash mixes alone.

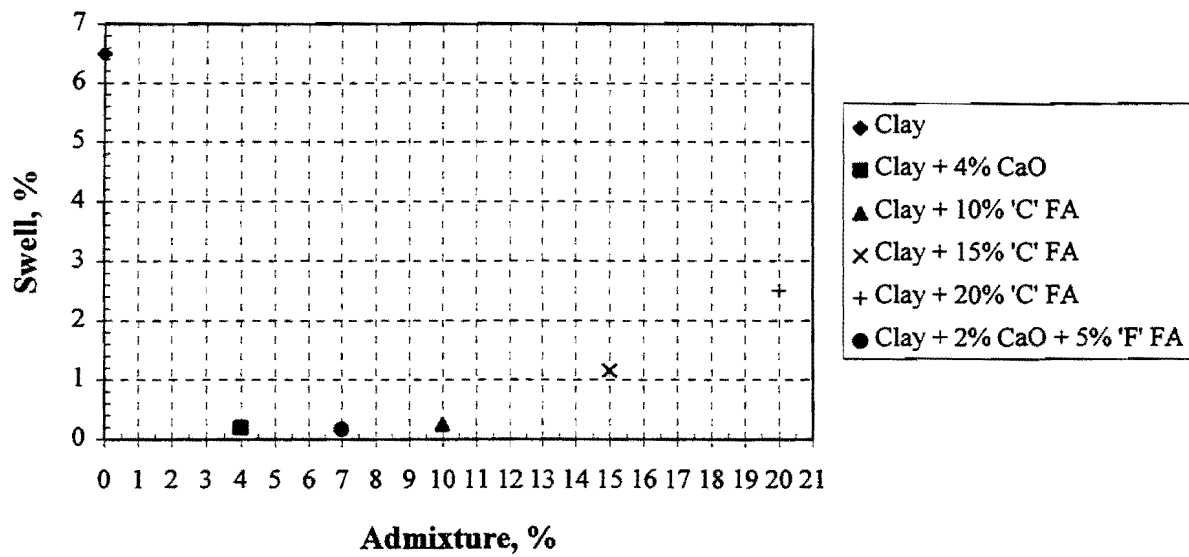


Figure 2-20. Influence of Admixtures on Swell of A-7-6 Soil.

- For the poorly graded fine sand (A-3), Class 'C' fly ash or lime-fly ash mixes are more effective stabilizers than lime alone.
- The maximum dry density for the A-3 soil increases with the addition of fly ash.
- The optimum moisture content for the A-3 soil decreases with the addition of lime and/or fly ash.
- The unconfined compressive strength of the A-3 soil increases with increasing Class 'C' fly ash content.
- The unconfined compressive strength of the A-3 soil is greater with quick lime-Class 'F' fly ash mixes than with either quick lime or Class 'F' fly ash only.
- For the A-3 soil, a delay between mixing the stabilizer and molding has a greater influence on the unconfined compressive strength than on the maximum dry density.
- For the plastic clay (A-7-6), lime is the most effective stabilizer.
- Class 'F' lime-fly ash mix is an effective alternative stabilizer.
- The addition of lime and/or fly ash to the A-7-6 soil makes the soil more workable but reduces the maximum dry density achieved.
- The optimum moisture content increases with the addition of lime and/or fly ash to the A-7-6 soil.
- An increase in Class 'C' fly ash content reduces the PI and increases the strength of the A-7-6 soil. However, with increasing Class 'C' fly ash content, the amount of swell reduction decreases.
- For very highly plastic clays,  $PI \approx 60$  or more, excavation and disposal may be a more cost-effective alternative than stabilization.

The design requirements for subgrades are that the maximum dry density should be at least  $1800 \text{ kg/m}^3$ , the PI should be reduced to 10-12% and the swell should be kept to a minimum (personal communication with Tom Parker). Also, it is suggested that a 7-day

unconfined compressive strength of 350 kPa is adequate for lime treated subbase soils (Tex-121-E). However, the laboratory tests indicate that although maximum dry density and PI may not meet required values, the strength and swell can at least meet if not exceed suggested requirements. Also, the strength of soils stabilized with lime and/or fly ash increases with time due to pozzolanic reactions. Hence, based on the design requirements and the laboratory observations, the following mixes are recommended for pilot field sections:

- A-3 soil + 15% Class 'C' fly ash at 7.5% moisture content,
- A-3 soil + 20% Class 'C' fly ash at 6.5% moisture content,
- A-3 soil + 3% quick lime (type C grade DS) + 8% Class 'F' fly ash at 11.5% moisture content,
- A-7-6 soil + 10% Class 'C' fly ash at 11.5% moisture content,
- A-7-6 soil + 15% Class 'C' fly ash at 14% moisture content, and
- A-7-6 soil + 2% quick lime (type C grade DS) + 5% Class 'F' fly ash at 14.5% moisture content.

The percentages recommended above should be increased by 0.5% to 1% to account for losses in the field. From strength considerations, it is desirable to compact the soils within 1-2% dry of the optimum moisture content. The stabilized subgrades should be monitored to study the long-term effectiveness of stabilization. The influence of a delay between mixing the stabilizer and compacting, in the case of cohesionless soils, is expected to be more severe under field conditions. Hence, the field trials should include sections wherein such delays are introduced. Also, the subgrade should be stabilized to different depths ranging from 0.15 to 0.45 m to study its effect on the overall performance of the pavement.

Bredenkamp and Lytton (2.14) have reported benefits of swell reduction in subgrade soils with high sulfate content by using low-calcium fly ash over lime as a stabilizer. It is recommended that high-sulfate soils be identified in the Bryan District and that the effectiveness of low- and high-calcium fly ashes and lime-fly ash mixes as stabilizers be evaluated. Also, studies should be carried out to compare the effectiveness of fly ash and lime-fly ash mixes with those of cement stabilization.



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## SECTION III

### USE OF BY-PRODUCT GYPSUM IN ROADWAY CONSTRUCTION

#### BACKGROUND

##### By-Product Gypsum

Gypsum, or calcium sulfate dihydrate, is a naturally occurring, non-metallic mineral used as a raw material in the manufacture of gypsum board, portland cement, plaster products and in agriculture as a soil enhancer. By-product, or synthetic gypsum, includes a family of calcium sulfates including Flue Gas Desulfurization (FGD) gypsum, phosphogypsum, fluorogypsum, titanogypsum, and disulfogypsum. These are names used in engineering and scientific literature for by-product calcium sulfates to reflect the particular process or industry that produced them. Current practices, as they relate to managing these materials, offer the following three alternates (3.1):

1. Waste disposal or stockpiling,
2. Replacing sulfuric acid acidulation with other mineral acids that do not precipitate calcium salts in a solid form, and
3. Exploitation for commercial applications.

Until recently, waste disposal was the primary method used in most of the world.

Texas Transportation Institute (TTI) has studied virtually all types of by-product gypsum and has been intimately involved with roadway applications of phosphogypsum, (3.2) fluorogypsum (3.3), and FGD gypsum (3.4, 3.5). Strength development through stabilization with fly ash and portland cement have been exploited with both the dihydrate and hemihydrate forms of gypsum (3.6).

### **Coal Combustion By-products (CCBP)**

Based on a study conducted by the American Coal Ash Association, Inc. (ACAA), approximately 81 million megagrams (Mg) of coal ash by-products were produced in 1991 of which only about 25 percent was utilized (3.7). These by-products include fly ash, bottom ash, boiler slag, and FGD sulfates and sulfites. The ACAA lists the following as the primary outlets for these materials: cement and concrete products, structural fills, roadbases and sub-bases, mineral filler in asphalt, road surface grit for snow and ice, blasting grit and roofing granules, grouting, coal mining applications, and the solidification and stabilization of industrial wastes (3.7).

### **Dry Bottom Ash and Boiler Slag**

When pulverized coal is burned for the production of steam at electric power generation plants, a portion of the ash content (usually 10 to 40 percent) collects on the furnace walls or falls to the furnace bottom. The type of pulverized coal-burning furnace (wet or dry) has a major influence on the physicochemical characteristics of the resultant by-product. In general, dry bottom ash is a gray, cinder-like granular material that is both insoluble in water and inert. Boiler slag or wet bottom ash, is black, glassy, and angular and resembles crushed, dark-colored glass. Boiler slag is heavier in weight than dry bottom ash, and its hardness permits its use in blasting and roofing applications.

### **Flue-Gas Desulfurization Gypsum**

FGD gypsum is generally produced from power plants burning lignite or sulfur coals. Sulfur is a natural contaminant in some coals, and it is almost completely converted to sulfur oxides ( $\text{SO}_x$ ) when coal is burned. Consequently, sulfur oxides, being released primarily in the form of sulfur dioxide, are converted by atmospheric processes to a sulfate. Once they enter the atmosphere, research has shown they contribute to the formation of acid rain. For this reason, the Environmental Protection Agency (EPA) has established stringent regulations about discharging sulfur dioxide emissions into the air.

In accordance with the 1977 Clean Air Act Amendments, the EPA has established regulations that require electric power companies and industries to take steps to reduce SO<sub>x</sub> emissions and particulates. The most prevalent means of SO<sub>x</sub> control are FGD systems, which remove sulfur dioxide from exhaust gases before they are emitted into the air. The Clean Air Act Amendments of 1990 require coal-burning plants to further reduce SO<sub>x</sub> emissions by either installing costly scrubber systems or switching to low-sulfur coal. In the former, sulfur dioxide gases are washed with a limestone slurry to produce calcium sulfate, usually of the dihydrate form, which is subsequently pumped to large settling basins.

As the volume of FGD by-products increases, the cost of dedicating more and more land to accommodate these materials grows accordingly. Managing these vast storage areas to assure minimal environmental impact adds a cost that must ultimately be borne by the electric power consumer. Without suitable high-volume outlets for these by-products, these conditions can only be expected to worsen.

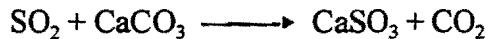
The largest single source of SO<sub>x</sub> emissions is coal combustion (3.8). It is estimated that annual consumption of coal by the electric power utilities is expected to increase from 707 million metric tons per year in 1992 to between 600 million and one billion metric tons in the year 2000 (3.1). Consequently, the amount of SO<sub>x</sub> could increase from about 20 million to 41 million metric tons per year during the same period. However, as plants put new, more efficient SO<sub>x</sub> control systems on line, future emissions are expected to remain at approximately the current annual level.

Coal contains sulfur in two forms: mineral sulfur in the form of pyrite and organic sulfur that is chemically bound in the coal. Mechanical coal-cleaning processes can remove most of the mineral sulfur. However, the removal of organic sulfur requires chemical processing and can be achieved by either microwave desulfurization or hydrothermal desulfurization (3.1).

FGD is the most common method of removing sulfur oxides resulting from the combustion of fossil fuels. FGD processes result in SO<sub>x</sub> removal by inducing exhaust gases to react with a chemical absorbent as they move through what is called a scrubber. The absorbent is dissolved or suspended in water, forming a solution or slurry that can be sprayed

or otherwise forced into contact with the flue gases. Ninety percent of the FGD systems in use today use limestone (CaCO<sub>3</sub>), calcium hydroxide (Ca(OH)<sub>2</sub>), or calcium oxide (CaO) as the chemical absorbent. However, portland cement may also be used as an absorbent (3.8).

Reactions taking place in flue gas desulfurization can be summarized as follows:



Texas coal-fired electric power plants produce about one million tons/year of a dihydrate form of by-product gypsum (i.e., CaSO<sub>4</sub> · 2H<sub>2</sub>O). Table 3-1 summarizes the annual production of FGD gypsum at several power plants in East Texas. This rate is expected to increase due to the addition of new fired units currently in the planning stages or under construction. All new units will have wet scrubber systems to comply with EPA regulations for sulfur oxide emissions (3.7).

Table 3-1. Annual Production of FGD Gypsum in Texas.

<u>Location</u>	<u>Estimated Annual Production (Mg)</u>
TU-Electric/ALCOA Rockdale	149,688
TU Electric Martin Lake	435,360
Southwest Electric Power Co. (SWEPCO)	117,910
Houston Lighting & Power	<u>204,075</u>
Total Production	907,033

## **TTI EXPERIENCE WITH BY-PRODUCT GYPSUM**

Research activity at Texas A&M University for the beneficiation of by-product gypsum began in 1982 (3.2, 3.3) with an attempt to exploit the phosphogypsum stockpiles at the Mobil Chemical Company and Gulf States Materials, Pasadena, Texas, plants for use in road base construction. Three significant findings were reported:

1. The strength achieved following stabilization using either a high-lime fly ash or portland cement was considerably higher in a 7-year-old, inactive pile (Pile 2) than in one that was currently active (Pile 3) see Figure 3-1.
2. At that time, the factor that appeared to be the primary influence on strength was stockpile acidity as reflected by pH. The active pile (pH - 2.5) developed little or no strength, whereas the aged pile (pH - 5.5) showed a higher degree of stabilization.
3. The particle-size distribution and shape of the material in the older pile were more varied than in Pile 3. These features formed the basis for achieving higher densities and stability through better compaction.

### **Fly Ash and Cement-Stabilized Gypsum**

In 1985, the Bureau of Mines sponsored a new study to establish a mix design rationale for the use of by-product gypsum in roadway applications (3.9). A significant outgrowth of that study was a set of procedures for material selection, sample preparation, and testing gypsum mixtures (3.10). These criteria were employed in a series of experiments wherein gypsum samples were stabilized over a range of fly ash and cement contents. Figures 3-2 and 3-3 show the results.

Figure 3-2 shows strength development in mixtures stabilized at Class 'C' fly ash contents of 10, 15, 20, and 30 percent. The American Society for Testing and Materials (ASTM) recommends an allowable strength for lime-fly ash-stabilized bases of 2725 kPa after three days. Using this criteria, acceptable strengths would be developed with 15 percent fly ash. The addition of 30 percent fly ash boosted this strength to 12,410 kPa. The 28-day strength of the 15 percent fly ash mixture approached 4137 kPa.

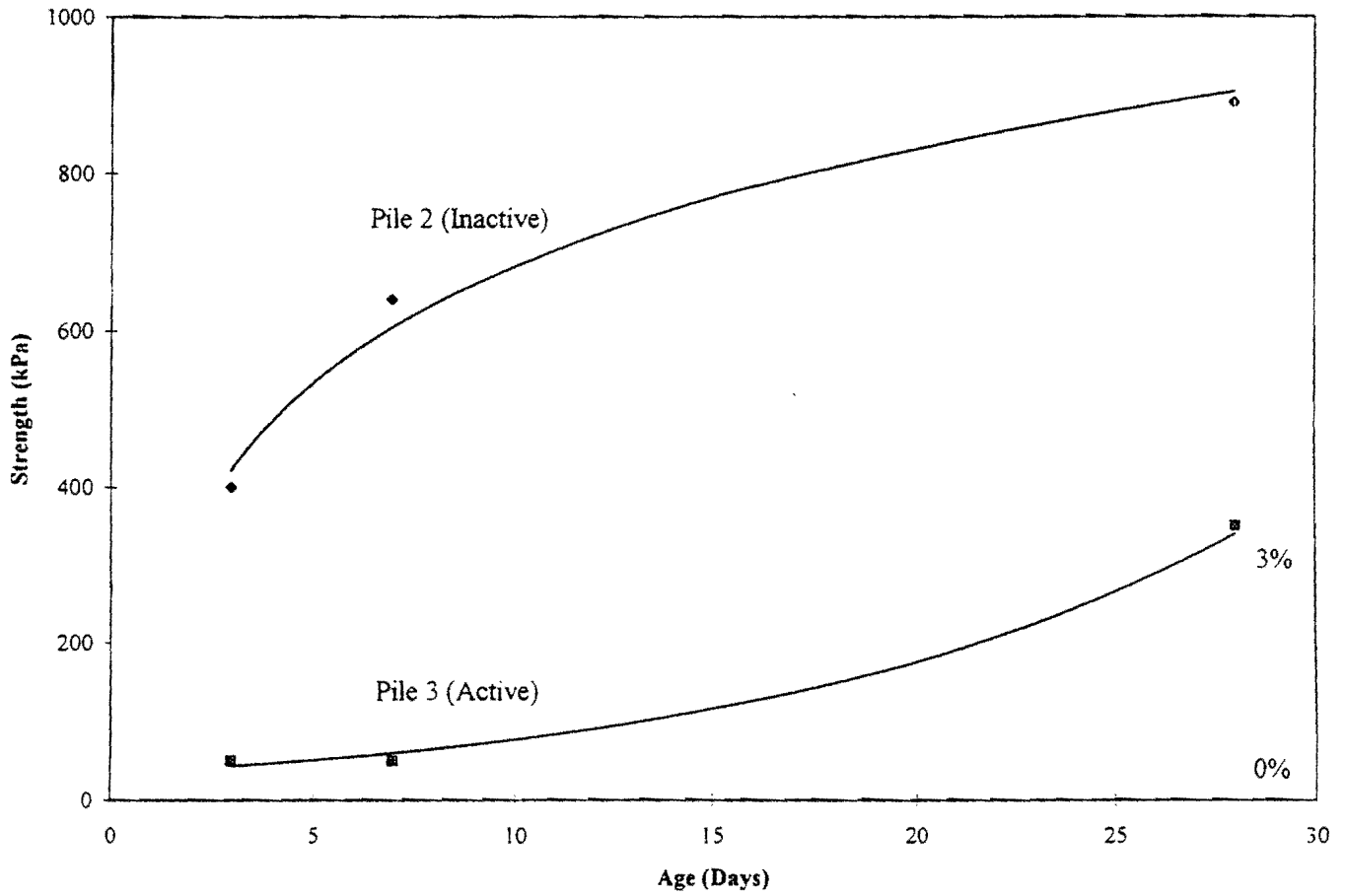


Figure 3-1. Development of Unconfined Compressive Strength with Time for Aged (Pile 2) and Unaged (Pile 3) Phosphogypsum.



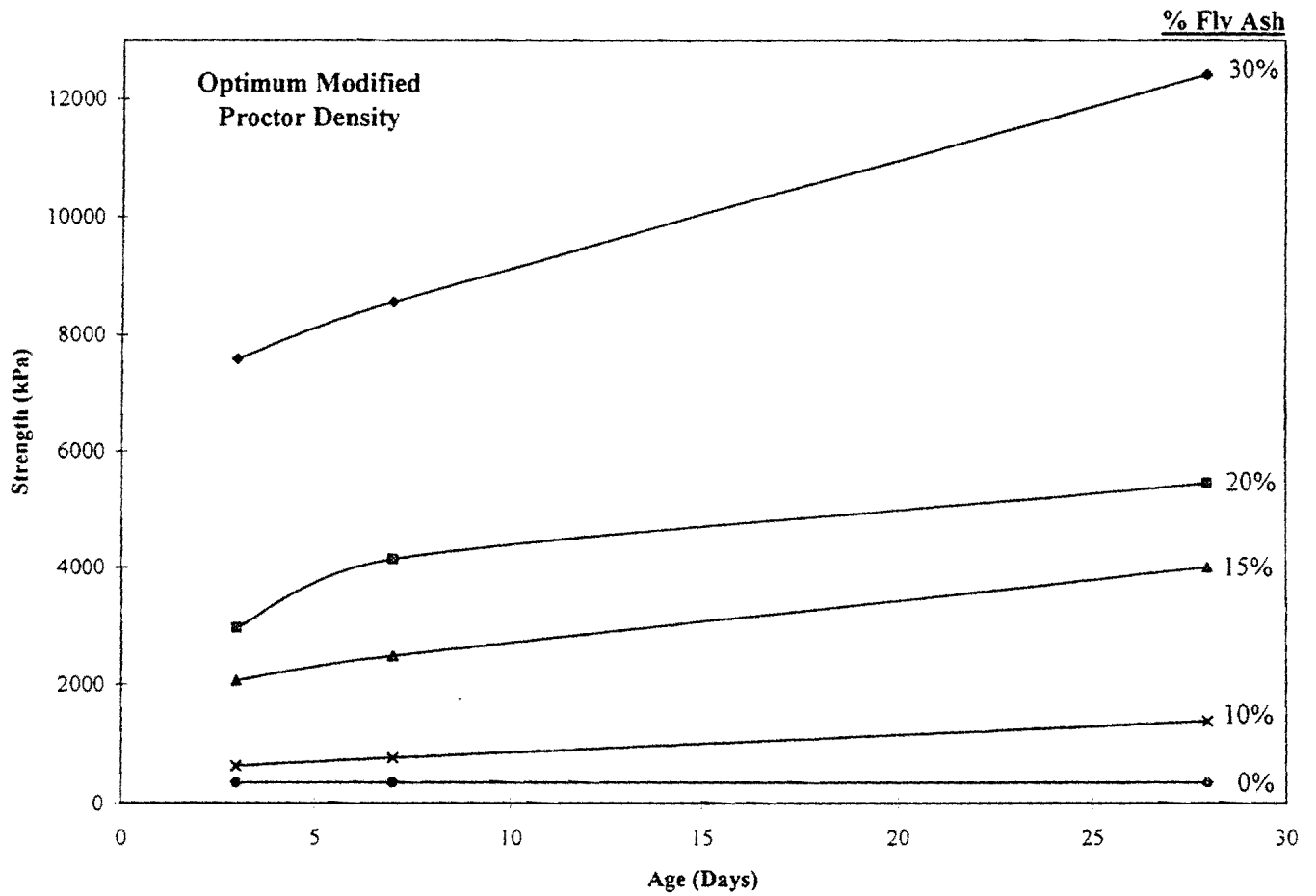


Figure 3-2. Development of Unconfined Compressive Strength with Time for Fly Ash-Stabilized Pile 2 Phosphogypsum.

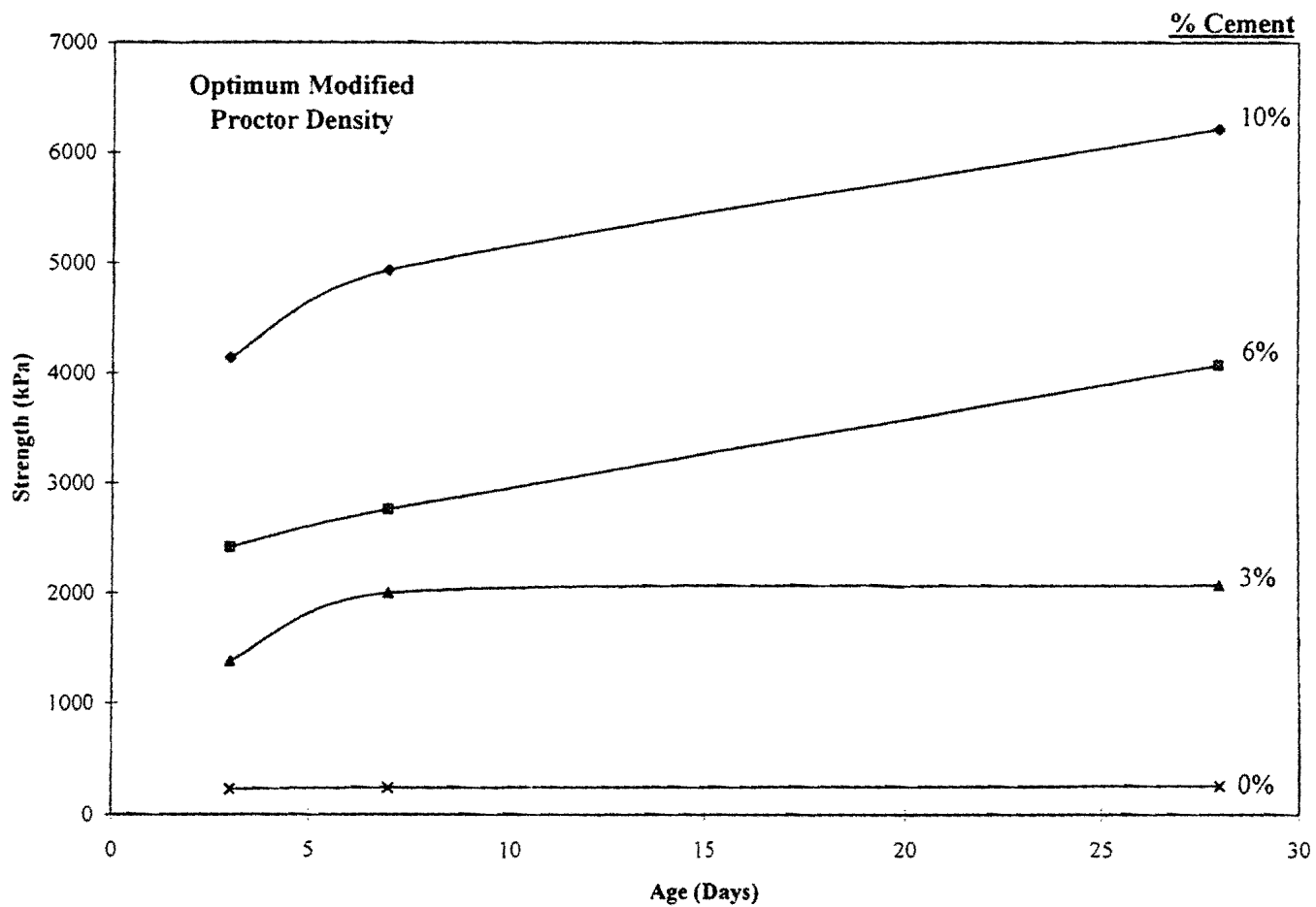


Figure 3-3. Development of Unconfined Compressive Strength with Time for Portland Cement-Stabilized Pile 2 Phosphogypsum.

Figure 3-3 depicts strength development with the use of portland cement. The Texas Department of Transportation (TxDOT) requires a 7-day compressive strength of 4481 kPa for its cement-stabilized bases. On this basis, only those mixtures with 30 percent fly ash and 10 percent cement would meet this specification. The results of the Bureau of Mines study (3.9) suggest that a more realistic value would be based on a criteria in which the confined compressive strength be on the order of 10 times the flexural strength in the pavement.

Studies have shown that the flexural strength is approximately 0.2 times the confined compression strength (3.10). In a well-designed pavement system this flexural strength can be around 241-275 kPa. On this basis, the design strength would be around 2410 to 2750 kPa instead of 4481 kPa as normally required.

On this basis, the materials specifications recommended for stabilized gypsum mixtures are as shown in Table 3.2.

Table 3-2. Recommended Materials Specification for Stabilized Gypsum Road Base Mixtures.

<u>Gypsum</u>	
Dense Graded:	100 percent - 200 sieve
Moisture Content:	Optimum -2+1 percent pH $\geq$ 5.0
7-Day Compressive Strength:	2,413 kPa (min) or comp $\geq$ 10 flex
Compaction:	Modified Proctor (ASTM 1557)
<u>Element</u>	
Sulfate Resistant; $C_3A \leq 4\%$	
<u>Fly Ash</u>	
Class 'C' - CaO $\geq 15\%$	

Several experimental field projects were constructed using different stabilized by-product gypsum mixtures. One project involved seven road base sections in the City of LaPorte, Texas (3.11). TTI supervised the construction of these experimental test sections utilizing by-product phosphogypsum material from Mobil's stockpile and received by-product fluorogypsum from Gulf States Materials of LaPorte, Texas. This project took place during the summer of 1983 and a four year post-construction evaluation program.

Field samples in the form of corings indicated no sign of strength degradation in that time period. Table 3-3 shows this and with other projects in the Houston area incorporating stabilized by-product gypsum. The test sections using the fluorogypsum (Item 7) were not stabilized since this by-product was produced in the hemihydrate form. Only the addition of water and compaction were required to produce the road base. All sections were subsequently given a chip seal surface treatment. Pavement analyses indicate the structural integrity of all the LaPorte test sections was at least equal to and in some cases superior to conventional crushed limestone or cement-stabilized base materials (3.11).

Table 3-3. Some Field Sites Reflecting Various Uses of Stabilized Phosphogypsum.

Location	Function of Facility	Approximate Date of Construction	Materials Utilized
1. Fertitex Cross-over Road; vicinity of MCC Pasadena, Texas, Plant	Haul road for phosphate rock and sulfur, etc.	12-79	Phosphogypsum, fresh fly ash, cement flue dust
2. Maritime Services, Port of Houston	Container yard	4-81	Phosphogypsum, reclaimed fly ash
3. MCC Parking Lot; Pasadena Chemical Corporation	Parking lot for employees	6-82	Phosphogypsum, fresh fly ash, cement, reclaimed fly ash, lime
4. Heights Bonded Warehouse, Houston	Pipe storage yard	7-82	Phosphogypsum, fresh fly ash
5. Robin Pipe Yards, Houston	Pipe storage yard	Unknown	Phosphogypsum, reclaimed fly ash
6. City of LaPorte, Texas (3.11)	City Streets	6-83	Phosphogypsum, fly ash, cement
7. City of LaPorte, Texas (3.3)	City Streets	6-83	Fluorogypsum

### **Problem Areas with By-Product Gypsum Construction**

Road bases for city streets, shopping centers, truck terminals, parking lots, and loading platforms have been successfully constructed in the Houston area of Texas using cement and fly ash-stabilized gypsum such as phosphogypsum and fluorogypsum. Personal contacts with two suppliers, Gulf States Materials (fluorogypsum) and Mobil Chemical Company near Pasadena, Texas (phosphogypsum), indicate a better than 95 percent success rate on over 200 projects (3.12, 3.13). TTI was involved in the mix design development of the base courses utilizing both of these by-product gypsums (3.2, 3.3, 3.11, 3.14, 3.15).

Construction difficulties have been encountered when attempts are made to extend the stabilized-gypsum road base concept to state and federal roads. One project using a 10 percent cement-stabilized phosphogypsum base on Texas SH 146 proved unsuccessful (3.16). One other project in Texas (3.15) using varying amounts of fly ash and cement as stabilizers for a by-product gypsum also had to be replaced after less than a year in service.

In virtually every case, when construction difficulties were encountered, the problems could be related to one or more of the following sources:

- a. Too much moisture added during construction,
- b. Overstabilization,
- c. Incomplete mixing,
- d. Incompatible stabilizers, prime coats, etc.,
- e. Insufficient compaction and weather sealing, or
- f. Road opened to traffic too soon.

### **Problems Areas with Gypsum Roadbase Construction**

a. Moisture: Overwatering in the field, either while trying to achieve the specified moisture content or to maintain dust control, will weaken the base during its most critical period of strength development. One of the prime times for this type of damage is at transitions from one day's work to the next. An improperly prepared transition at the end of a roadway or a changed mix design are also potentially

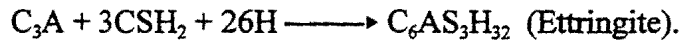
vulnerable to swelling due to the accumulation of excessive moisture or improper compaction.

b. Stabilization: Different states qualify their allowable strengths for stabilized bases based on different numbers of days permitted for curing. For example, Texas requires 4481 kPa strength after 7 days, whereas Illinois specifies 4481 kPa after 14 days. The chemical interaction between gypsum and cement is one that proceeds at a slow rate of hydration. The need to use slower curing, Type II, (sulfate resistant) cements further affects the rate. As a consequence, the inability to reach a required 7-day strength is usually compensated by adding excessive stabilizer. This was the case on SH 146 (3.16), which, along with excessive moisture, produced severe swelling.

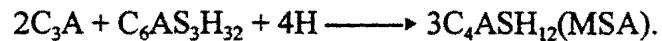
c. Mixing: Blending of mix ingredients can be accomplished successfully either in-place (3.2) or in a pug mill (3.5). The latter has advantages of allowing for field calibration checks to ensure compliance with job mix specification and achieving good mix homogeneity. On small jobs, pug mill operations tend to be more expensive than mixing in place unless these initial operational costs can be distributed over other projects. Smaller projects, or projects that cannot be conveniently located near a pug mill, may favor mixing in place. In this case, the ability to deliver good mix homogeneity may be a problem. Experience with one project in Texas indicated that the lower 5 cm of a 20 cm base constructed by mixing in place was not successfully blended by the pulverizer (3.5). Constructing the section in multiple lifts or using a pulverizer with longer tines could have alleviated this problem. Unfortunately, these deficiencies are not normally encountered until core samples are taken, usually long after the road has been opened to traffic.

d. Incompatible Stabilizers and Prime Coats: Cement type and content have a great influence on strength development in stabilized by-product gypsum mixtures. Tricalcium aluminate ( $C_3A$ ) is one of the principal aluminate compounds in portland

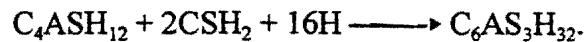
cement. To achieve sulfate resistance in portland cement concretes, ASTM C150 recommends that the  $C_3A$  content in Type II cements should be kept below 7 percent. Studies involving cement-stabilization of gypsum-based mixtures have shown that  $C_3A$  contents no greater than 3 percent are required to prevent sulfate attack and swelling (3.6, 3.10). The hydration of  $C_3A$  in portland cement involves a reaction with sulfate ions, which are supplied by the dissolution of gypsum. The primary initial reaction of  $C_3A$  is:



Ettringite is a stable hydration product only while an ample supply of sulfate is available. If all the sulfate is consumed before the  $C_3A$  has completely hydrated, the ettringite transforms to a monosulfoaluminate (MSA) that contains less sulfate:



When monosulfoaluminate is brought into contact with a new source of sulfate ions, ettringite is reformed:



This potential for reforming ettringite is the basis for sulfate attack of portland cements when they are exposed to an external supply of sulfate ions. Once ettringite has formed, it continues to grow expansively. If the temperature of the system drops below approximately 15°C (59°F), ettringite, through a series of intermediate reactions is transformed to thaumite, (3.17), a complex calcium-silicate-hydroxide-sulfate-carbonate-hydrate mineral. Both ettringite and thaumite are hydrous minerals. Without an abundance of water or excessive  $C_3A$ , they cannot form.

Typical mixtures of cement-stabilized by-product gypsum contain between 4 and 10 percent cement. The remainder of the mix ingredients is gypsum and water.

Therefore, it is correct to assume that there is a large supply of sulfate ions available to hydrate all the aluminate ions in the cement. Monosulfoaluminate,  $C_4ASH_{12}$ , will never form since there is no sulfate ion deficiency. Consequently, the phenomenon of reforming ettringite in portland cement does not apply to by-product gypsum systems. However, thaumite may form at temperatures below 15°C. Since both ettringite and thaumite are expansive products, extreme caution should be taken in the indiscriminate specification of sulfate-resistant cements and mortars to be used for the stabilization of by-product gypsum.

Specifying sulfate-resistant (Type II) cements without further specifying a maximum allowable  $C_3A$  content can cause major swelling problems. The maximum  $C_3A$  content in a Type II cement as set forth by ASTM C150 can be as high as 7 percent, which is more than 4 percentage points above the maximum allowed for gypsum mixtures (3.18). It should be noted that when cement/fly ash blends are used as stabilizers, their combined  $C_3A$  contents should not exceed the 3 percent allowable. Recently, several suppliers have started producing high-sulfate-resistant (HSR) cements.  $C_3A$  contents for these cements can range from 0 to 3 percent.

Two prime coats that have shown to work well as a tack over compacted stabilized gypsum bases have been MC30 and RC250 cutback asphalts. Attempts to use emulsions have proven unsuccessful since they tend to add additional water to the base while it is in its initial curing phase and most vulnerable. The presence of this excess water tends to create a weak shear plane about 1.25 cm below the surface during compaction, subsequently compromising the integrity of the entire base when traffic is introduced or when deep freeze climates are encountered.

e. Compaction and Sealing: The degree and type of compaction are critical factors affecting the ultimate strength achieved in stabilized gypsum bases. Figure 3-4 shows



the effect of compaction on both optimum moisture content and dry density. Similarly, Figure 3-5 shows the effect of compaction on tensile and unconfined compressive strength (3.18). The specification of a field density testing method should be coordinated with the local state highway department. Similar to state standards for 7-day strengths, some deviation from standard practice should be permitted, given the slow hydration rate of gypsum with cement.

For example, the laboratory compaction recommended for stabilized gypsum is Modified Proctor as prescribed by ASTM D1557, which delivers 2697 kJ/m<sup>3</sup> of energy to the specimen (3.10). TxDOT uses its own Modified Proctor test and specimen configuration under Texas Method 113-E, which only delivers 1100 kJ/m<sup>3</sup> of energy. Figures 3-4 and 3-5 indicate that the latter would predict a higher optimum moisture content and, consequently, achieve a lower strength than that obtained using ASTM D1557. Using Texas Method 113-E on stabilized-gypsum base mixtures would produce non-conservative decision criteria for the design of the base by usually indicating an unnecessary need for more water and stabilizer. Saylak et al. (3-10) developed a set of recommended testing procedures and material selection criteria. These procedures were developed over the 10 years TTI has been studying various gypsum by-products and stabilized-FGD gypsum base mixtures.

Insufficient sealing of the base can make it susceptible to premature damage. Two treatments of a standard chip seal surface treatment or a 38 to 50-mm-thick hot mix asphalt concrete wearing coarse has been found to be effective.

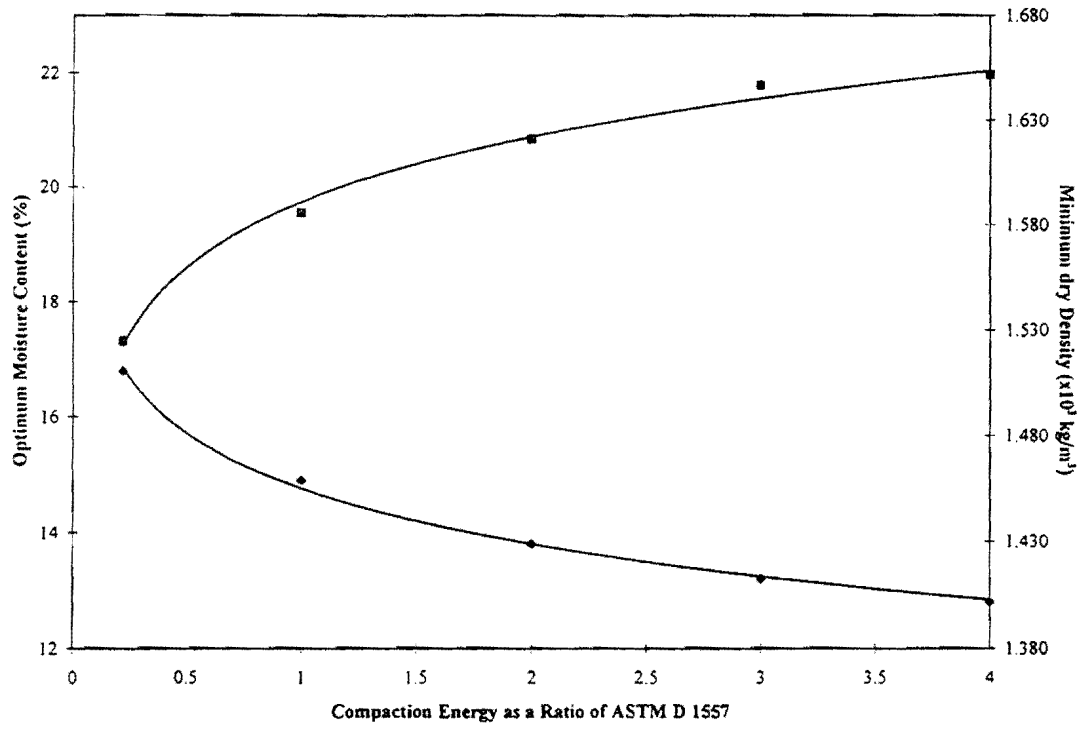


Figure 3-4. Effect of Compaction Energy on Density and Optimum Moisture Content.

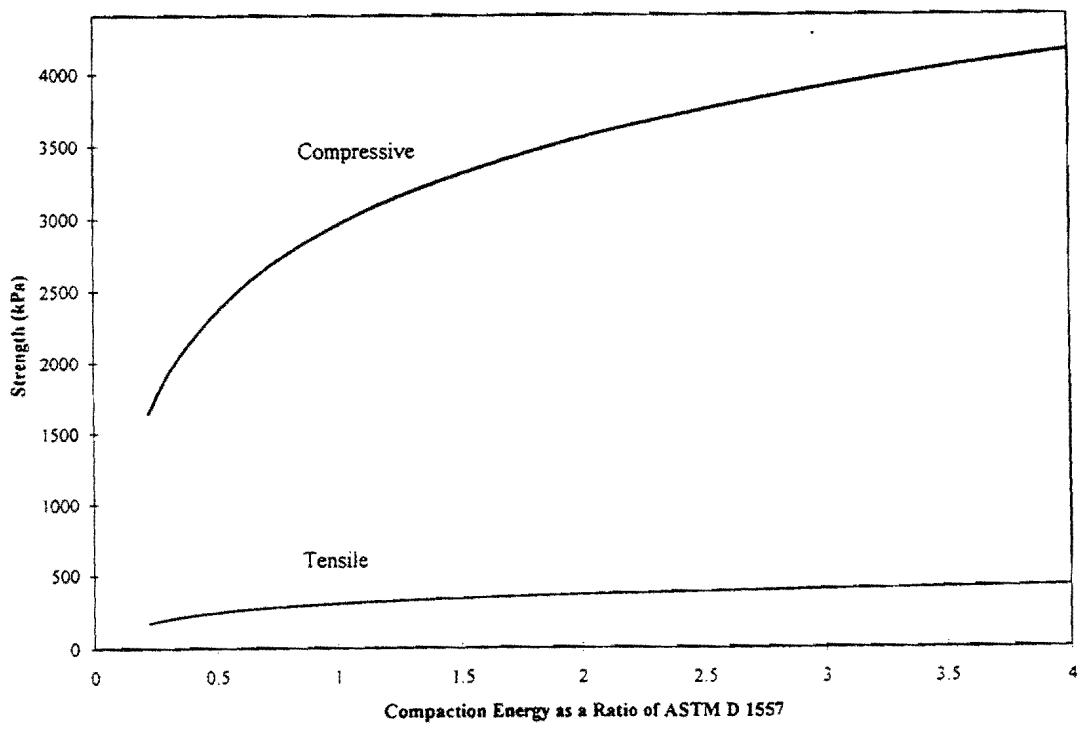


Figure 3-5. Effect of Compaction Energy on Tensile and Compressive Strength.

## **TTI EXPERIENCE WITH THE UTILIZATION OF CCBP FGD GYPSUM AND GYPSITES IN ROAD CONSTRUCTION**

A number of researchers ([3.2](#), [3.3](#), [3.5](#), [3.6](#), [3.11](#), [3.14](#), [3.19](#), [3.20](#)) have provided evidence that FGD materials can be used in road bases or sub-bases through stabilization with either portland cement, fly ash, or combinations of both. Figure 3-6 shows a comparison of the particle-size distributions for several different by-product gypsums currently being generated in Texas. When properly mixed, compacted, and cured, these materials will develop sufficient strength for field applications. The strengths for phosphogypsum and fluorogypsum and the results of field trials in which they were utilized were discussed above. Figure 3-7 shows typical 7-day unconfined compressive strengths for TU Electric's FGD gypsum stabilized over a range of cement contents. Figure 3-8 shows that the strength for both 8 and 13 percent cement-stabilized FGD gypsum continues to increase for 1 year. Strengths above 2400 kPa are considered sufficient for light to medium traffic loads. Figures 3-7 and 3-8 show that the material, when sufficiently stabilized and compacted, should qualify for most road bases and sub-bases. Similar tests run on TU-Electric and ALCOA FGD gypsum showed strength development to be similar to that generated in the Martin Lake material.

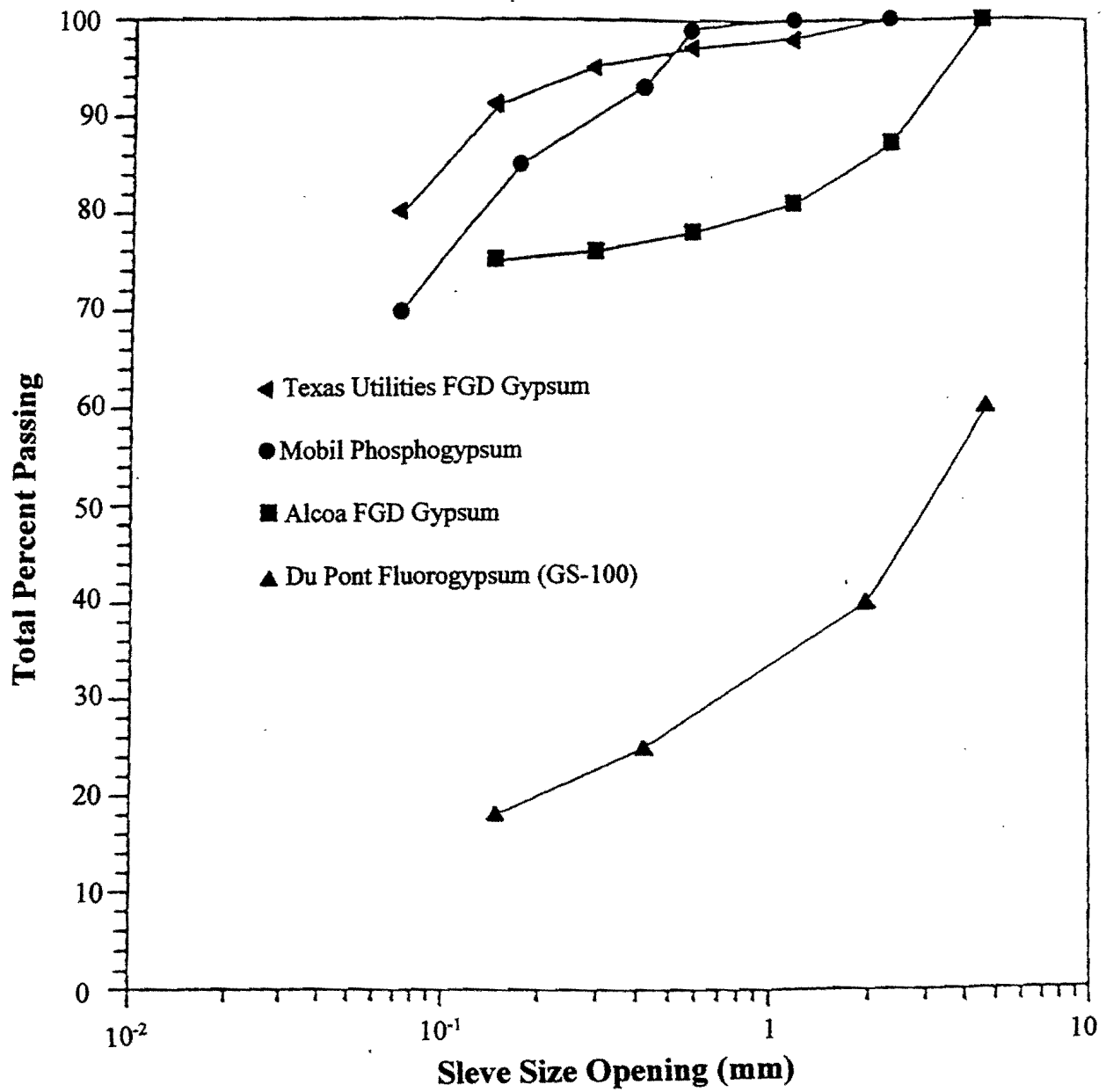


Figure 3-6. Particle-Size Distribution of Different Types of By-Product Gypsum.

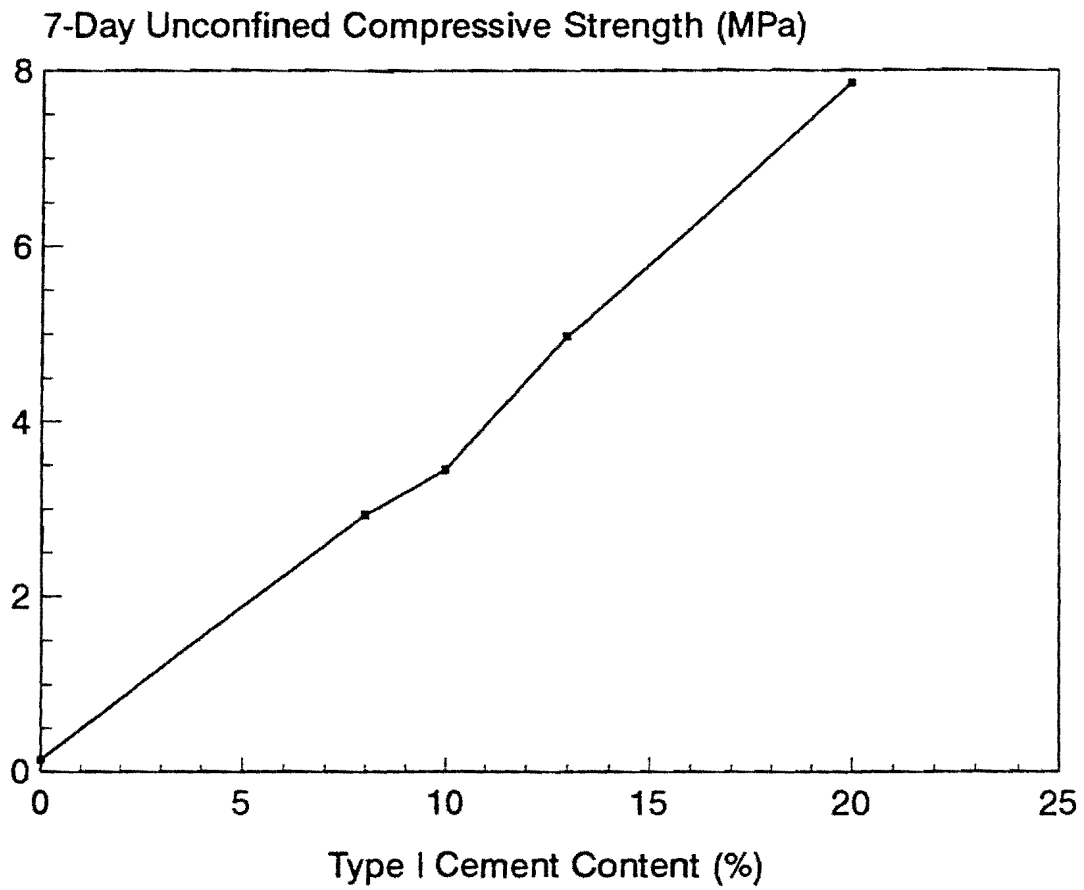


Figure 3-7. Unconfined Compressive Strength Versus Cement Content for Stabilized Texas Utilities FGD Gypsum Mixtures.

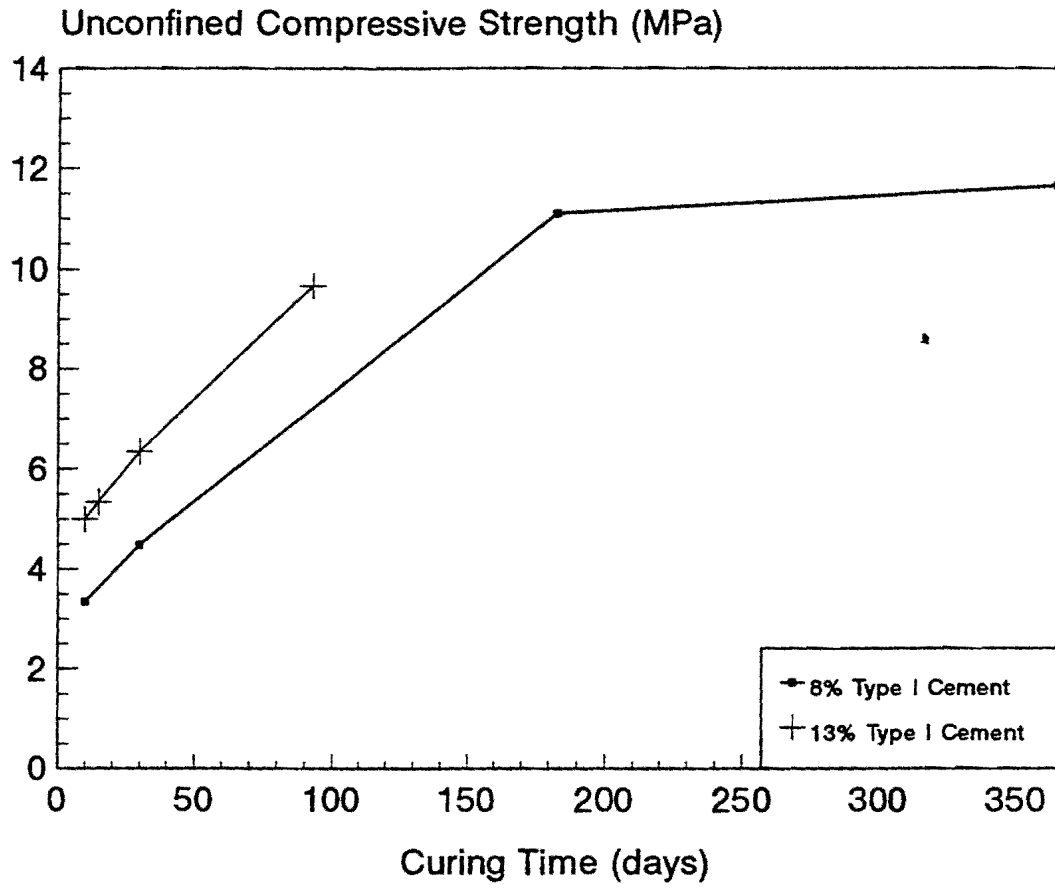


Figure 3-8. Unconfined Compressive Strength Versus Curing Time for Cement-Stabilized FGD Gypsum Mixtures.

### **1991 Cement-Stabilized FGD Gypsum Road Base Test Sections**

Since 1988, TTI has been investigating the stabilization of FGD gypsum produced at the TU Electric Martin Lake Plant near Tatum, Texas (3.5, 3.6). This study was integrated into a Texas Engineering Experiment Station (TEES) Study (3.15) directed toward establishing a materials selection criteria, suitable mix design rationale, and construction procedures to permit cement-stabilized FGD gypsum road bases to perform on Texas state roadways. The concepts generated out of this program were successfully demonstrated in the Summer 1991 and Fall 1992 when two-lane, 90-m-long test sections were consecutively placed at Texas A&M's Riverside Campus.

#### *Materials Selection*

The chemical constituents that made up the cement-stabilized gypsum/bottom ash mixtures used in Phase I were as follows:

**Cement:** The cement was a high-early strength (HES), high-sulfate-resistant (HSR), Class C Oil Well portland cement produced by Texas Industries, Inc. (TXI) of Midlothian, Texas. Table 3-4 gives a chemical analysis of the cement. Note that the  $C_3A$  content is 2.34 percent, which is consistent with that found in Type V portland cements. Appendix A shows company specification data.

**Gypsum:** ALCOA furnished the FGD gypsum generated at the TU Electric Sandow Power Station in Rockdale, Texas. It is a fine-ground, orthorhombic crystal with a grain-size distribution shown in Table 3-5. The chemical breakdown is given in Table 3-5, and the mill specification data are shown in Appendix A.

Table 3-4. Chemical Analysis of TXI Class C, Oil Well Cement.

Constituents	Weight Fraction	Constituents	Weight Fraction
SiO <sub>2</sub>	19.61	P <sub>2</sub> O <sub>5</sub>	0.21
Al <sub>2</sub> O <sub>3</sub>	4.10	TiO <sub>5</sub>	0.21
Fe <sub>2</sub> O <sub>3</sub>	5.68	ZnO	0.03
CaO	64.07	MnO <sub>3</sub>	0.32
MgO	0.87	Na <sub>2</sub> O	0.24
SO <sub>3</sub>	3.24	K <sub>2</sub> O	0.42
C <sub>3</sub> S	64.12	C <sub>2</sub> S	7.85
C <sub>3</sub> A	2.34	C <sub>4</sub> AF	17.30
CaSO <sub>4</sub>	5.5		
Blaine	4600		
Strength, kPa			
1 day	13,927		
3 days	27,399		
7 days	34,950		
28 days	42,699		

Table 3-5. Chemical Composition for TU-Electric/ALCOA FGD Gypsum.

Constituents	Weight Fractions
Ca	24
SO <sub>4</sub>	54
CO <sub>3</sub>	3
S <sub>1</sub> O <sub>2</sub>	2.7
Inert	1.3
H <sub>2</sub> O	15
pH = 7.0	



### *Construction*

The construction of the 1991 cement-stabilized FGD gypsum test section began in June 1991 (3.15). Work began by removing the existing roadway and compacting the remaining subgrade to a depth of 20.3 cm. The gypsum and cement were hauled to the job site separately and mixed on the ground using a pulverizer. Workers added water to achieve a moisture content of 13 percent. The mixture was then spread to grade. A padded-foot roller provided the initial compaction, which was followed by three passes of a 22.7 Mg pneumatic roller. Field density was measured using a Troxler Nuclear Density Meter that was calibrated to allow for the 2 molecules of structural water in the dihydrate gypsum.

One full day was allowed for the road base to cure after which a prime coat of MC-30 was applied at a spread rate of 54.3 l/m<sup>2</sup>. Two days later, a 38 to 50 mm Texas DOT Type D hot mix asphalt concrete (HMAC) surface course was placed over the base. The section was open to traffic the next day. Post-construction evaluations began after the test section was opened to traffic.

### **1992 Cement-Stabilized FGD Gypsum/Bottom Ash Road Base Test Section**

In the latter phase of the TU Electric/TEES coordinated program (3.15), some tests were run on cement-stabilized mixtures using various blends of bottom ash and gypsum. The bottom ash fractions consisted of a mixture of black, glassy boiler slag also designated "wet" bottom ash and a gray cinder like ash referred to as "dry" bottom ash. In these mixtures, different ratios, R, of wet bottom ash (WBA)/dry bottom ash (DBA) were blended on a 50/50 weight basis with a TU Electric FGD gypsum. This blend was stabilized using 7 percent by weight of dry solids of the same high early-strength (HES), high sulfate-resistant (HSR) cement used in the 1991 test section. Figures 3-9 and 3-10 show the results of these tests.

Five different ash ratios were used ( $R = \text{WBA/DBA} = 0/100, 25/75, 50/50, 75/25,$  and  $100/0$ ). The blend designated by  $R = 0/100$  (Control) was the same as that used in the 1991 gypsum base course demonstration project discussed above. Aside from a shift in optimum moisture content from 13 percent for the control to 9.5 percent for the mixes containing ash,

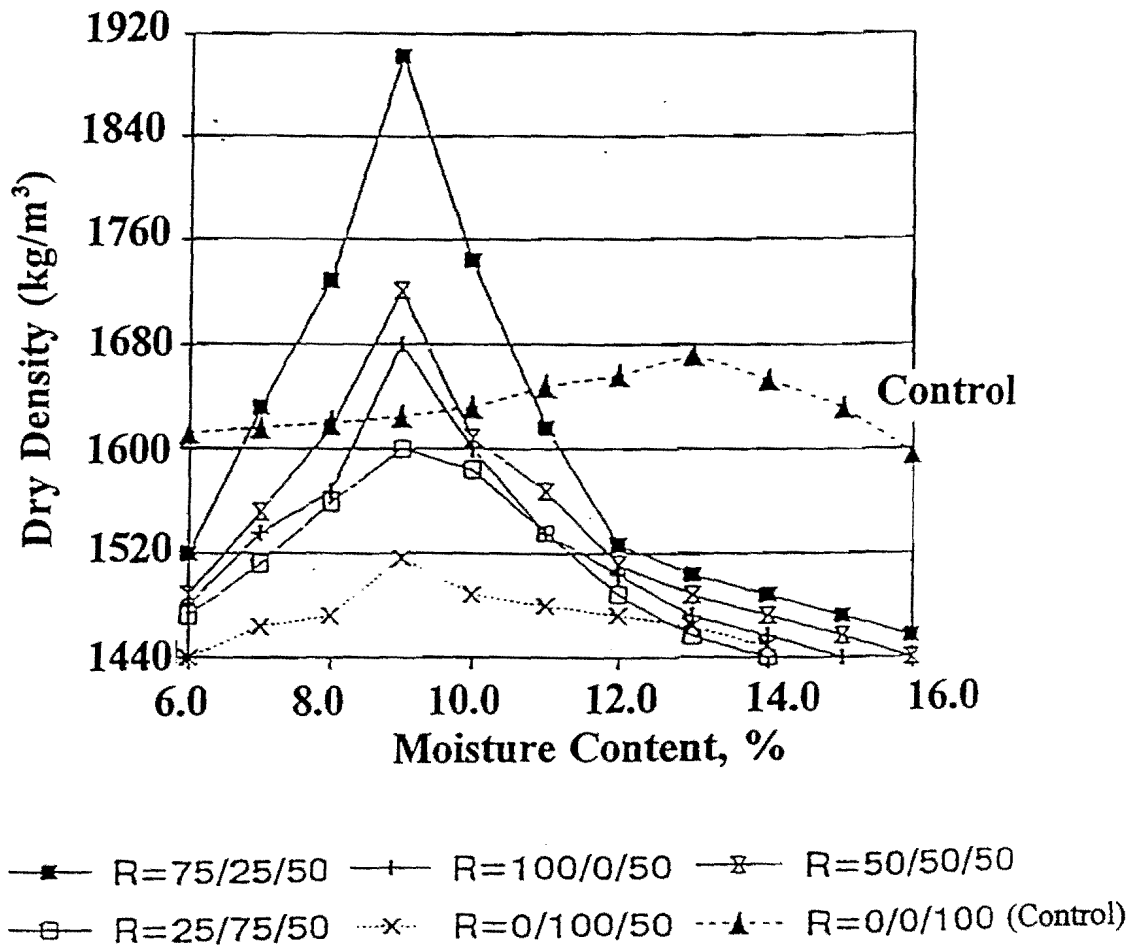


Figure 3-9. Moisture Content and Density at Various Ash to Gypsum Ratios, R. R=WDA/DBA/Gypsum.

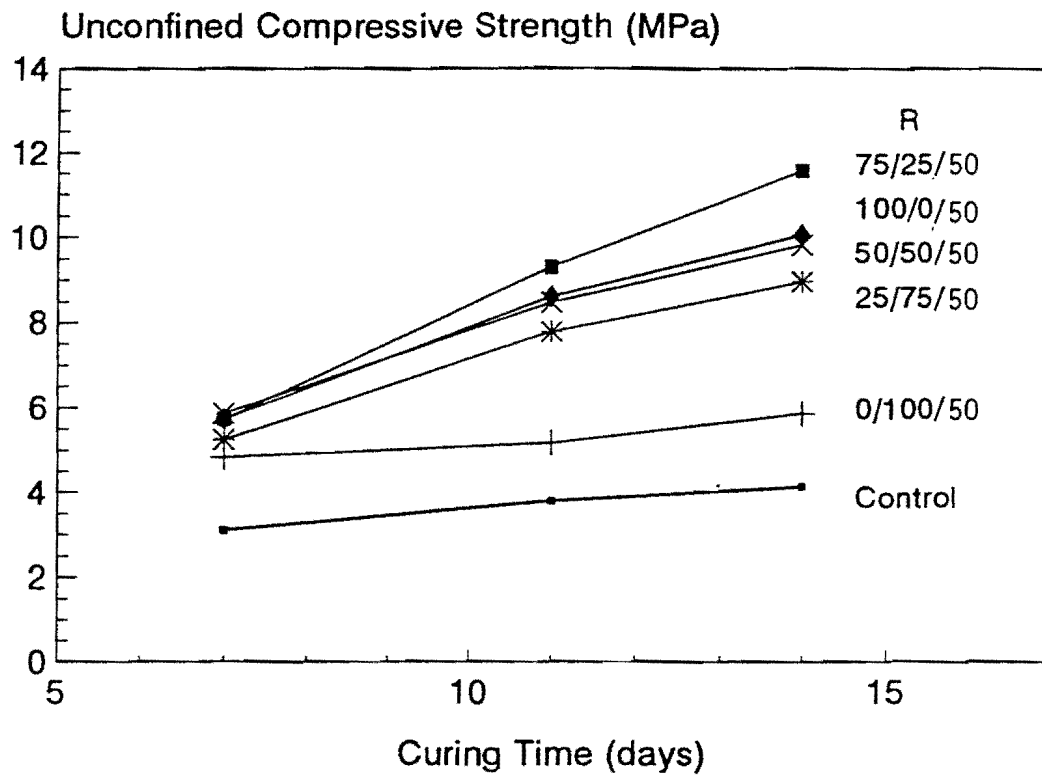


Figure 3-10. 7-Day Unconfined Compressive Strength Versus Cure Time at Various Ash to Gypsum Ratios. R=WBA/DBA/Gypsum.

there was also a significant variation in the dry compacted densities. The R = 75/25/50 blend (i.e., 75 percent WBA to 25 percent DBA blended on an equal weight basis with gypsum) had the highest density ( $1890 \text{ kg/m}^3$ ) as compared to  $1650 \text{ kg/m}^3$  for the control (i.e., where R = 0/0/100). As Figure 3-10 shows, this also translated into higher compressive strength. It is also interesting to note that the strength increases at a higher rate in the mixtures containing bottom ash as shown in Figure 3-11.

On the basis of these results, the Electric Power Research Institute sponsored a 2-year research project to study the use of stabilized-FGD gypsum/bottom ash mixtures for roadway construction (3.20). In the fall of 1992, a field demonstration similar to that performed in 1991 began using the R = 75/25/50 ash/gypsum blend in the road base.

#### *Materials Selection*

**Bottom Ash:** The TU-Electric/ALCOA plant in Rockdale, Texas, produces two types of bottom ash. One is a dry, gray cinder-like ash that is referred to as dry bottom ash (DBA) in this report. The second is a glassy, black ash called boiler slag. Because of its relative moisture contents this ash is designated as wet bottom ash in this report. The differences in the ashes' chemical composition are given in Table 3-6.

#### *Optimum Mix Design Determination*

The primary procedures for performing laboratory evaluations on road base materials were those given by Saylak et al. (3.10). This paper presents a rationale for the selection of materials, preparation of mixtures, and their testing and storage during cure. Tables 3-7 and 3-8 show the individual tests, ASTM designations, and tentative specification limits taken from (3.10).

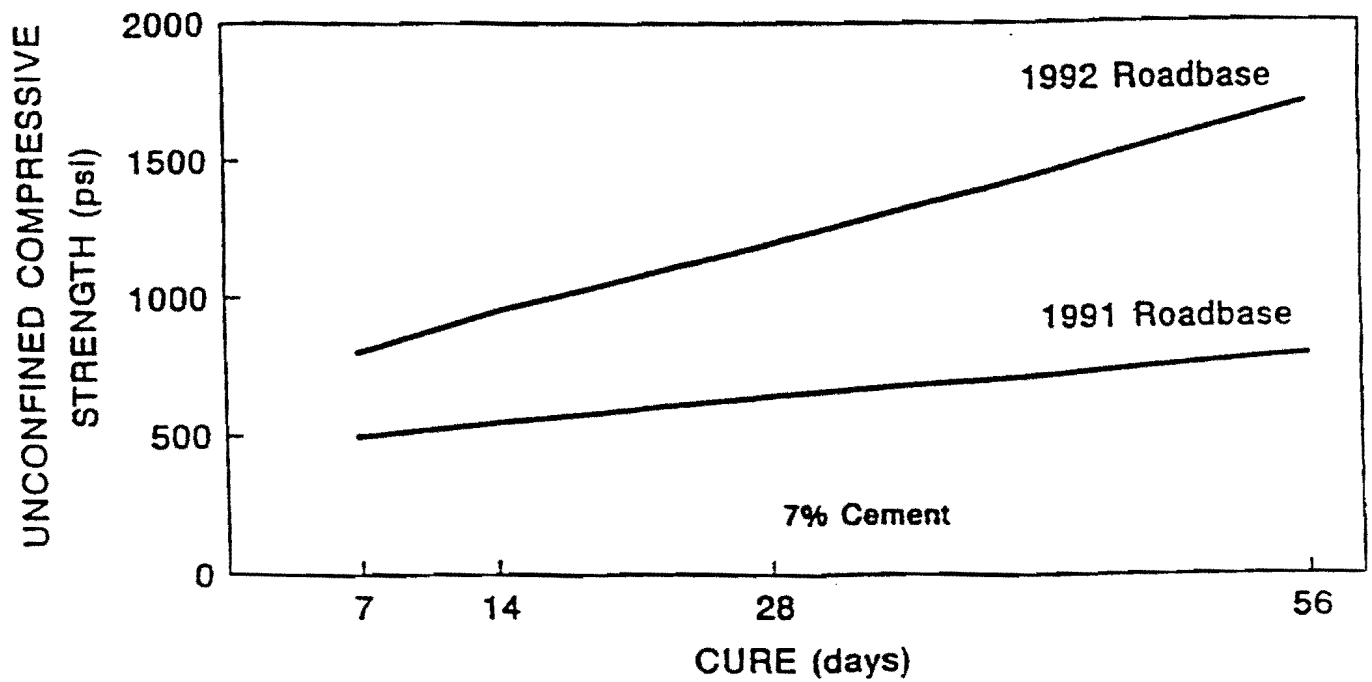


Figure 3-11. Comparison of Unconfined Compressive Strength Development in 1991 and 1992 Road Base Mix Designs.

Table 3-6. Chemical Composition for TU-Electric/ALCOA Bottom Ashes.

Elemental	Wet Bottom Ash (WBA), %	Dry Bottom Ash (DBA), %
Silica (amorphous)	30	30
Silica (quartz)	0	8
Aluminum	10	12
Calcium	0	8
Iron	8	5
Magnesium	1	1
Titanium	1	0
Water	50	36

**Physical Data**

Density: 881 kg/m<sup>3</sup> to 961 kg/m<sup>3</sup> (2.7 g/cm<sup>3</sup> - bulk density)

Water solubility: negligible

pH 8.1 (saturated solution)

The production rates of these ashes are 181,440 Mg/year for WBA and 272,160 Mg/year for DBA.

Table 3-7. Recommended Procedures for Evaluating Stabilized Gypsum Mixtures (3.12).

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<b>Moisture Content</b> ASTM D 2216	Drying temperature: 40° C (104°F)
<b>Moisture Density</b> ASTM D 1557 or AASHTO T 180	10.16 cm dia. x 10.8 cm sample size and Modified Proctor Compaction
<b>Unconfined Compression Test</b> Sample preparation Cure Test procedures	ASTM D 1557 Sealed bags or paraffin ASTM D 1633 using sample size from ASTM D 1557

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Table 3-8. Tentative Specification Limits.

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<b>Gypsum</b>	
Dense Graded	80% passing #200 sieve
pH	> 5.0
<b>Mixture</b>	
Moisture Content	Optimum + 1% - 2%
Compressive Strength $\sigma_c$	$\sigma_c \geq 10 \sigma_{Flex}$ *
Compaction	95 percent Modified Proctor (ASTM D1557)
	110 percent - Tex 113E
<b>Cement</b>	
Commercial grade, High-sulfate resistance, (C <sub>3</sub> A) < 3% ** High early strength	

---

\*7-Day Cure  
 \*\*Reduced from that given in Reference (3.10)

*Construction*

The test section for this project was identical in width and length and was placed in tandem with the 1991 test section. The base thickness for both sections was 20.32 cm. The existing roadway had a 30.48 cm crushed iron ore gravel road base topped with a two-course standard Texas Highway Department chip seal (See Figure 3-12 (a)). Figure 3-12 (b) and (c), respectively, show the cross sections for the 1991 and 1992 test sections.

The method for placing was essentially the same as that used on the 1991 test section except for the sequence in which the base materials were delivered to the site and placed. After the subgrade was prepared, the base materials were placed in the following optional sequence: (a) wet bottom ash, (b) gypsum, (c) dry bottom ash, and (d) cement. Following placement of the gypsum, and with each successive layer, the base was evenly distributed over the length of the section with a grader and pulverized to provide homogeneity. Water was added, as required, to achieve the optimum moisture content of 10.5 percent. The compaction of the base was similar to that performed on the 1991 test section as was the application of the MC-30 prime coat and 38 to 50 mm HMA surface course. The comparison of the new test site dimensions and job mix properties with the

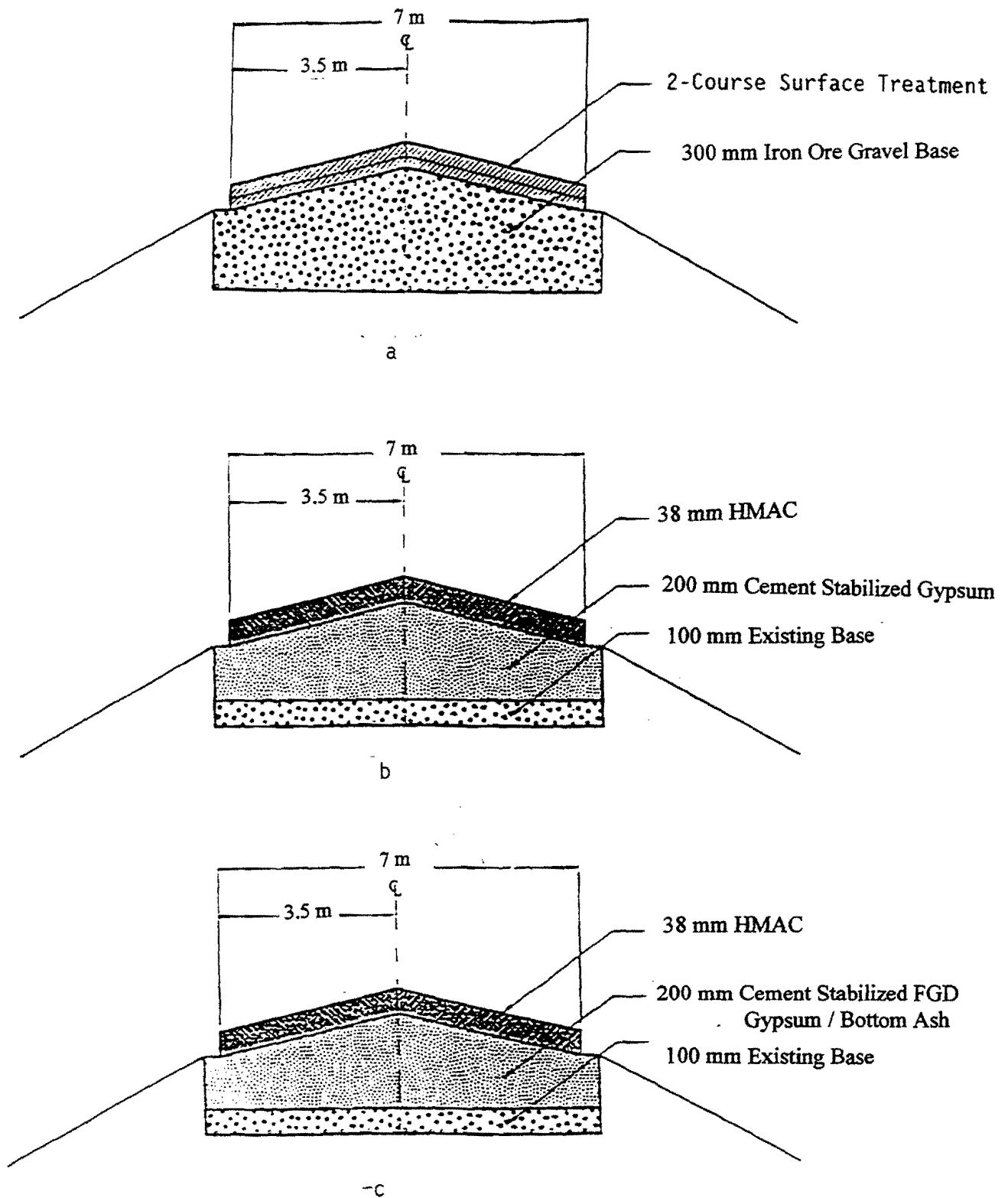


Figure 3-12. Cross Sections of (a) Existing, (b) 1991, and (c) 1992 Test Sections.



1991 test section are given in Table 3-9, which reflects the higher initial and faster rate of strength development in the gypsum/ash mixture. It should also be noted that field densities and moisture contents were more closely related to the ASTM D1557 compaction values as recommended by Reference (3.10) than those generated by Texas Method 113E.

Table 3-9. Comparison of 1991 and 1992 Test Section Construction Data.

Parameter	1991 Section	1992 Section
Base Thickness	20 cm	20 cm
Width	5.5 m	5.5 m
Length	91.0 m	91.0 m
Compaction, kg/m <sup>3</sup>		
ASTM D1557	1,682	1,626
TEX 113E	1,425	1,569
Actual	1,649	1,810
Optimum Moisture		
Content, %	13 $\frac{+2}{-3}$	10.5 $\frac{+2}{-2}$
Unconfined Compression		
Strength, kPa		
7 days	3,447	5,860
14 days	3,999	8,618
28 days	4,481	13,789
56 days	5,515	16,547

\*Numbers in parenthesis represent percents of the density achieved by ASTM D 1557

### **Post-Construction Evaluation of 1991 and 1992 Sections**

After their construction, both test sections were subjected to a series of evaluations to assess their in-service structural integrity. Two primary factors govern the long-term performance of stabilized road base materials:

- The ability to withstand shrinkage forces without inducing reflection cracks, and
- The ability to withstand traffic loads without fatigue cracking.

In either case, cracks in the surface permit water to seep into both the base and subgrade, generally accelerating the rate of pavement failure. To monitor the post-construction performance of the experimental sections, investigators used three methods of evaluation:

- Visual inspections to detect any shrinkage or load-associated cracking;
- In-situ structural evaluations using a Falling Weight Deflectometer (FWD) to non-destructively measure base layer strengths and moduli; and
- In-situ structural evaluations under simulated traffic loads using an embedded multidepth deflectometer (MDD) to monitor the pavement's response to repeated heavy axle loads.

#### *Visual Inspection*

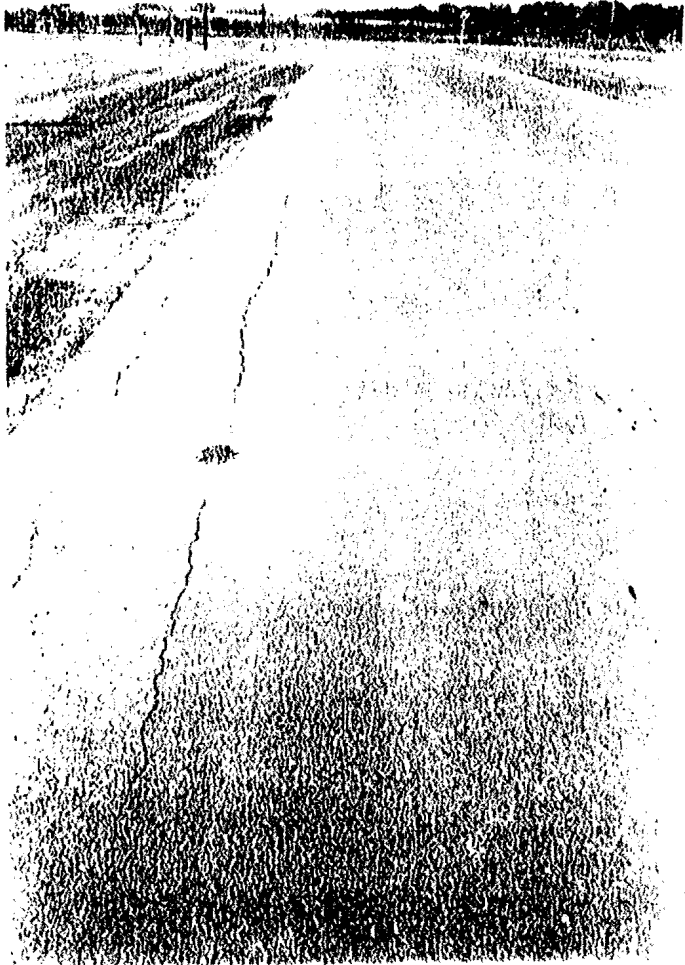
Over the course of their respective post-construction evaluation periods, the experimental test sections were inspected visually for surface cracks or anomalies. Within 2 weeks following construction, a transverse expansive bump, 2 to 3 cm in height, appeared at each end of the 1992 roadway. This phenomenon can occur with FGD materials at transitions or work stoppage points due to variations in compaction between the new and existing pavement or with the accumulation of excessive water pushed along during grading or compaction. It should be noted that no such bump occurred in the 1991 section, which, at

this writing, has been in service for about 5 years. The physical dimensions of these bumps did not change beyond their initial configuration and were easily removed by milling and resurfacing with asphalt. However, it was decided to allow the bumps to remain during the course of the post-construction evaluation to serve as a visual indicator of the need for caution in the proper preparation of these transitional areas during construction.

During the evaluation period (i.e., circa August 1993), longitudinal, hairline cracks appeared 46 cm from the outer edge of the eastbound lane of the 1992 test section (see Figure 3-13a). This was similar in location and nature to cracks that appeared in the 1991 test section shortly after its first summer in service (See Figure 3-13b). Further examination established that these types of cracks were associated with shifts in expansive clay subgrades after a wet season. The cracks were sealed with an asphaltic patching material and remained stable for the remainder of the evaluation period. After more than 5 and 4 years, respectively, the 1991 and 1992 test sections are still in service and performing well.

No cracks or incipient distress patterns were encountered in the 1992 test sections until 21 months into the evaluation period. At that time, a series of "block-type," hairline cracks appeared as shown in Figure 3-14. These crack formations are typical for cement-stabilized bases, especially when placed over expansive subgrade materials. In this case, the subgrade was a Burleson clay, also referred to as "gumbo" or "blue" clay, known for its expansive behavior. The high initial compressive strength (5865 kPa at 7 days) of this base material may also have contributed to these cracks. It is interesting to note that the 1991 test section, whose 7-day strength was closer to 3450 kPa, has not, at this writing, shown any of the "block-type" cracking patterns being revealed in the 1992 section. This is a significant observation and should be incorporated into future design criteria for FGD roadbases.

Based on discussions with several local contractors and state highway materials personnel, the appearance of these types of cracks in a cement-stabilized system do not warrant any immediate remedial action and is not an indication that the road is in the process of progressive failure. This was borne out by Falling Weight and Multidepth Deflectometer data to be discussed later. For this reason, except for filling the cracks with patching material, no further action was deemed necessary.



a. 1992 Test Section



b. 1991 Test Section

Figure 3-13. Longitudinal Cracking Due to Subgrade Expansion.

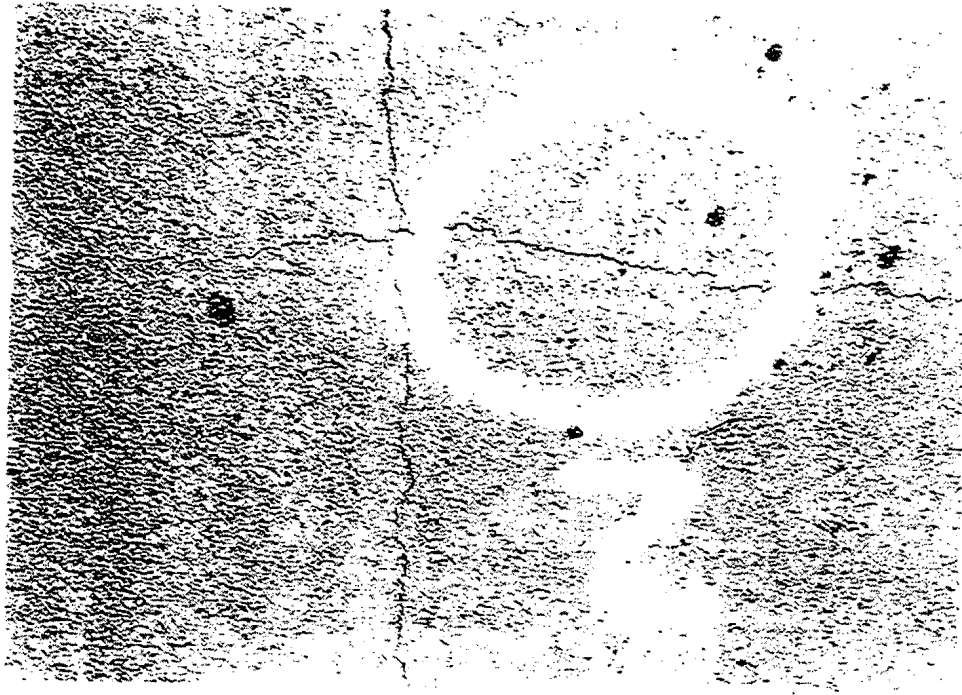


Figure 3-14. Block-Type Crack Pattern Developed in the 1992 FGD Test Section After 22 Months of Service.

### *Falling Weight Deflectometer*

The FWD is one of the more common non-destructive testing devices used for monitoring the structural integrity of pavements. Figure 3-15 shows the FWD used to monitor the two experimental test sections. A load that simulates a truck load of 30 ms duration is applied to the pavement through a 30-cm-diameter load plate. Loads up to 9072 kg can be applied, but usually a load of 4082 kg is used because it simulates a typical legal load limit. The deflection of the pavement is measured at the center of the load plate by six geophones located at 0.3, 0.6, 0.9, 1.2, 1.5, and 1.8 m from the center of the load plate. The seven deflections under a known load produce what is known as a "deflection bowl." The magnitude of the maximum deflection and the shape of the deflection bowl can be used to back-calculate the stiffness of each layer in the pavement system (3.21).

Typically FWD deflection bowls are measured at regular intervals along the length of the roadway so that the strength and its variations can be determined. On both the 1991 and 1992 experimental sections, the deflections were taken soon after construction and at several intervals thereafter. The parameters of interest are the maximum pavement deflection and its variation along the roadway and the back-calculated modulus of the stabilized base layer. The MODULUS 4.2 software (3.21) performed the back-calculation. This involves matching the measured deflection bowl with theoretically generated bowls from a layered-elastic computer simulation. These programs require the test load, the thicknesses, moduli and Poisson ratios of each layer as input from which they compute a theoretical deflection bowl. The programmer changes the layer moduli, from run to run, and an error minimization routine is used to arrive at the final modulus.

Figures 3-16 and 3-17 show the FWD test results from the 1991 and 1992 test sections, respectively. The lower graph in each figure shows the base deflections, whereas the upper indicates the moduli as measured along the length of the test section. The designation "original base" refers to the existing iron ore gravel base.

The deflections in the two FGD sections consistently fell below 10 mils, which represents a good quality base. In contrast, the deflections in the original base ranged from 30 to 40 mils. These deflections remained relatively constant except for one test run in April

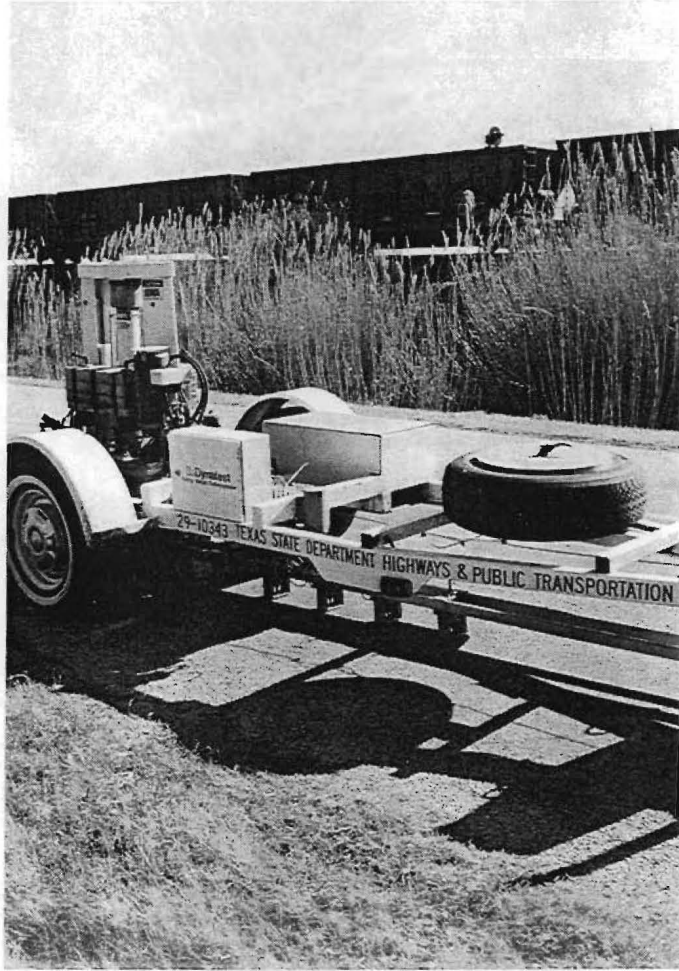


Figure 3-15. Falling Weight Deflectometer Used for Structural Strength Testing.

1991 GYPSUM/BOTTOM ASH TEST ROAD

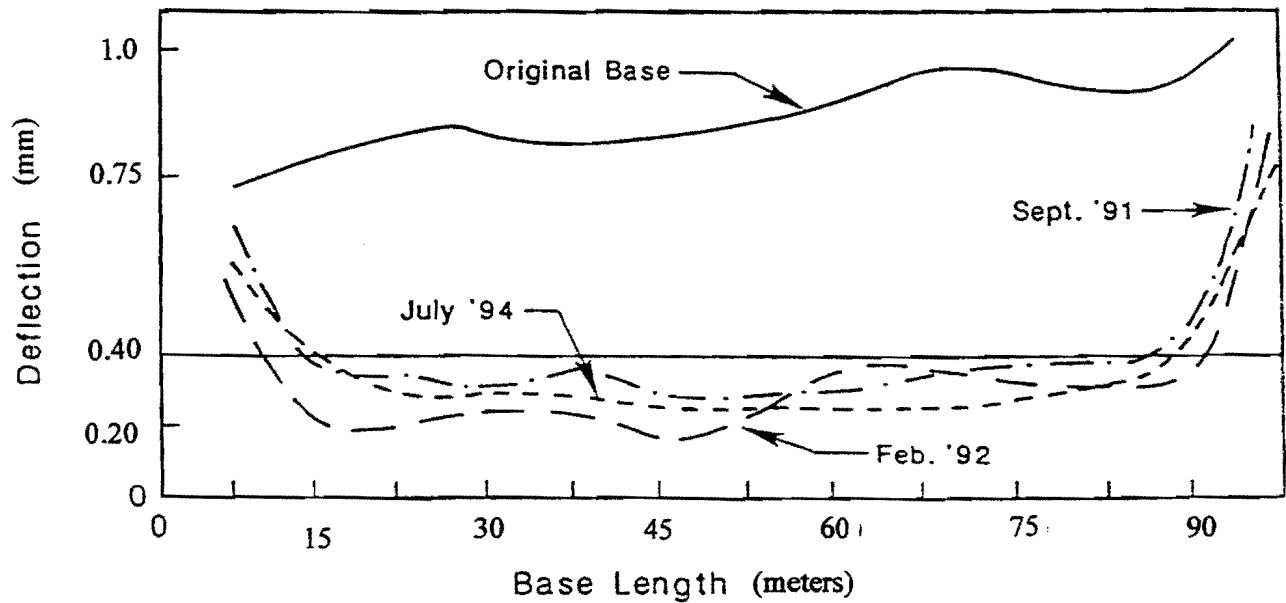
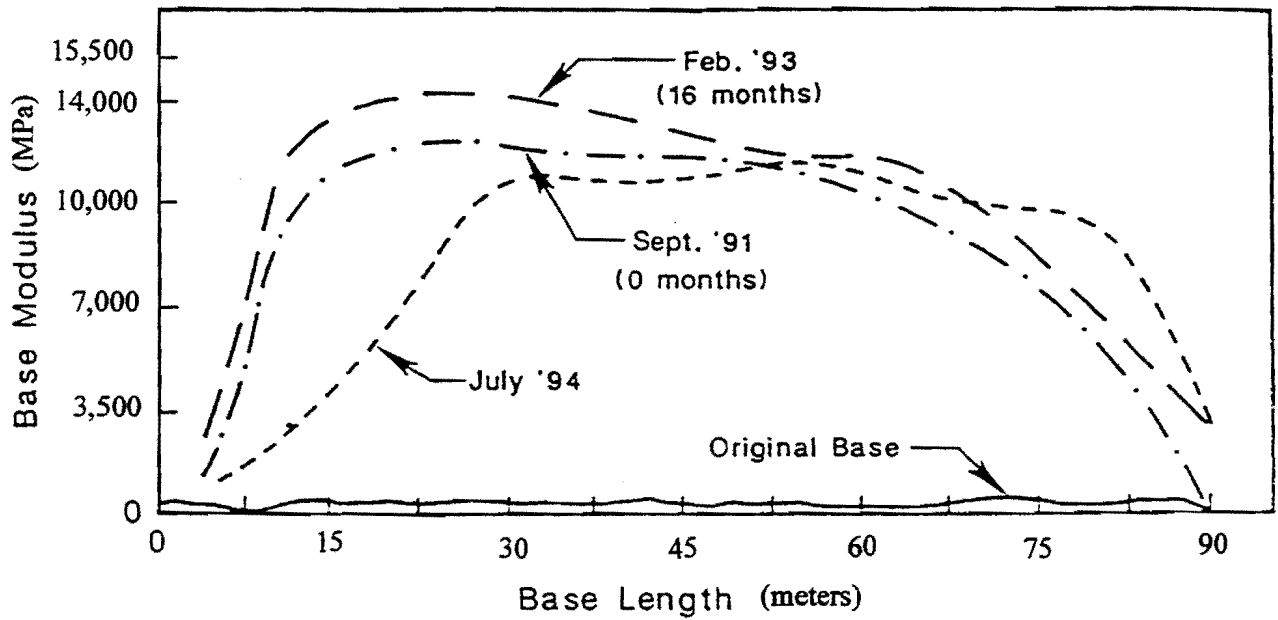


Figure 3-16. Falling Weight Deflectometer Results on 1991 FGD Test Section over 36-Month Post-Construction Evaluation Period.



1992 GYPSUM/BOTTOM ASH TEST ROAD

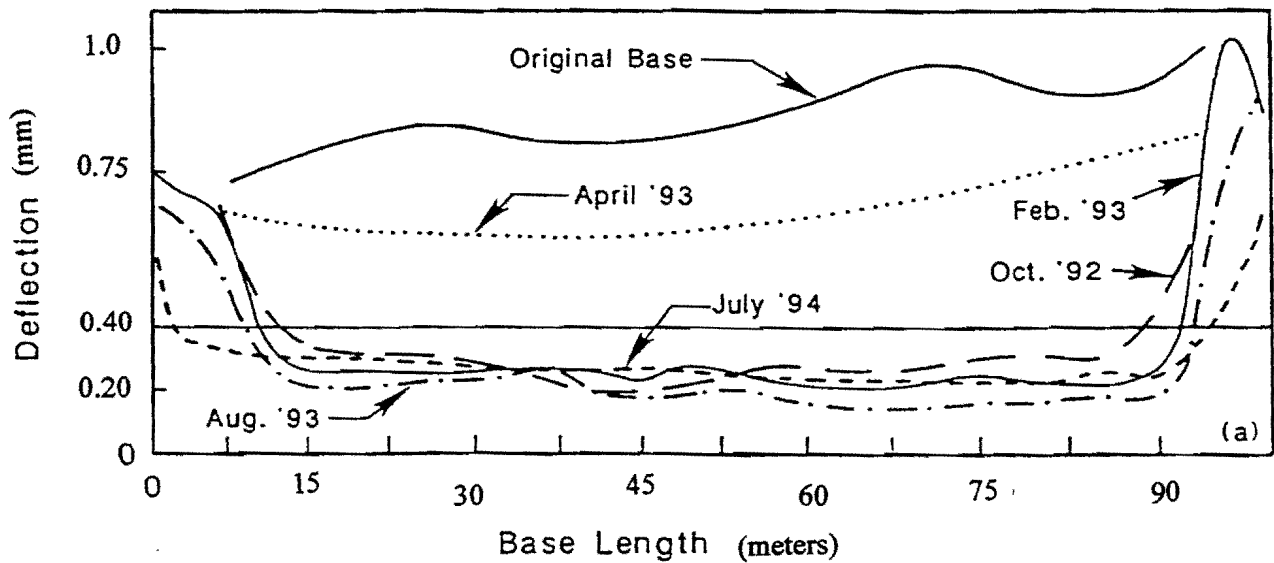
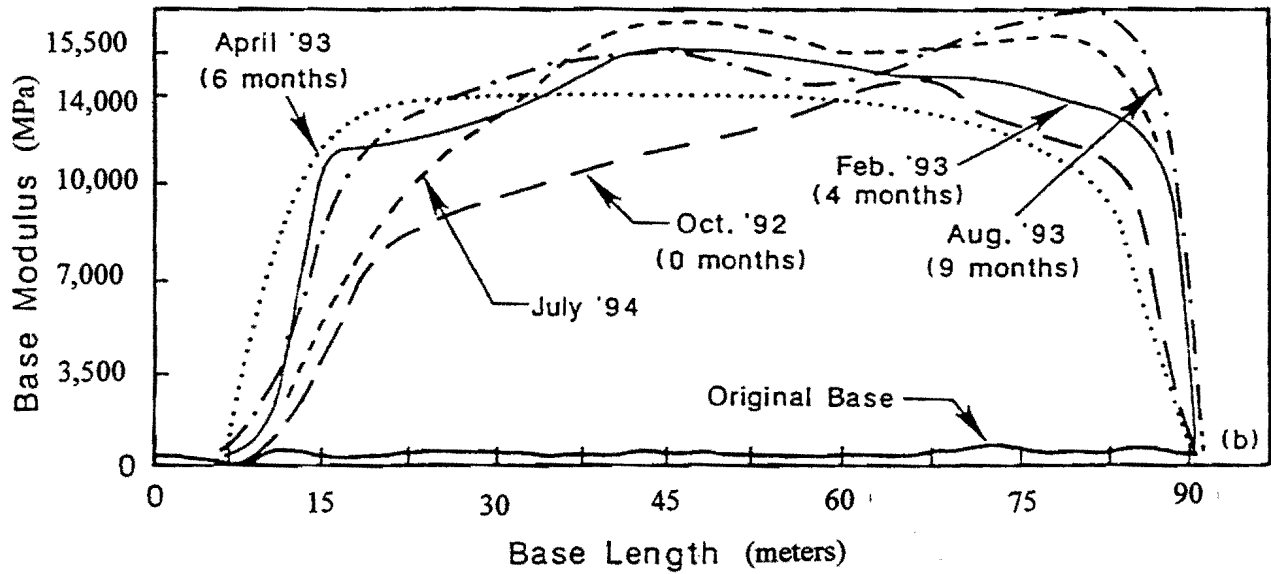


Figure 3-17. Falling Weight Deflectometer Results on 1992 FGD Test Section over 22-Month Post-Construction Evaluation Period.

1993, which showed a dramatic increase in the 1992 section. This measurement was taken right after a prolonged period of rainfall that caused the drainage ditches on both sides of the section to be full of water. The test was rerun in August 1993 and showed that the deflections had returned to pre-April 1993 levels and remained relatively constant for the remainder of program.

The back-calculated base moduli for the two sections are shown in the upper graphs of Figures 3-16 and 3-17. The moduli for the original base ranged from 138 MPa to 276 MPa. The values for the 1991 and 1992 sections appeared to equilibrate at  $13.8 \times 10^6$  kPa, respectively. The change in deflections experienced in April 1993 was not consistent with the drop in the moduli for that period, and the subsequent moduli remained relatively constant throughout the remainder of the program.

In general, low deflections and high moduli would indicate a better ability for the road base to distribute its load over the subgrade and to carry traffic loads without rutting and cracking. The appearance of shrinkage cracks in the 1992 section and not in the 1991 section would suggest an upper limit to this hypothesis. This further suggests that the 7-day compressive strength for designing stabilized FGD mixtures should be in the range of 3102 to 3792 kPa (3.10) to deal with expansive subgrades. The results of the FWD tests show, conclusively, that both test sections appear to have substantially improved load carrying capabilities compared to the original base.

#### *Multidepth Deflectometer Testing*

The most important component of any evaluation of a new roadway material is its ability to stand up to multiple passes of fully loaded trucks. A material that has a low stiffness may be prone to excessive permanent deformation, which eventually results in wheel path rutting. Conversely, a material that is too stiff may fail in flexural or fracture primarily by cracking in the wheel paths. To evaluate the experimental roadway's ability to withstand traffic loads, both the 1991 and 1992 test sections were instrumented with multidepth deflectometers (3.22).

Figures 3-18 and 3-19 show a schematic of the MDD module used for measuring in-situ deflections and a cross-section of a typical MDD installation, respectively. The location of the MDD modules in the two experimental test pavements is shown in Figure 3-20. Scullion (3.22) describes the devices and their use in greater detail.

The MDD is comprised of a series of deflection sensors (LVDTs), which are stacked at various depths in the same hole. The hole size is 32 mm in diameter, and the sensors are usually located close to the layer interfaces. The MDDs are normally installed after construction and measure both the transient deflections and permanent deformations under truck loads within the pavement layers relative to an anchor located at a depth of approximately 3 mm. The MDD system is unique in that:

- It places multiple sensors in a single hole,
- It can be calibrated in place, and
- The sensors are recoverable after testing is complete.

Both test sections were tested with a fully loaded dump truck. The rear axle was weighed at 8165 kg, which is close to the legal limit on Texas state roads. The truck was driven at approximately 40 kph over each instrument. The resulting deflections after 50 truck passes for the 1991 and 1992 sections as measured during July 1993 are shown in Figures 3-21 and 3-22, respectively. Similar traces for each section for measurements taken during July 1994 are shown in Figure 3-23 and 3-24. The testing procedure was identical to the 1991 test except that the axle load was reduced from 8165 to 6078 kg. However, the deflection given in Figures 3-23 and 3-24 were computed to be equivalent to an 8165 kg truck load.

Each line represents the deflections measured at different depths within the structure. The smaller amplitude peak at approximately 200 ms represent the steering axle load, and the larger peaks represent the fully loaded rear axle. The magnitude of the two pulses, marked

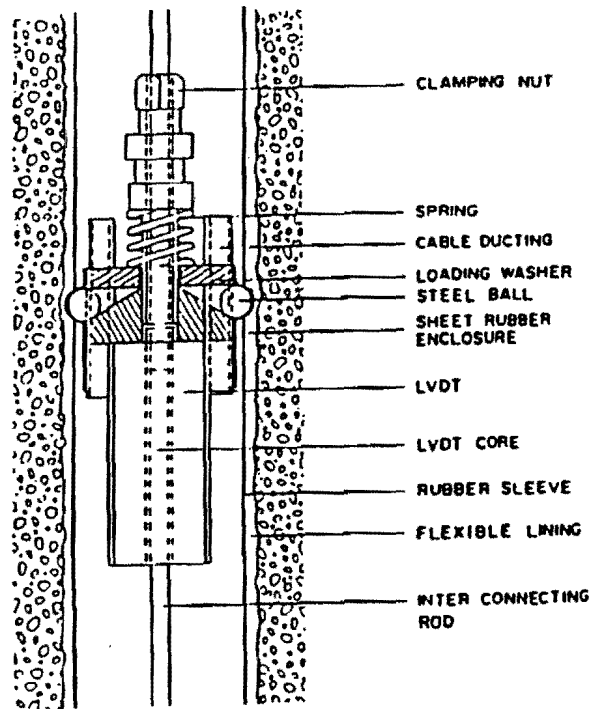
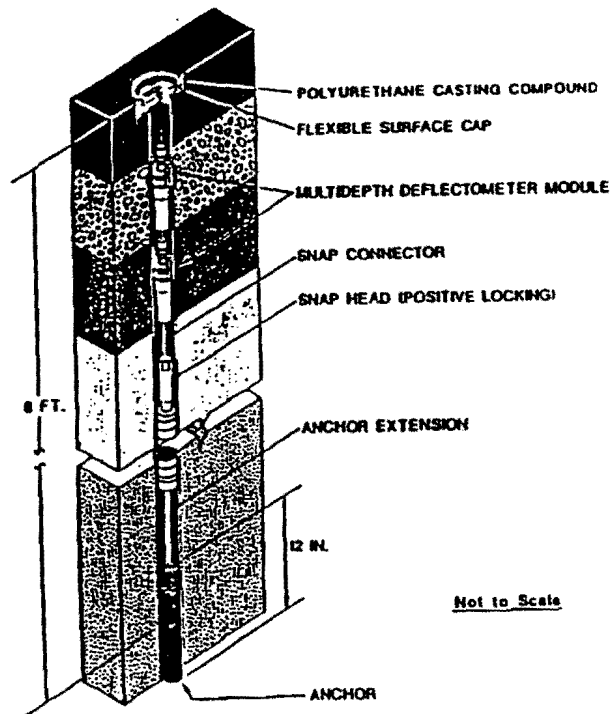
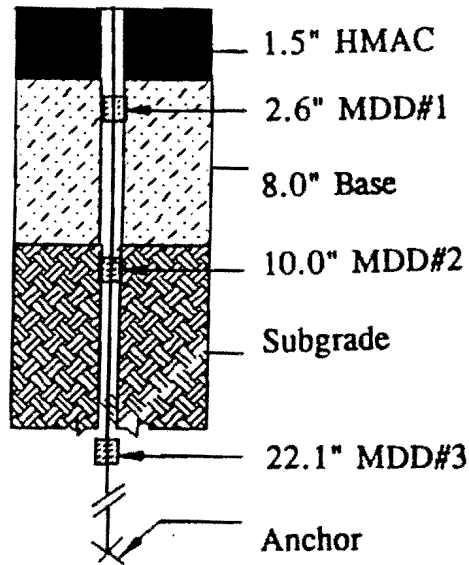


Figure 3-18. The MDD Module.

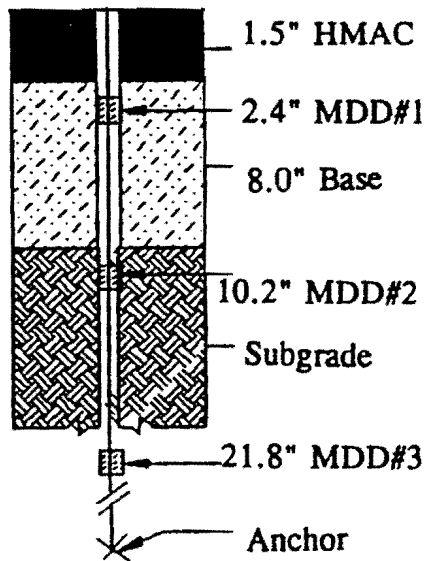


3-19. The Complete MDD System Including Two MDD Modules Installed in a Pavement Structure.



1991 TEST SECTION

☒ Indicates MDD Devices



1992 TEST SECTION

Figure 3-20. As-Placed MDD Installations for 1991 and 1992 Test Sections.

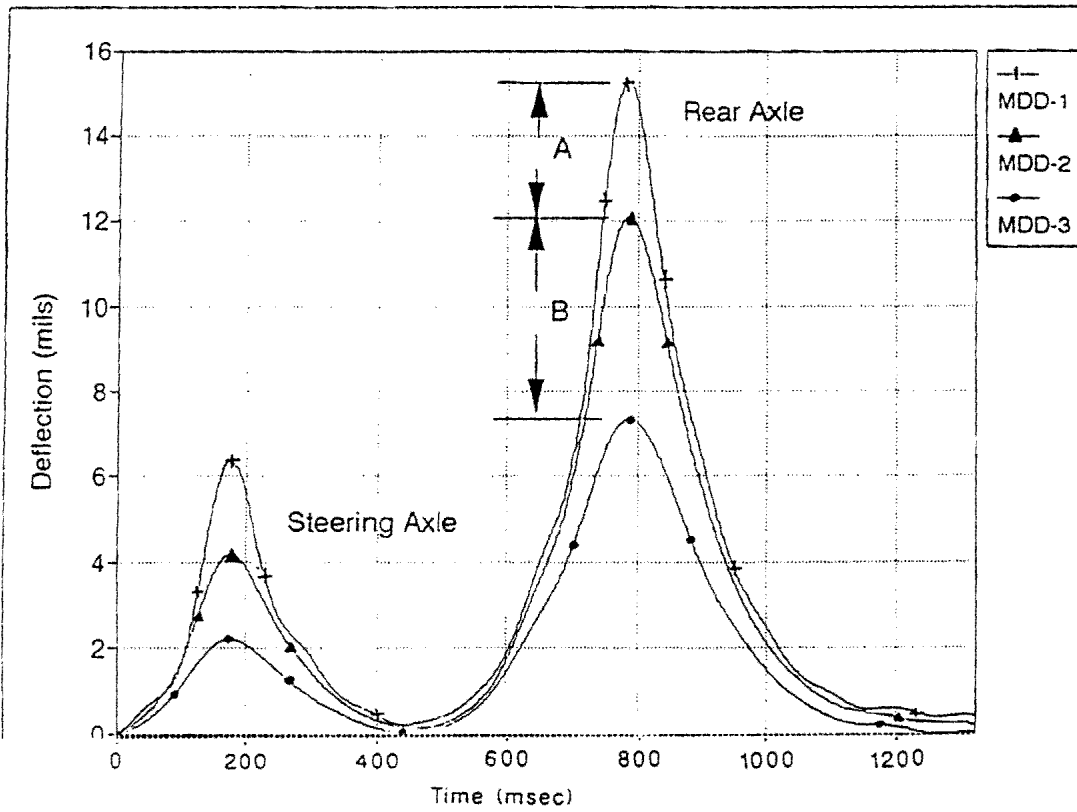


Figure 3-21. MDD Deflections in 1991 Test Section after 10 Months.

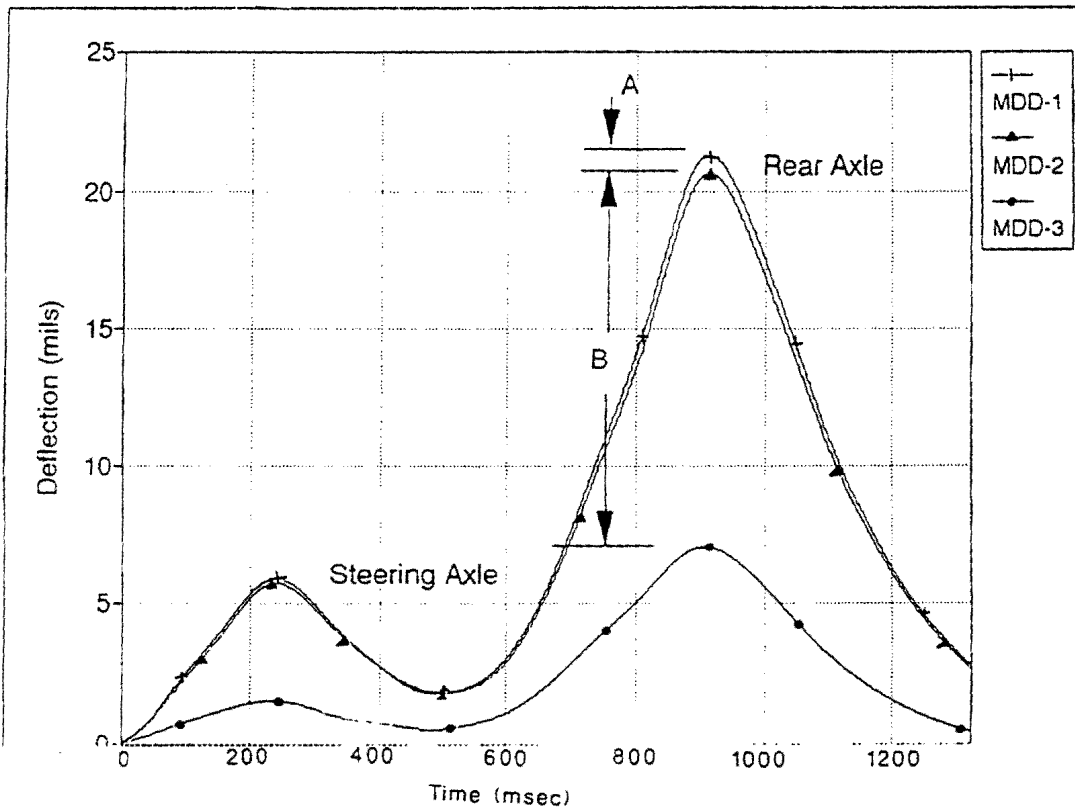


Figure 3-22. MDD Deflections in 1992 Test Section after 10 Months.

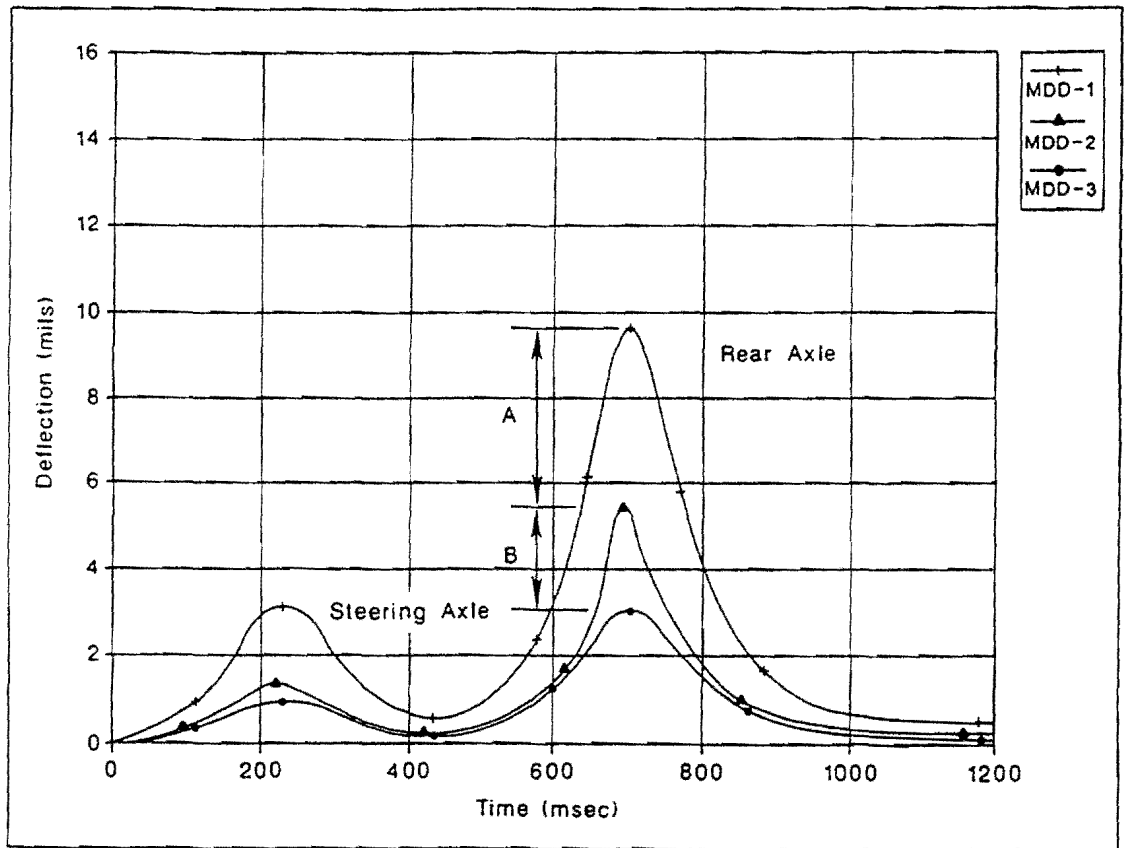


Figure 3-23. MDD Deflections in 1991 Test Section after 22 Months (July 1994).



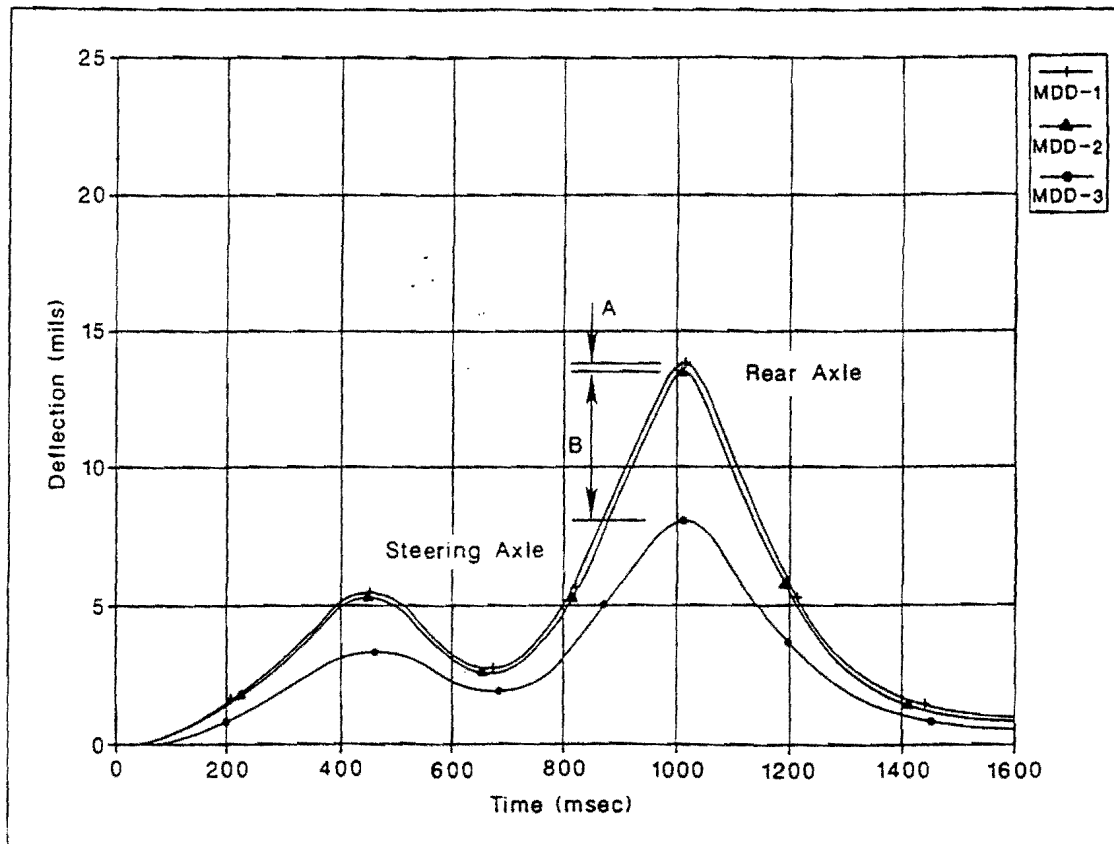


Figure 3-24. MDD Deflections in 1992 Test Section after 22 Months (July 1994).

"A" and "B," can be used to calculate the strain induced in the base and subgrade, respectively, and, hence, provides an indication of the stiffness of each layer. From Figures 3-21 and 3-22, the following can be implied:

1. The A value is substantially smaller in the 1992 test section. This indicates that its stiffness is substantially higher than the 1991 base and remained so during the year between obtaining the two measurements.
2. The B value (or strain) is substantially higher in the 1992 test section. This implies that a weak layer exists within the subgrade just beneath the gypsum/bottom ash road base layer.
3. The amplitudes of the deflections dropped in both sections, indicating that the resistance to traffic loads (i.e., rutting) had improved over the 1-year period between the two tests.
4. The strains in both sections are very low, but the weaker subgrade under the road base of the 1992 section may have contributed to the "block-type" cracking observed in this section, but not in the 1991 roadway. Such a weakness could be temporary in nature and is frequently encountered after excessive rainfall. Similar indications were found in the April 1993 analysis of the FWD data on the two test sections.

In conclusion, after 4 and 5 years, respectively, in service, the 1991 and 1992 sections are structurally sound and showed no significant deterioration under truck loads. This is a further indication that the block cracking in the 1992 section has had only a minimal effect on the integrity of the overall road.

In testing each of these sections, 50 passes of a fully loaded truck were made over each section, and the following information was collected:

1. The change in maximum deflection with number of truck passes;

2. The accumulated permanent deformation after each pass. (This is measured by the final asymptote of MDD 1. This is the residual deformation remaining once the load has passed. In both cases the deformations were close to zero.); and
3. The change in shape of the deflection bowl as measured by a Surface Curvature Index (SCI).

Stabilized layers typically are designed to spread the load over a wide distance. However, if the layer should crack under load, the shape of the bowl will change significantly. To evaluate the shape of the bowl, an SCI parameter was calculated. The SCI is defined as the change in deflection (in mils) from the maximum (i.e., when the MDD is directly under load) to the deflection measured when the load is 305 mm from the MDD. The measured change in maximum deflection with number of truck passes is shown below:

Table 3-10. Multidepth Deflectometer Data.

Number Passes	Max Deflection (mils)			
	1991 Section		1992 Section	
	Date	7/93	7/94	7/93
1	16.66	13.4	21.2	21.9
10	15.87	12.8	21.9	20.1
50	17.55	12.9	21.5	18.4

Investigators observed no significant changes in maximum deflection over the 50 passes. The deflections in the 1992 section were higher than in the 1991 section, initially. However, both sections indicate a stiffening over the 1-year period between the two tests. It is also noteworthy that the July 1994 data for the 1991 section represents maximum deflections after 3 years of service, whereas data for the 1992 section were obtained after 22 months. The variations indicated in the table were probably attributed to wander of the test

truck (i.e., wheels not being directly over the sensor). It can be concluded that no significant reduction in strength has occurred over the post-construction period of the two test sections. The function of a base course is to spread the pavement loads so that no permanent deformations occur in the subgrade layer. The accumulated subgrade strains were monitored by observing changes in MDD 2, which was located at the top of the subgrade. The measured accumulated deformations are tabulated below.

Table 3-11. MDD Accumulated Deflections.

Accumulated Deformation (mils)				
Number of Passes	1991 Section		1992 Section	
Date	7/93	7/94	7/93	7/94
1	0.02	0.02	0.01	0.01
10	0.04	0.05	0.08	0.10
50	0.04	0.04	0.17	0.17

Total accumulated permanent deformations after 50 passes for the 1991 and 1992 sections were 0.04 mils and 0.17 mils, respectively, during July 1993 and July 1994. Very little change occurred within each test section over the 1-year time period. The top of the subgrade on the 1992 site was known to be weak (wet) during this testing. The fact that such low deformations were induced is an indication that both bases are doing a good job of spreading the load and minimizing subgrade damage even over a weak subgrade.

The change in SCI over the 50 passes is also tabulated below. If a layer within the pavement cracks, it is expected that the SCI values will become significantly larger. The results shown above indicate that the truck loads employed in this test did not induce any cracks in the stabilized layers and that the block cracking in the 1992 test section has not adversely affected the integrity of the overall roadway. As reflected in the data discussed above, SCI values for the 1992 test section did not change significantly over the testing period. However, the SCI values for the 1991 section experienced about a 50 percent drop.

### 1993 Stabilized-FGD Gypsite Road Base Test Section (3.19)

#### Materials Selection

**FGD Gypsite:** In the 1991 and 1992 test sections the FGD by-product was calcium sulfate hydrate ( $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$ ) that was produced at TU-Electric/ALCOA Rockdale, Texas, plant. In 1993 Houston Lighting and Power Company sponsored a project utilizing FGD by-products generated at its Parish and Limestone coal-burning electric power generating stations. This material has been referred to as Gypsite in this report and consists of calcium sulfate dihydrate, calcium carbonate, and a solid solution of calcium sulfate and calcium sulfite hemihydrate, the largest ingredient being calcium sulfite. Gypsite slurry is very difficult to handle because of its high water content and texture. It is also difficult to dewater and requires a large storage area. Table 3-12 gives the mineralogical composition of the gypsite materials produced at the Parish and Limestone generating stations.

Table 3-12. Mineralogic Composition of FGD Materials.

FGD Material Source	Gypsum: $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$	Solid Solution: $\text{CaSO}_3 \cdot \text{CaSO}_4 \cdot \frac{1}{2}\text{H}_2\text{O}$	Percentage of $\text{CaSO}_3 \cdot \frac{1}{2}\text{H}_2\text{O}$ in Solid Solution	$\text{CaCO}_3$
W.A. Parish	10.99%	86.14%	73.22%	2.87%
Limestone	16.24%	78.84%	67.02%	4.92%

**Fly Ash:** Two separate fly ashes were used in the production of the final material blends. Parish fly ash results from the burning of Wyoming Powder River Basin subbituminous coal and conforms with ASTM C 618-Class 'C'. Limestone fly ash is generated from Texas lignite and conforms to Class 'F'. Physical and chemical characteristics for these two ashes are listed in Table 3-13 and in the Appendix A (3.23).

Table 3-13. Chemical and Physical Analyses of Fly Ash. (3.23)

Chemical Analysis, %		
	Parish Class 'C' Ash	Limestone Class 'F' Ash
SiO <sub>2</sub>	33.63	57.46
Al <sub>2</sub> O <sub>3</sub>	19.03	18.77
Fe <sub>2</sub> O <sub>3</sub>	6.73	10.80
Sum of SiO <sub>2</sub> + Al <sub>2</sub> O <sub>3</sub> + Fe <sub>2</sub> O <sub>3</sub>	59.93	87.03
MgO	4.82	1.82
SO <sub>3</sub>	2.45	0.23
CaO	27.10	6.80
Moisture Content	≤0.01	≤0.01
Loss on Ignition	0.25	0.21
Available Alkalis as Na <sub>2</sub> O	1.20	0.52
Physical Analysis		
Fineness (Percent Retained on #325 sieve)	17.71%	22.54%
Water Requirement, % Control	94%	95%
Specific Gravity	2.63	2.38
Autoclave Expansion, %	0.03%	0.00%
Strength Activity Index w/Portland Cement, 28 days	96%	80%

**Cement:** A high early-strength, high-sulfate-resistant Class C oil well portland cement used in the 1991 and 1992 test sections was also used as the stabilizer for the Limestone material. Its composition is similar to an ASTM C 150-Type V cement, with fineness characteristics similar to a Type III cement. It was selected because its C<sub>3</sub>A content (2.3%)

was low enough (<4%) to inhibit sulfate attack in mixtures containing FGD by-products. The chemical and physical analyses of this cement was given in Table 3-4.

**Scrubber Base:** The FGD material produced at both the Parish and Limestone Generating Stations is normally blended with fly ash in a ratio that approximates 1:1 by weight. The blending is done primarily to reduce the moisture content of the FGD material and to allow it to solidify for land filling. The blending is accomplished in the material processing building, adjacent to the scrubber systems. After a multistage dewatering process, unblended FGD material is conveyed to a pug mill, where fly ash is added. This material is moved to a temporary stockpile outside by means of a radial arm stacker. It is transported by truck to permanent disposal cells on site. The blended FGD/fly ash material is typically called "scrubber base." Chemical and physical analyses of Parish and Limestone scrubber base are shown in Table 3-14 and in Appendix A.

Table 3-14. Chemical and Physical Analyses of Scrubber Bases (3.23).

Chemical Analysis, %		
	Parish Scrubber Base	Limestone Scrubber Base
SiO <sub>2</sub>	21.37	42.99
Al <sub>2</sub> O <sub>3</sub>	12.5	14.27
Fe <sub>2</sub> O <sub>3</sub>	4.08	8.29
MgO	3.05	1.62
SO <sub>3</sub>	22.56	14.13
CaO	32.41	15.80
Loss on Ignition	4.51	0.85
Physical Analysis		
Maximum Dry Density (ASTM D-698)	1445 kg/m <sup>3</sup>	1510 kg/m <sup>3</sup>
Optimum Moisture (ASTM D-698)	27.0%	21.7%

When the Parish plant's scrubber base is to be utilized, it is typically removed from the permanent disposal cell with a backhoe. Although the Class 'C' fly ash loses some of its cementitious properties in the blending/stockpiling process, it generally retains enough latent strength characteristics to achieve at least 1725 kPa at 28 days (3.23). This was confirmed in laboratory aging studies in which 7-day unconfined compressive strength was measured against age after blending. Figure 3-25 shows that the 7-day strength is approximately 3726 kPa if specimens are made immediately after blending. However, the 7-day strength decreases to 1930 kPa if specimens are made 56 days after blending. This strength is adequate for secondary or utility roads, but it does not meet state specification requirements of 4485 kPa in 7 days.

Virtually none of the Limestone Plant's scrubber base is utilized. The Class 'F' fly ash does little more than dry the FGD material. The fly ash needs a reactive lime component such as portland cement or a high-lime fly ash to activate the pozzolanic properties of the fly ash. Without this reactive component, the unconfined compressive strength of the Limestone ash is typically between 173 to 345 kPa.

### *Mix Design Selection*

Research selected two mix designs were selected from the many possibilities tested in the lab, one for Parish FGD material blends and one for Limestone FGD material blends. Material properties (primarily strength), ease of production, and economy were the principal criteria used in selecting the two mixes. Table 3-15 lists mixture proportions and optimum moisture/density data for the two job mix designs, whereas Figures 3-26 and 3-27 show moisture/density relationships and early age strength development.



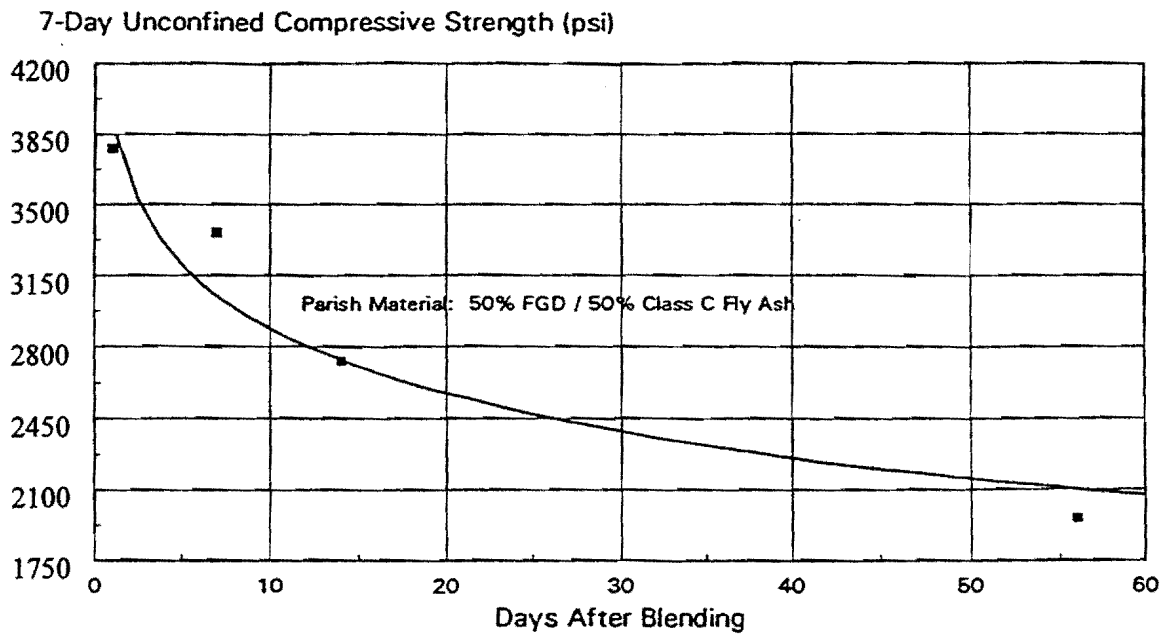


Figure 3-25. 7-Day Unconfined Compressive Strength Versus Age of Blended Material When Specimen Prepared.

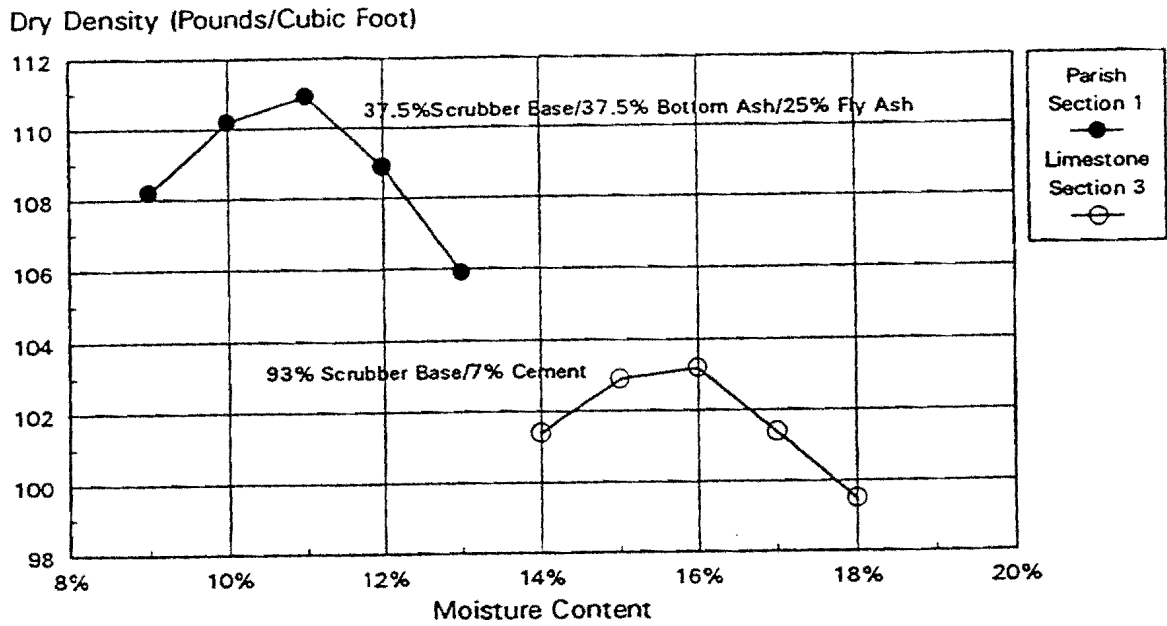


Figure 3-26. Moisture/Density Relationship Parish and Limestone Selected Mix Designs.

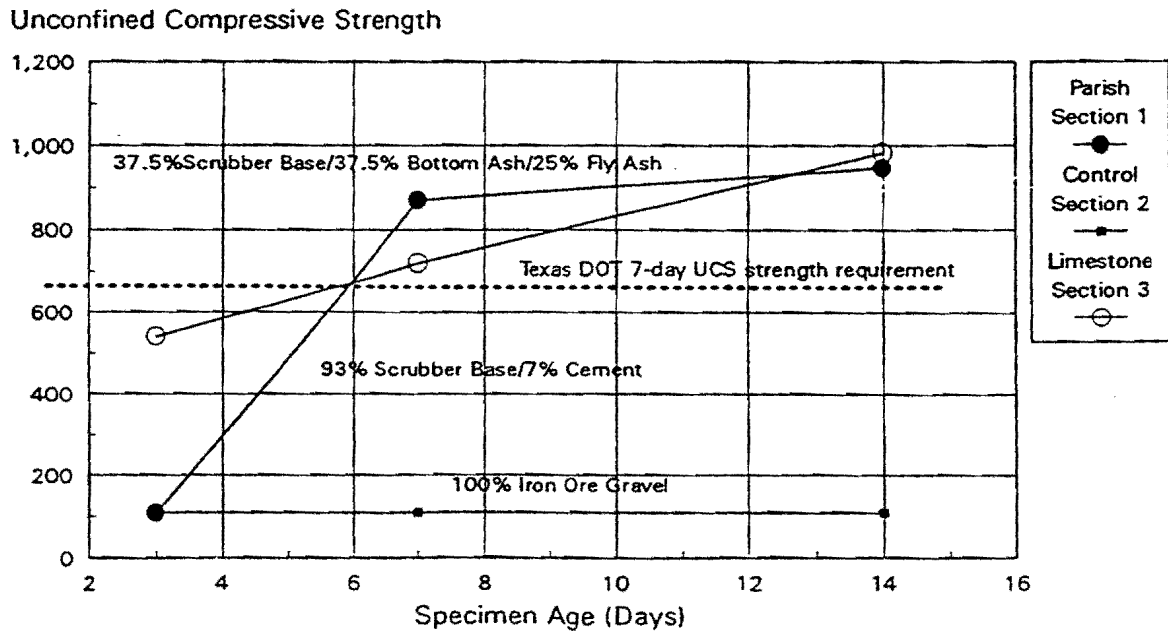


Figure 3-27. Early Age Unconfined Compressive Strength Parish, Control, and Limestone Selected Mix Designs.

Table 3-15. Final Mix Design Constituents, Densities, and Strengths.

Material	Parish (Section 1)	Control (Section 2)	Limestone (Section 3)
Stockpiled Scrubber Base (approx. 1:1 blend of FGD and fly ash)	37.5%	0%	93%
Additional Parish Class 'C' fly ash	25%	0%	0%
Cement	0%	0%	7%
Iron Ore Gravel	0%	100%	
Maximum Dry Density, kg/m <sup>3</sup> (ASTM D-1557A)	1730	2162	1650
Optimum Moisture Content	11%	8.5%	16%

For the Parish materials, fly ash is available on site for blending and thus is more economical to use than any other stabilizer. Also available at the disposal stockpiles is a pug mill that can economically blend dry fly ash with moisture-conditioned scrubber base. The mix design chosen minimized the use of Class 'C' fly ash (a higher value material), and maximized the use of scrubber base, while exceeding targeted strengths.

The Limestone materials, on the other hand, were only able to achieve the necessary strengths with the addition of cement, or Parish fly ash. A mix consisting of only scrubber base and cement was chosen. Investigators made this decision based on the fact that no external pug mill--like the one at Parish--currently exists at the Limestone plant. However, the pug mill located at the Limestone scrubber facility cannot only blend fly ash and FGD material, but it also can add cement. The plant was designed to produce cement or lime-injected scrubber base (although this capability is not used in normal disposal operations).

HL&P and TTI recognized that although the material in the test sections would be mixed in place with a pulver-mixer, competitive commercial production of these materials would be more economical when mixed at the plant site and hauled to the construction location.

### *Construction*

This experimental project entailed three sections constructed at the Texas A&M University Riverside Campus during December 1993 (3.19). Each section was 5.4 m wide, 90 m long and 300 mm in depth. Two sections (Sections 1 and 3) were made from the FGD material blends noted in Table 3-15. The third section (Section 2) utilized the existing iron ore gravel base and was designated as the control. The subgrade material is an expansive Burleson clay with a plasticity index greater than 45; therefore, a 100 mm sub-base consisting of crushed iron ore gravel was placed to minimize subgrade effects. After compaction, each road base was primed with MC-30 at a rate of 0.68 l/m<sup>2</sup> and overlaid with a 3 cm TxDOT Item 340, Type D hot mix asphalt surface course. Figure 3-28 details the typical road cross sections.

The three road sections were excavated to the existing Burleson clay subgrade. Excavated iron ore gravel was placed and compacted as a 10.2 cm subgrade for each test section. On December 8, 1993, the necessary coal combustion by-products were transported to the construction site.

Section 1, using Parish by-products, was constructed in the following manner: Scrubber base and bottom ash in the necessary quantities were deposited from covered dump trucks and spread uniformly over the 90 m section with a motor grader, then mixed in place with a pulver-mixer. A pneumatic tanker equipped with a spreader bar placed Class 'C' fly ash over the entire section. The fly ash was then pulver-mixed into the scrubber base/bottom ash material. A water truck made a series of passes to achieve optimum moisture content, which was continually monitored using the calcuim carbide gas pressure test method and microwave method. After several passes of the pulver-mixer, the section was re-graded and crowned. Initial compaction was accomplished with a sheep's foot roller. A 13.6 Mg steel drum roller was used for secondary compaction followed by a pneumatic-tire compactor for final compaction.

Section 2, the control section, required no mixing, since it consisted of iron ore gravel alone. The material was graded and compacted in a manner identical to Section 1. MC-30 was applied to Sections 1 and 2 13 days after construction.

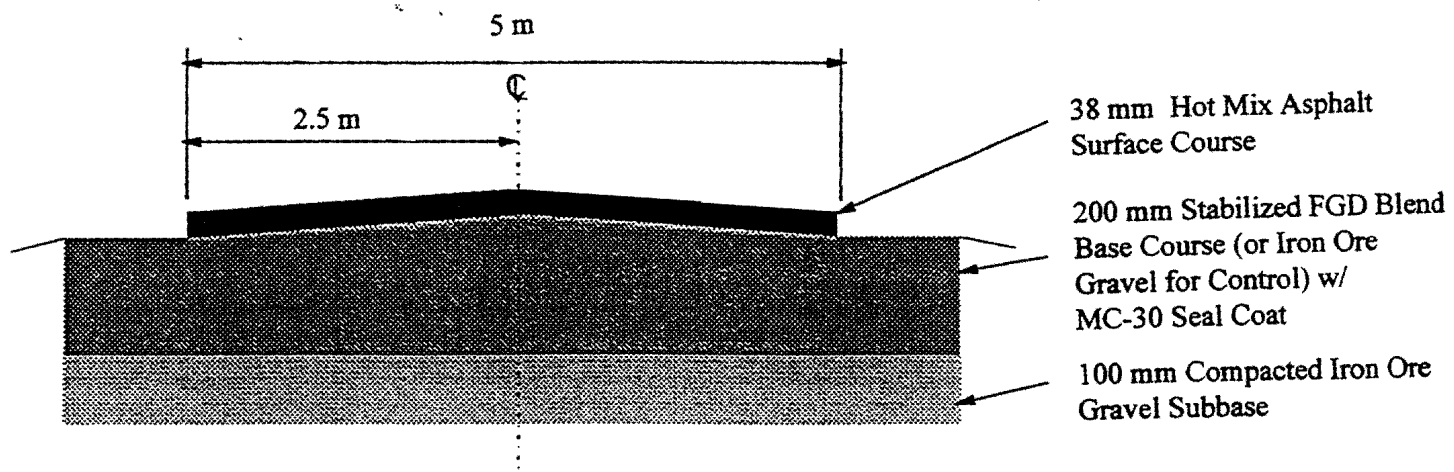


Figure 3-28. Typical Cross Section of Experimental Roadway.

The Section 3 Limestone plant scrubber base was delivered with a moisture content of 19 percent, somewhat above optimum moisture content of 16 percent. Workers made several attempts to reduce moisture content by aerating the material with the pulver-mixer, but a light, steady shower began, which made further moisture reduction impossible. (The control and Parish sections had been placed, compacted, and finish-graded by the time the rain began and thus were not significantly affected.) The decision was made to complete the section when drier weather would allow proper mixing and compaction. Construction of this section resumed on December 17, 1993. Weather conditions in the interim period were rainy and thus did not allow sufficient drying of the scrubber base. TTI researchers and HL&P jointly decided to incorporate additional dry Class 'F' fly ash from Limestone to lower moisture content to the optimum level. Therefore, the fly ash content of the scrubber base, which originally was approximately 50%, was now increased to 60%. The fly ash, deposited with a pneumatic tanker equipped with a spreader bar, was pulver-mixed into the scrubber base. The final moisture content was close to optimum. At this point, Class 'C' oil well cement was placed with a spreader bar and pulver-mixed into the material. The road base was graded and crowned to elevation. Shortly after grading, an unexpected rain shower began and lasted the remainder of the day. Surface tackiness of the base material allowed use of only the pneumatic roller for compaction, which was performed while the rain was falling. Section 3, therefore, was placed under very unfavorable weather/moisture conditions, and compaction was not performed in as comprehensive a manner as for Sections 1 and 2. There was concern that the integrity of the materials might have been compromised. It was decided that any construction-incurred damages would be revealed during post-construction evaluation.

### **Post-Construction Evaluation of 1993 Test Sections**

A post-construction evaluation of the stabilized FGD Gypsite and control test sections was conducted along the same lines used in the 1991 and 1992 sections, using visual inspection and FWD measurements. Since MDD sensors could not be made available in time for installation in these sections, Dynamic Core Penetrometer (DCP) data were generated. The results of each of the tests are discussed below.

### *Visual Inspection*

As of this writing, all three test sections appear sound with no base course-induced surface cracks evident. Several transverse, hairline cracks--common in soil-cement bases--appeared at approximately 6 m intervals in Section 1 before the surface treatment was applied. These cracks could be an indication that the sections may have been overstabilized, resulting in shrinkage cracking. Many of them disappeared shortly after some of the construction equipment was driven over the unpaved sections. This phenomenon had been experienced in other stabilized gypsum road bases and might indicate that the FGD materials are capable of autogenous healing. This would be a significant long-term performance benefit that is not normally available in most cement-stabilized soil bases.

### *Dynamic Cone Penetrometer*

Investigators performed a series of DCP tests on the three base sections prior to laying the asphalt surface course, primarily because of concerns regarding the integrity of Section 3. A DCP test consists of dropping an 8.2 kg hammer on a spike with graduated markings. The relationship between number of blows and depth of penetration (in mm) is plotted, and the slope ( $n$ , in blows/mm) generated. The California Bearing Ratio is related to the DCP slope, and Texas Triaxial Classification (TTC) ratings can be derived from the CBR values (3.24). Table 3-16 shows the results of the DCP test run December 22, 1993 (14 and 5 days after placement of the Parish and Limestone sections, respectively), and February 10, 1994, about 60 days after placement. The December 22 data depict the low relative early-age strengths of the control and Limestone sections compared with the Parish section. However, the February 10 tests indicate that the strength of the Limestone section, after curing for nearly 60 days, had approached that of the Parish section. Additionally, Table 3-16 illustrates that the CBR and TTC data derived from the DCP results indicate that Sections 1 and 3 qualify as "High Volume" road bases, whereas the Control is classified as a borderline "Low Volume" road base or "Subbase."

Table 3-16. Dynamic Cone Penetrometer Results on 1993 Stabilized Road Base Test Sections.

Section	December 22, 1993				February 10, 1994			
	Age*	Slope, n, mm/blow	CBR	TTC**	Age*	Slope, n, mm/blow	CBR	TTC**
1. Parish	14	1.58	174.0	1 +	64	1.76	154.6	1 +
2. Control	14	5.28	45.4	2.8	64	3.75	66.5	2.4
3. Limestone	5	5.27	45.4	2.8	55	1.83	148.4	1 +
	CBR		TTC		Quality			
	150+		1		High Volume Traffic			
	85		2		Low Volume Traffic			
	38		3		Subgrade			
	14		4 and below		Poor			

\* Days after stabilization/compaction

\*\* Texas Triaxial Classification (TTC)

#### *Falling Weight Deflectometer*

FWD data from the three test sections are shown in Table 3-17. Mean deflections in the two FGD sections (Sections 1 and 3) were 7.7 and 7.8 mils, respectively. The control (Section 2) exhibited a mean deflection of 37.7 mils. The mean base moduli results for FGD Sections 1 and 3 are 2253 ksi and 2227 ksi, respectively. The control yielded a mean base modulus value of 28.3 ksi. From these deflection and moduli values it is readily apparent that the FGD sections exhibit stiffness about two orders of magnitude greater than the iron ore gravel control. This is considered to be extremely stiff and equivalent to the strength found in typical cement-stabilized bases.



Table 3-17. Falling Weight Deflectometer Average Results.

Section	Base Modulus		Deflections	
	Modulus (ksi)	Std. Dev.	Deflection (mils)	Std. Dev.
1. Parish	2253	371	7.7	0.7
2. Control	28	6	37.7	4.2
3. Limestone	2227	359	7.8	0.7

	SCI (mils)			
	1991		1992	
	7/93	7/94	7/93	7/94
Number of Passes				
1	5.31	2.08	2.2	2.13
10	4.83	2.21	2.1	1.46
50	5.57	2.71	1.8	1.39

From the above discussion, the researchers concluded that the integrity of the 1991 and 1992 road base test sections are excellent after 36 and 22 months of service, respectively, and that neither had sustained any noticeable damage during the application of the simulated traffic loads.

### Environmental Impact Analysis

Inasmuch as CCBP materials are by-products, they contain some of the chemical constituents of the coal sources as well as the process that produced them. To assure safety for construction workers as well as for the public, these materials need to be evaluated for their potential impact on groundwater and drinking water systems. This is normally satisfied through EPA established Toxicity Characteristic Laboratory Procedures (TCLP). In addition, Texas Natural Resource Conservation Commission (TNRCC) has established its own procedures and maximum contaminant levels (MCL) for the State.

As part of the activity under "Task A Literature Survey," investigators obtained a composite of the TCLP and TNRCC data taken on CCPB materials from a number of power stations in Texas. Appendixes C and D provide a statistical summary of the tests results for fly ash, bottom ash, and FGD materials produced at these plants. Also included are TNRCC maximum allowable contaminant levels to comply with leachate, secondary drinking water (SDW) and primary drinking water (PDW) standards. Where test results exceed those allowable, the materials fall into a Class 2 category. Those that fall below TNRCC standards can be classified in the non-toxic Class 3 category.

The results shown in Appendixes C-1, C-2, and C-3 indicate that none of the CCBP materials tested toxicity levels to warrant a Class 1 (most toxic) designation. As will be discussed later, the TCLP analyses of leachate and groundwater samples taken in conjunction with the post-construction evaluation of the 1991, 1992, and 1993, test sections containing CCBP materials tend to support the data shown in Appendixes C and D. Additionally, the EPA has designated CCBPs as non-hazardous materials and promulgates the use of CCBPs whenever technically and environmentally appropriate. (3.25)

### *Test Descriptions*

Throughout the post-construction phase program, sampling and soil analyses were generated in the vicinity of the job sites to provide a data base for evaluating the environmental impact associated with the use of CCBP road base mixture. These tests were also designed to determine the type and amount of testing that would be needed should a similar environmental impact of gypsum road bases have to be evaluated on state and federal highway systems. This program involved the following tasks:

**Leachate Characterization:** Leachates were generated by an equilibrium batch extraction, slurry technique. Mix ingredients, soils, and their various combinations were analyzed for the chemical constituents and summarized along with their respective test methods in Table 3-18. The purpose of this effort was to identify a list of indexing parameters to be monitored during the post construction period.

Table 3-18. Leachate Chemical Analysis and Test Methods.

Parameter	Method
Aluminum	7020*
Arsenic	7061*
Barium	7080*
Cadmium	7130*
Calcium	3151**
Chromium	7190*
Copper	7210*
Iron	7380*
Lead	7420*
Manganese	7460*
Mercury	7471*
Selenium	7741*
Silica	370.1**
Silver	7760*
Sulfur	ASTM D-2492
Total Carbon	29-2.3***
Total Inorganic Carbon	11-2.3***
Total Organic Carbon	29-3.3***
Zinc	7950*

- \* Test Method for Evaluating Solid Wastes. Physical/Chemical Methods, EPA, Office of Solid Waste, SW-846, November, 1986, March Edition, Revised.
- \*\* Methods for Chemical Analysis of Water and Wastes. EPA-600/4-79-02, Revised March, 1983.
- \*\*\* Methods of Soil Analysis, Part 2, Chemical and Microbiological Properties, A. L. Page (ed.) Second Edition, Argon No. 9, Amer. Soc. Argon, Madison, Wisconsin, 1982.

All test materials were extracted at solid: solution ratios of 1:5, 1:20, and 1:100 at 24 and 72-hour equilibration times. Extract component concentrations that reflected simple source/dilution effects were evaluated by serial batch extraction techniques to identify any changes in leachate composition with succeeding portions of water extract. Changes in the soil's capacity to remove and/or enhance test constituents were also evaluated.

**Unsaturated Zone Moisture Cups:** Vacuum extraction soil moisture cups monitored subsurface migration of chemical constituents potentially released after construction and weathering of the test pavement. Groundwater at the test site was not monitored because it was too deep to be influenced by the road base construction activities within the 2-year duration of the project and would therefore be of little value. Soil moisture cups (see Figure 3-29) provide an alternative means of collecting interstitial fluids percolating through the soil profile consistent within the post-construction time frame of the project. Six, 250 ml soil moisture cups were installed, in duplicate, 3.04 m below the surface in the unsaturated zone along the roadway locations shown in Figure 3-30. The large 250 ml capacity cup was used to provide a sufficient collection volume for analysis of the indicator parameters identified in Table 3-19. Samples were taken immediately following the completion of road construction, 0 months, then at 3, 6, and 20 months during the post-construction evaluation.

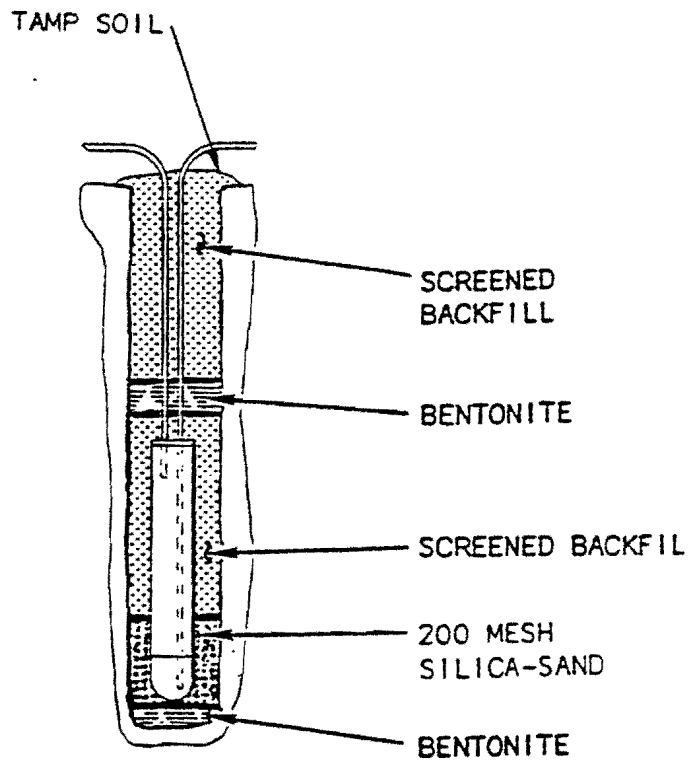


Figure 3-29. Schematic of Vacuum Extraction Soil Moisture Cup.

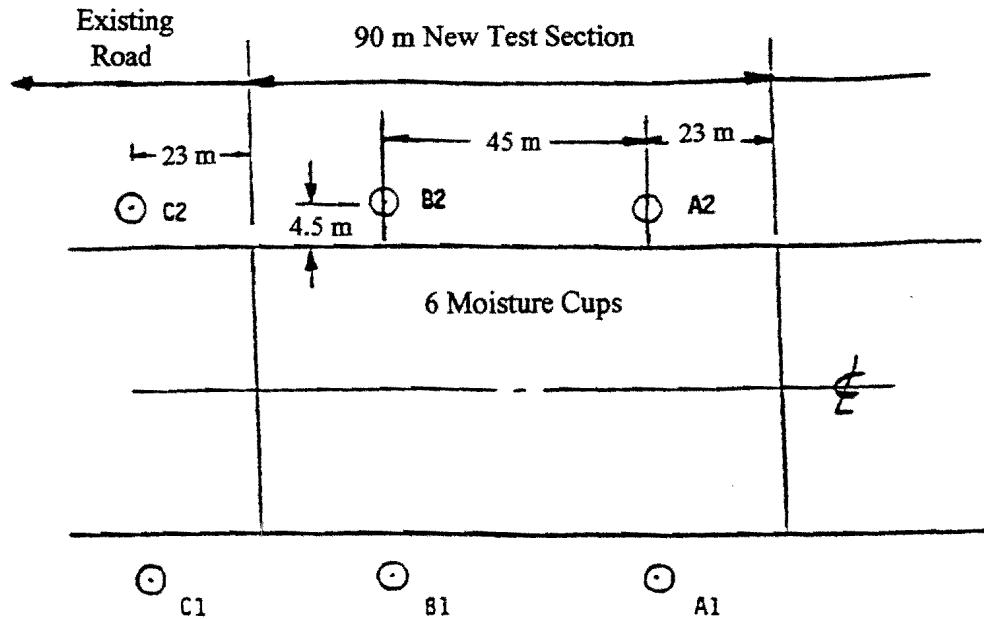


Figure 3-30. Location of Soil Moisture Collection Cup (Typical for 1991, 1992, and 1993 test sections).

Table 3-19. Surface and Groundwater Analysis Parameters and Test Methods.

Parameter	Frequency	Method
pH	M	150.1*
Conductivity	M	120.1*
Aluminum	Q	7020*
Arsenic	M	7061*
Barium	Q	7080*
Boron	Q	25.3*
Cadmium	Q	7130*
Chromium	Q	7190*
Copper	Q	7210*
Iron	M	7380*
Lead	Q	7420*
Magnesium	M	242.1**
Mercury	Q	7471*
Selenium	M	270.3
Silica	Q	370.1**
Silver	Q	7760*
Sulfates	M	375.4**
Total Carbon	Q	29-2.3***
Total Inorganic Carbon	Q	11-2.3***
Zinc	M	7950*
Total Dissolved Solids	M	209B****
Calculated Hardness	M	314A****
Total Suspended Solids (Surface Monitoring Only)	M	209C****

\* Test Methods for Evaluating Solid Wastes. Physical/Chemical Methods. EPA Office of Solid Waste, SW-846, November, 1986, March Edition, Revised.

\*\* Methods for Chemical Analysis of Water and Waste, EPA-600/4-79-020, Revised March, 1983.

\*\*\* Methods of Soil Analysis, Part 2, Chemical and Microbiological Properties, A. L. Page (ed.) Second Edition Agron No. 9, Amer. Soc., Agron, Madison, Wisconsin, 1982.

\*\*\*\* Standard Methods for the Examination of Water and Wastewater, 16th Edition, AP HA, AWWA, WPCF, Revised, 1985.

**Surface Water Collection and Analysis:** Surface waters were collected along the drainage ditch adjacent to the test site. Sample frequency and parameters analyzed corresponded with groundwater monitoring activities discussed earlier. During the course of the program, surface water and leachate could not be obtained due to insufficient sample volumes. The exception was found in the samples from the 1993 control section. These results indicated a level of the  $H_2^{++}$  irons which were leached out of the iron ore panel base.

### **Product Testing and Soil Leachate Analysis**

Laboratory analyses were conducted on cement, gypsum, 2-bottom ashes and the stabilized-FGD gypsum/bottom ash job mix, along with soil leachates and surface runoff to determine if they produced any run-off substances that could have a negative effect on the environment. Laboratory analyses of the four ingredients of the roadbase mixture were conducted for leachable metals, sulfur and carbonates. Soil leachates and surface waters were collected and analyzed for metals, salts, sulfates and carbon.

### *Stabilized-FGD Gypsum/Bottom Ash Mixtures*

Four of the components that were incorporated into the test section were analyzed individually. In addition, a loose compactor and a laboratory sample of the road base mixtures were also analyzed. Leachates were evaluated by means of equilibrium batch extraction, slurry technique, and the results are shown in Table 3-20. The materials were extracted at solid-to-solution ratios of 1:5, 1:20, and 1:100 for 24- and 72-hour equilibration times. TCPL protocol requires a 1:20 acidic extraction for 18 hours.

Values from leachate tests for the job mix do not approach the maximum allowable limits for TCLP. Most of the metals were even below the instrument detection limits. It is reasonable to assume, on the basis of these results, that neither the mixture nor its components represent any adverse threat to the environment.

Table 3-20. Leachate Characteristics of Job Mix and Individual Components.

Component	EPA Drinking Water Standards (mg/l)	EPA TCLP Standards (mg/l)	1:20 Leachate, 24 Hour Extraction				
			Dry Bottom Ash	FGD Gypsum & Gypsite	Cement	Wet Bottom Ash	Job Mix
As	0.05	5.0	<0.01	<0.01	<0.01	<0.01	<0.01
Ba	1.0	100.0	<0.50	<0.50	<0.50	<0.50	<0.50
Cd	0.01	1.0	<0.01	<0.01	<0.01	<0.01	<0.01
Cr	0.05	5.0	<0.05	<0.05	0.125	<0.05	<0.05
Pb	0.005	5.0	<0.05	<0.05	<0.05	<0.05	<0.05
Hg	0.002	0.2	<0.001	<0.001	<0.001	<0.001	<0.001
Se	0.01	1.0	<0.01	<0.01	<0.01	<0.01	<0.01
Ag	0.05	5.0	<0.05	<0.05	<0.05	<0.05	<0.05

3-70



**Soil Leachate and Surface Water Analysis:** Vacuum extraction soil moisture cups measured subsurface migration of chemical constituents during construction and weathering of the test pavement. Soil moisture cups consist of a sealed porous ceramic cup. Polyethylene tubing connects the cup to the surface. A vacuum pressure is applied to obtain a suction, which can range from 50 to 80 centibar. Any moisture in the soil will be collected by the cups. The water sample is then retrieved by applying pressure to one of the surface tubes, forcing it out a second tube.

Shortly after the test sections were constructed, Soil Analytical Services, Inc., of College Station, Texas, installed 30 (six for each of the five test sections), 250 ml moisture collection cups at a depth of 3 m. The cups were located in the drainage ditches on each side of the roadway about 4.6 m from the road bed with a distance of 45 m between collection sites (see Figure 3-29). Sites A and B are located along the FGD gypsum by-products road base. Site C served as the control and was located along the existing road 23 m from the experimental road base to serve as a background sample for reference. A vacuum pressure of -60 centibars was applied to each moisture cup.

Soil samples were taken at each texture change. The texture changes were at approximately 0-0.3 m for clay loam, 0.3-1.8 m, and 1.8-2.4 m for sandy clay loam, and 2.4-3.0 m for fine sandy loam to fine sand. The soil samples were analyzed only if there was an indication of contamination. As of this writing there was no indication of any such contamination. The soil remained very dry at 3.0 m. Water was difficult to collect from any of the four FGD road base sites because of the severe textural changes that occur in the soil profile.

Water samples were collected from the surface and the soil moisture cups the first week of each month during November 1992 (0 months), February 1993 (3 months), January 1994 (9 months), and July 1994 (18 months). Table 3-21 and 3-22 show the average test results for surface water and leachate analyses. As of July 1994, the vacuum extraction soil moisture cups were still holding a vacuum pressure. At this writing, the analytical results indicate there should be no detrimental environmental effects to the groundwater caused by the cement- or fly ash-stabilized FGD road bases, and there is no reason to expect any problems with future sampling.

Table 3-21. Surface Water Analysis of Stabilized FGD Road Base Test Section.

Component	Allowable		Date of Sampling			
	EPA	TCLP				
	Drinking Water mg/l	mg/l	0 (1) 11/92	3 (2) 2/93	9 (3) 1/94	18 (3) 7/94
Aluminum	0.20	N/R		<0.05	<0.08	<0.05
Arsenic	0.05	5.0	<0.01	<0.01	<0.01	<0.01
Barium	1.0	100.0	<0.50	<0.50	<0.50	<0.50
Cadmium	0.01	1.0	<0.01	<0.01	<0.01	<0.01
Chromium	0.05	5.0	<0.05	<0.05	<0.05	<0.05
Copper		N/R		<0.05	<0.05	<0.05
Iron	0.30	N/R		0.18	<0.001	<0.001
Lead	0.05	5.0	0.38	<0.05	0.03	<0.41
Manganese	0.05	N/R		0.04	<0.0002	<0.0012
Mercury	0.002	0.2	<0.001	<0.001	<0.001	<0.001
Selenium	0.01	1.0	<0.01	<0.01	<0.01	<0.01
Silver	0.05	5.0		<0.05		
Zinc	5.0	N/R		1.92	1.90	1.87
Sulphate	250	N/R	0.9	1.2	0.8	0.7
Total Diss. Solids	500	N/R	45.5	54.6	209	48.0

N/R: Not Required.

Data

(1) Collected on 1991 test section only

(2) Average values of data taken in 1991 and 1992 test sections

(3) Average values of data taken in 1991, 1992, and 1993 test sections

Table 3-22. Soil Leachate Analysis of FGD Stabilized Roadbase Test Section. (3.5)

Component	Allowable		Date of Sampling			
	EPA	TCLP				
	Drinking Water mg/l	mg/l	0 (1) 11/92	3 (2) 2/93	9 (3) 1/94	18 (3) 7/94
Aluminum	0.20	N/R	<0.05	<0.<	<0.16	<0.05
Arsenic	0.05	5.0	<0.01	<0.01	<0.01	<0.01
Barium	1.0	100.0	<0.50	<0.50	<0.17	<0.50
Cadmium	0.01	1.0	<0.01	<0.01	<0.002	<0.01
Chromium	0.05	5.0	<0.05	<0.05	<0.003	<0.05
Copper		N/R				
Iron	0.30	N/R	<0.04	0.18	<0.017	<0.19
Lead	0.05	5.0	<0.05	<0.05	0.025	<0.05
Manganese	0.05	N/R	0.03	0.02	0.009	0.009
Mercury	0.002	0.2	<0.001	<0.001	<0.002	<0.001
Selenium	0.01	1.0	<0.01	<0.01	<0.01	<0.01
Silver	0.05	5.0	<0.05	<0.05	0.023	<0.05
Zinc	5.0	N/R	0.024	0.026	0.007	0.009
Sulphate	250	N/R	0.81	2.98	81.6	7.2
Total Diss. Solids	500	N/R	256	Trace	396	Trace

N/R: Not Required.

Data

(1) Collected on 1991 test section only

(2) Average values of data taken in 1991 and 1992 test sections

(3) Average values of data taken in 1991, 1992, and 1993 test sections

Water samples from the soil moisture collection cups were taken on November 23, 1992, after a heavy rainfall. Water was retrieved only from the C1 site. The other sites were holding a vacuum suction, but apparently not enough rainfall ever occurred to wet the soil to a depth of 3.0 m. Surface water samples were also collected from the drainage ditches where the moisture collection cups were installed. The vacuum pressure on the moisture cups was increased from -60 to -80 centibars to aid in water collection. Again, no water was found in any of the cups. Due to the fact that there is 1.8 meters of clay/clay loam texture above the sand, it is understandably difficult to collect water into the soil moisture cups since the water is not even reaching a depth of 3.0 m. The difficulty of water penetrating to a depth of 3.0 m suggests that there should be little, if any, leaching of chemicals from the stabilized FGD road bases.

As Table 3-21 indicates, the water is very clean at site C1 and in the surface water samples collected on November 23, 1992. The leachate from C1 produced some metals concentrations but low enough to pass EPA drinking water standards. The total dissolved solids (TDS) for C1 were 2556 mg/l, which is high for the drinking water standard, but for agricultural or runoff purposes over 5,000 mg/l is acceptable. The EC is 1.48 mmhos/cm. The surface water passes the EPA drinking water standards for all metals, except lead. Water was collected on January 25, 1993, and again on February 1, 1993. Two collection dates were needed to obtain enough sample to run a partial analysis. Water was obtained from sites C1 and A1. Even after very heavy rains, there was only about 15 ml of water collected from A1 during the two visits. Site B2 continued to hold a vacuum, but apparently the soil wasn't moist enough to collect a sample. Table 3-21 gives sample results. There was a limited amount of sample and not enough for all analysis. From the metals analysis there is very little difference between the control section, C1, and the roadbase section, A1. The waters contain very few metals, and they all fall below allowable EPA drinking water standards.

Based on the results presented above, there was no adverse environmental impact created by the FGD gypsum/bottom ash experimental test section.

## **STABILIZATION AND AGING CHARACTERISTICS OF STOCKPILED PREBLENDED FGD SCRUBBER BASE**

It was mentioned earlier in this report that materials are often stockpiled after preblending with other elements of the CCBP waste stream. Materials at the TU-Electric generating stations are normally not pre-mixed, whereas those at the Houston Lighting & Power Limestone and Parish generating station are blended and stockpiled as a scrubber base. These blends involve various ratios, R, of fly ash to FGD material that range from  $R = 30/70$  to  $70/30$ . In the 1991 and 1992 test sections discussed above, the materials were delivered to the field in an unblended form. However, the materials for the 1993 test sections were stockpiled scrubber base that consisted of a 50/50 blend of FGD and fly ash (see Table 3-15). Hence, it becomes necessary to resolve questions relative to the influence of the variations of fly ash and FGD in a stockpiled scrubber base on stabilizer demand. A second question arises as to the aging characteristics of fly ash and FGD materials as they relate to stabilization requirements. These two areas will be discussed below.

### **Stabilizer Requirements for Stockpiled FGD Scrubber Base**

#### *Experimental Program*

The materials used in this portion of the laboratory study are listed below:

HL&P Limestone Scrubber Base,  
HL&P Parish Scrubber Base,  
HL&P Limestone Class F Fly Ash,  
HL&P Parish Class C Fly Ash, and  
Cement TXI Class C Oil Well.

To simulate the range of FA/FGD ratios, R, the respective fly ashes and FGD material from each plant were blended at  $R = 30/70$ ,  $50/50$ , and  $70/30$ . The unconfined compressive

strengths of each of these blends were determined following cement stabilization at 2, 4.5, and 7 percent. Thus, the test program consisted of establishing the following parameters:

- a. Moisture-density relationships,
- b. Optimum moisture content, and
- c. Unconfined compressive strength at 1, 7, 14, 28, and 56 days.

The test methodology for these tests were in order with those defined in Figure 3-10.

#### *Moisture-Density Relationship*

Figures 3-31 through 3-34 show variations in dry density of the materials represented by the test variables specified above. The optimum moisture contents of scrubber base stockpiles represented by the three R values tend to increase linearly with cement content. However, variations in optimum moisture content for each blend were within  $\pm 1$  percent. This indicates that cement content will not significantly affect the optimum moisture content of stockpiles of constant proportions of fly ash and FGD by-product. If this ratio varies within the stockpile, the optimum moisture content will not be consistent and could represent a problem for construction.

On the other hand, the scrubber-base at the HL&P Parish plant appears to have a relatively constant FA/FGD ratio of 50/50. This is based on relatively repeatable property values measured in the laboratory. The principal difference between the scrubber base materials produced at the Limestone and Parish plants can be attributed to the cementitious nature of the latter's fly ash, which is Class 'C'; the former's is a Class 'F' (see Table 3-13) and therefore, primarily pozzolonic. This difference is reflected in the 7-day strengths of the two stockpiled scrubber base materials:

Parish 1654 kPa and  
Limestone 275 kPa.

### Optimum Moisture Content Trials for FA/FGD 30/70 at 2%, 4.5%, and 7% Class "C" Cement

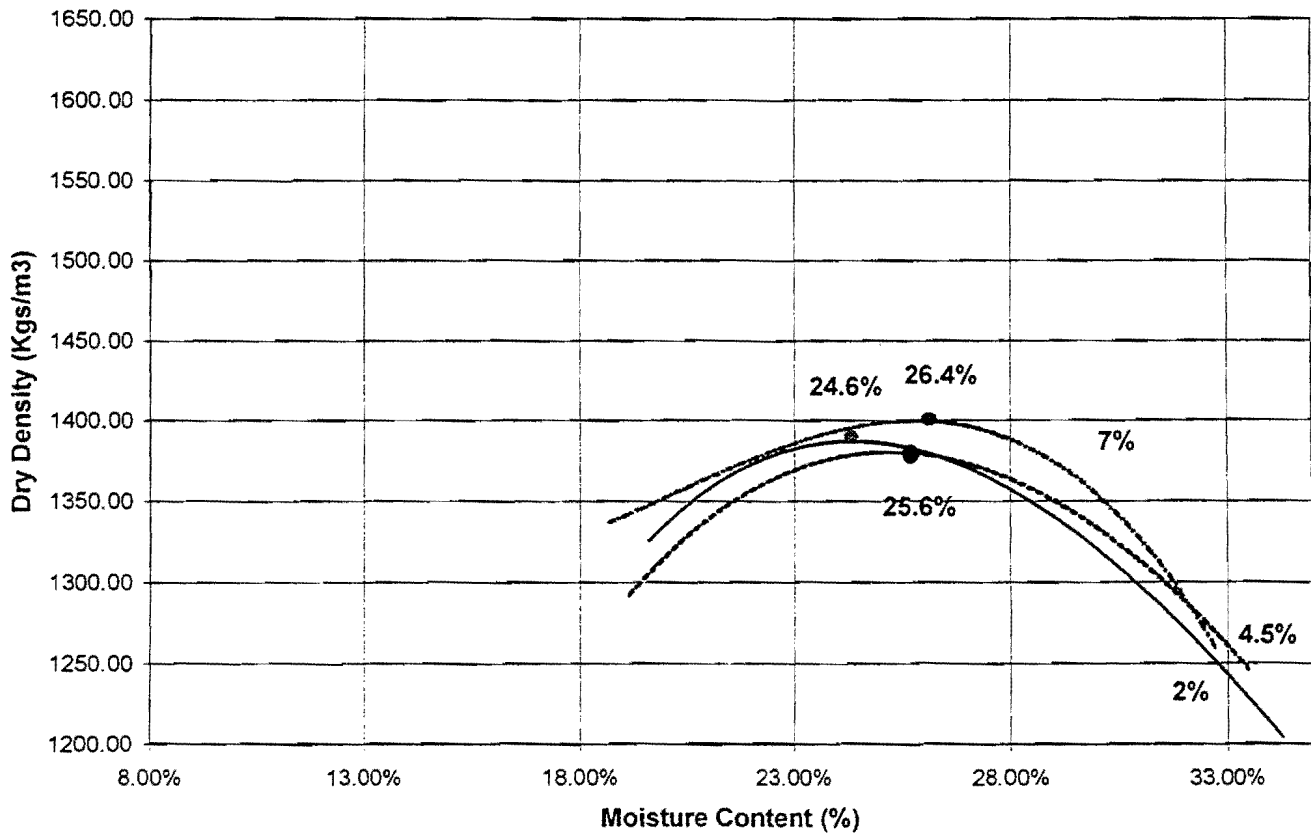


Figure 3-31. Optimum Moisture Content Trials for FA/FGD 30/70.

**Optimum Moisture Content Trials for FA/FGD 50/50  
at 2%, 4.5%, and 7% Class "C" Cement**

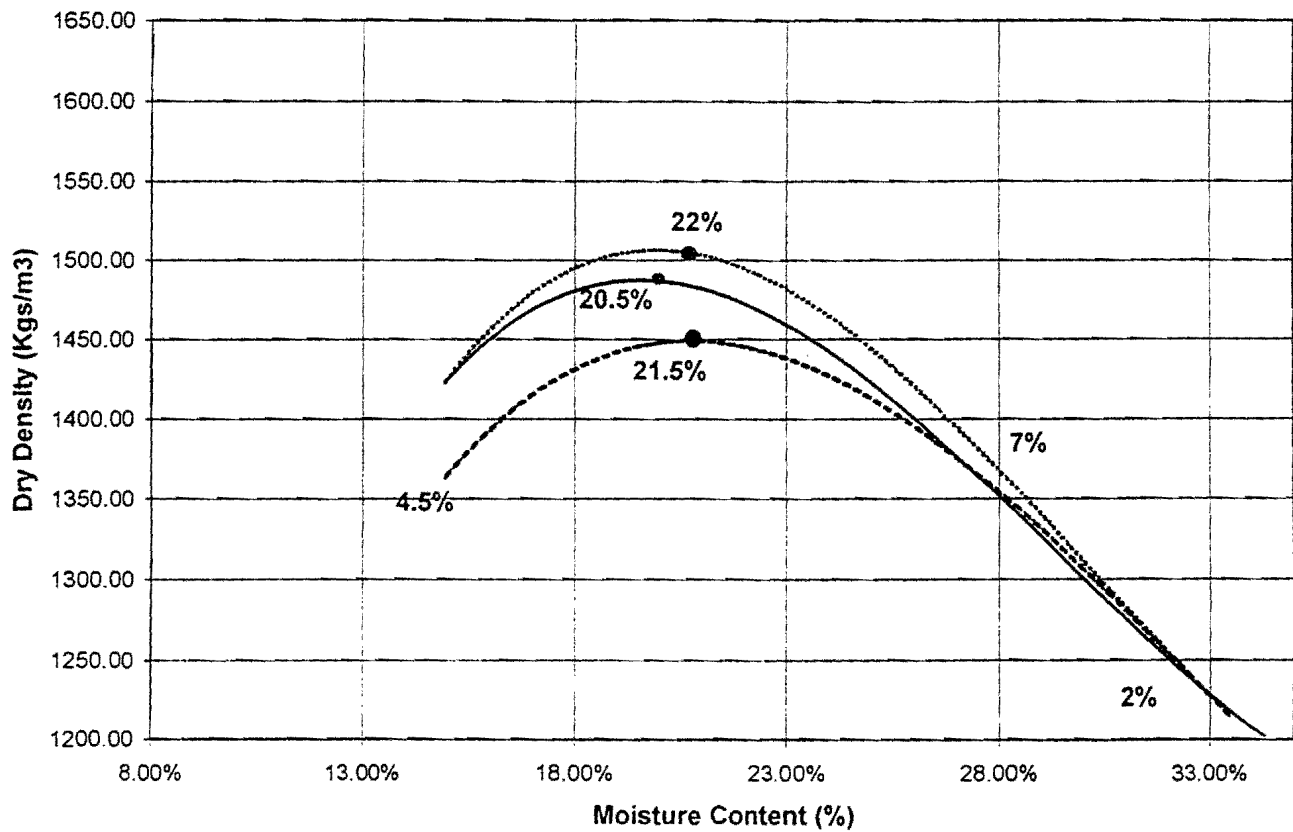


Figure 3-32. Optimum Moisture Content Trials For FA/FGD 50/50.



**Optimum Moisture Content Trials for FA/FGD 70/30  
at 2%, 4.5%, and 7% Class "C" Cement**

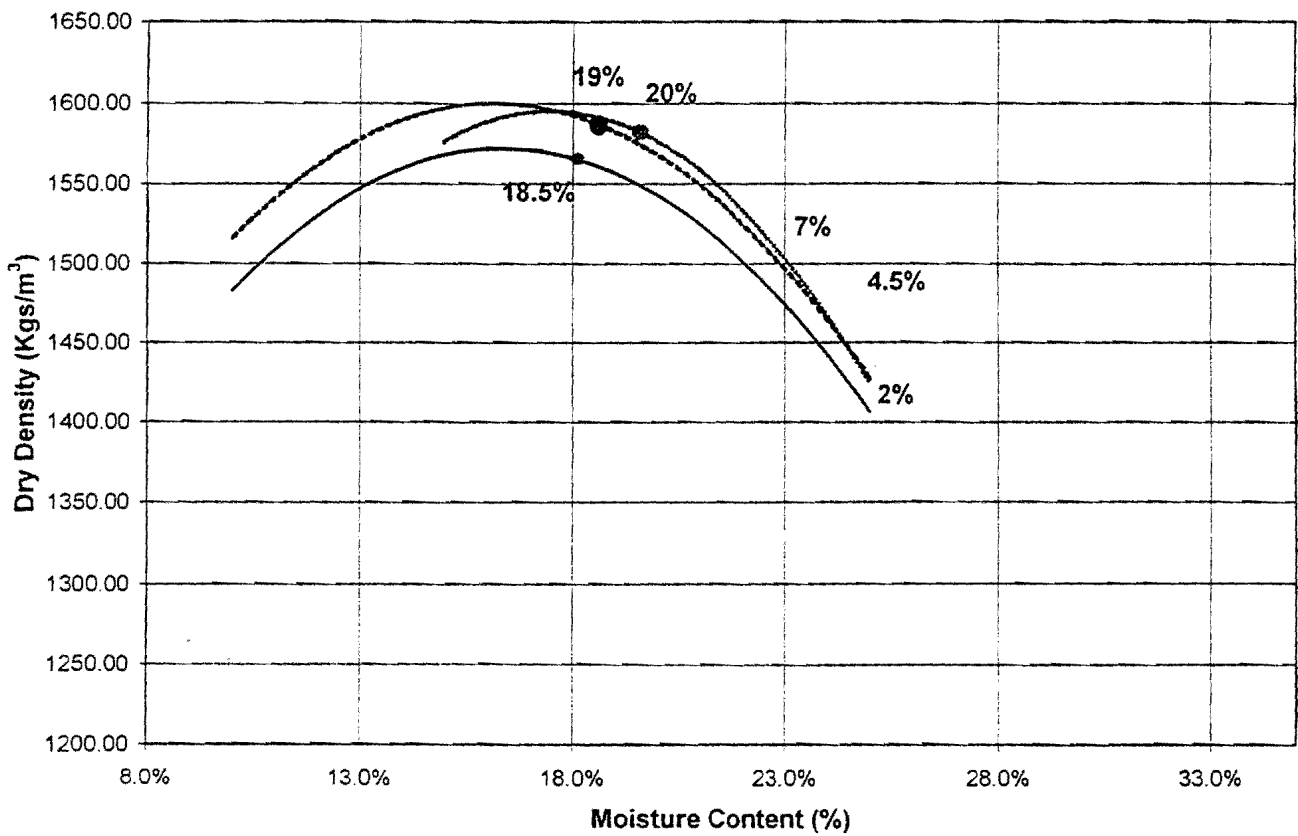


Figure 3-33. Optimum Moisture Content Trials for FA/FGD 70/30.

### Optimum Moisture Content Plot of FA/FGD Ratios vs. Cement Content of Mix

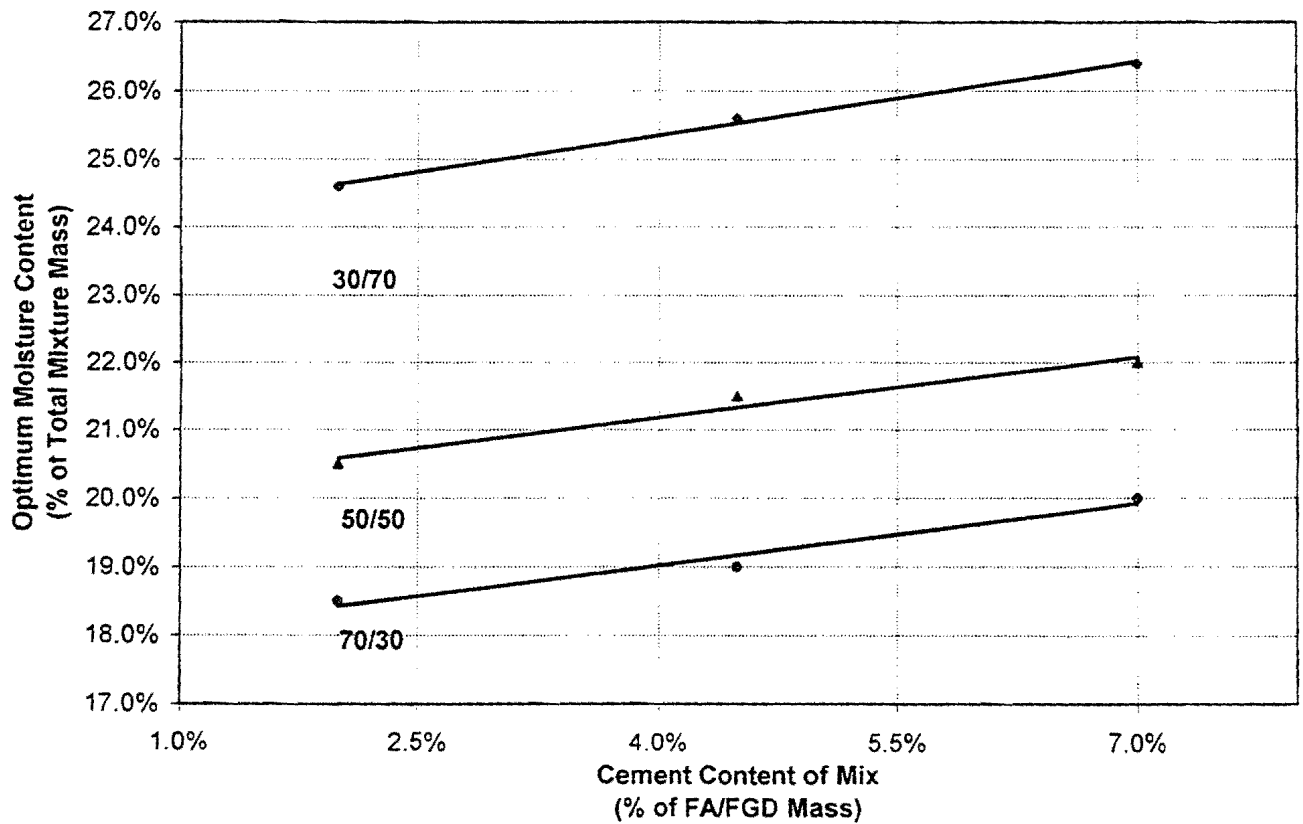


Figure 3-34. Optimum Moisture Content Plot of FA/FGD Ratios.

### *Additional Parish Class 'C' Fly Ash Used as a Stabilizer*

Parish fly ash was added in some trial mixes using both Parish and Limestone scrubber base to enhance strength. When Parish fly ash was added to Parish scrubber base, in amounts from 25 to 50 percent, 7-day UCS varied from 3378 kPa to 5240 kPa. Adding Parish ash to Limestone scrubber base in the same percentages resulted in UCS of approximately 2757 kPa to 37921 kPa for the blends. Only cement was considered as a stabilizer for the Limestone scrubber base material. Cement addition boosted the stockpile 7-day UCS from 275 kPa (no cement) to 5890 kPa at 4 percent cement and 7225 kPa at 7 percent cement. The interaction between fly ash and cement contents of Limestone scrubber base will be discussed below.

### **Analysis of Stockpile Aging**

#### *Parish Scrubber Base*

Since the  $R = FA/FGD$  ratio for stockpiled Parish material tends to remain constant at 50/50, the aging of the stockpiled scrubber base is best reflected by Figure 3-35. After 28 days on the pile, the strength of the material dropped from 3447 to 1930 kPa. This suggests that, when using scrubber that has been on the stockpile for an extended period of time, additional stabilizer (i.e., either cement or a Class 'C' fly ash) needs to be added. The specifications for the job mix design for Section 1 of the 1993 test sections took this into consideration.

#### *Limestone Scrubber Base*

More test variables were introduced into the aging study of the Limestone stockpile. Six mixture designs were prepared as follows:

FA/FGD ration = 30/70, 50/50, 70/30;

Cement Content = 2 and 7 percent;

Stockpile Age = 0 (unaged), 1, 7, and 28 days; and

Unconfined Compressive Strength at 3, 7, 14, and 28 days.

**Comaprison of Cylinder Strengths for 30/70 and 2%  
Cement From Stockpiles Aged at 0, 1, 7, & 28 Days**

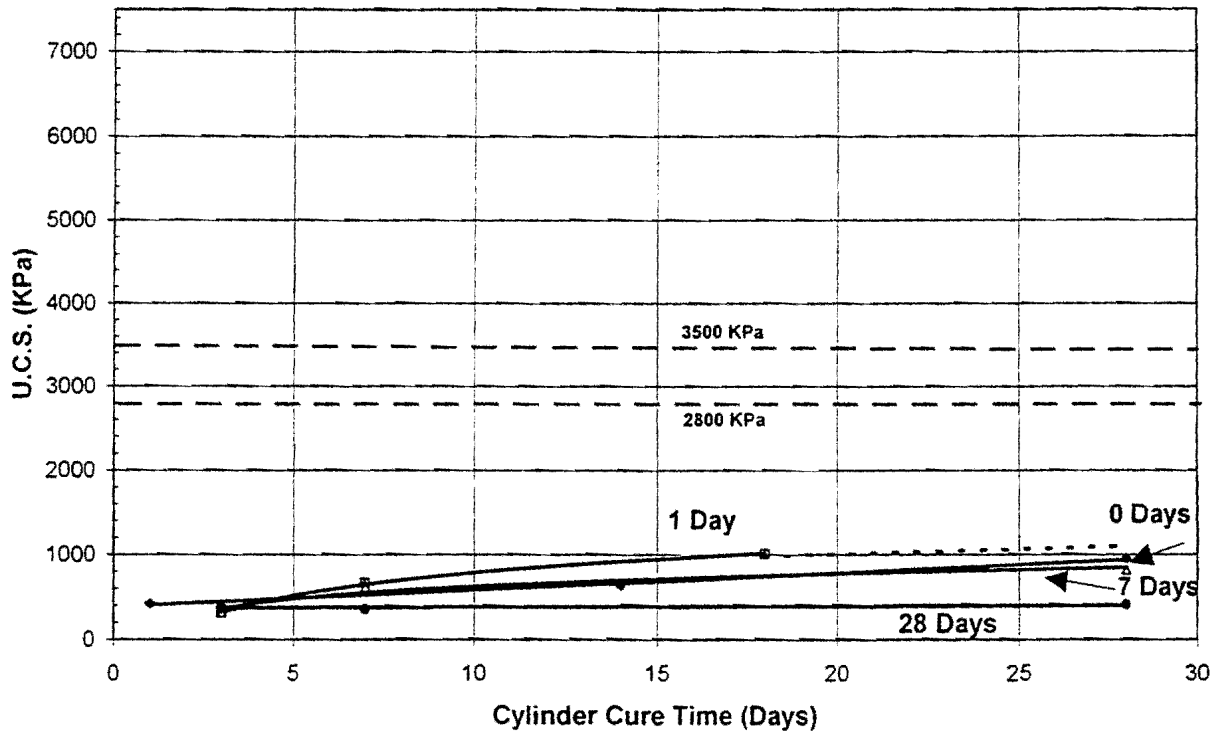


Figure 3-35. Comparison of Cylinder Strengths for 30/70 and 2% Cement from Stockpiles Aged at 0, 1, 7, and 28 days.

Figures 3-35 through 3-40 show the results. With a cursory examination of the results it might appear that the trend of strength decay is similar to that indicated in Figure 3-25 for the Parish scrubber. However, the cross plots of these data as shown in Figure 3-41 and 3-42, using strength versus stockpile aging, reveal an abnormal occurrence during the first 3 days on the pile. The strength appears to undergo an initial increase before it starts to decay. This might not have been revealed if the 3-day stockpile aging data were not taken, as was the case for the Parish material shown in Figure 3-25. It is therefore recommended that the strength after 3 days on the stockpile be determined for the Parish scrubber base to check if a similar anomaly exists.

Comparison of Figures 3-41 and 3-42 show that the magnitude of the stress increases with cement content. This indicates that an important window of opportunity exists to maximize road base strength with less stabilizer if the material is placed and compacted after 3 days. Figure 3-43 shows the effect of cement content on the 28-day strength of the three significant stockpile blends, whereas the strength remains relatively unchange by 2 percent cement. The strength decreases significantly as the ratio changes from 30/70 to 70/30.

The results of this effort strongly suggest that by taking into account the aging characteristics of stockpiled scrubber base, a more cost-effective mix design rationale can be achieved.

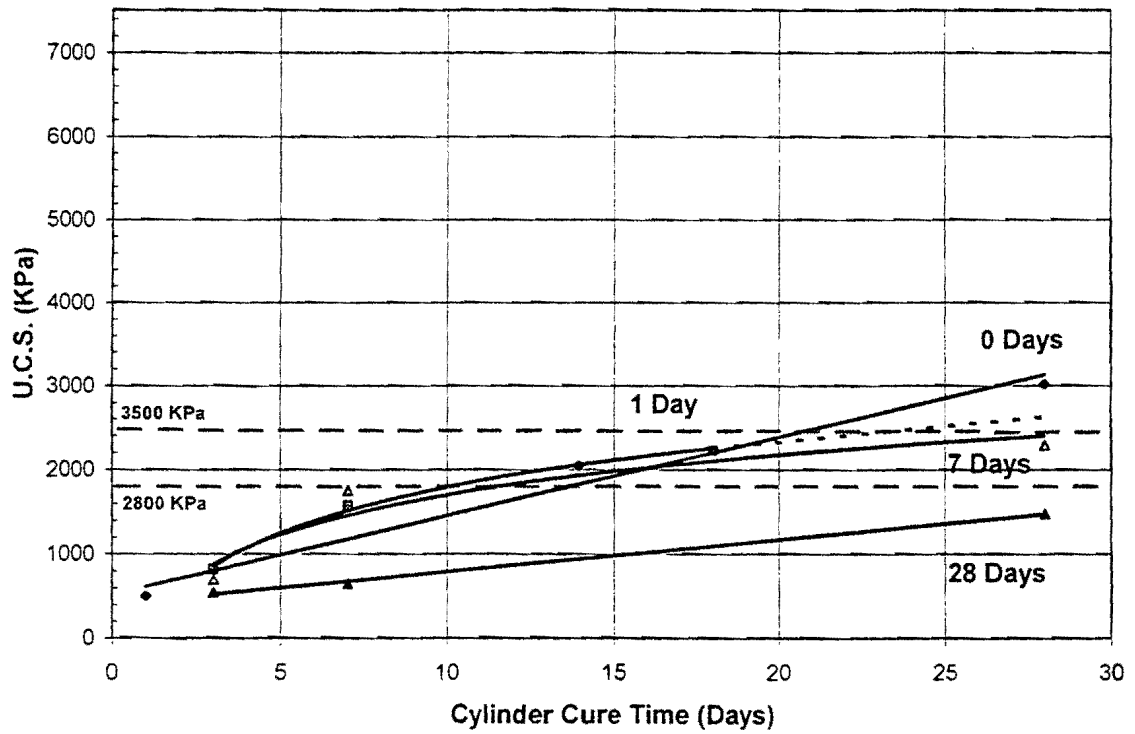


Figure 3-36. Comparison of Cylinder Strengths for 30/70 and 7% Cement from Stockpiles Aged at 0, 1, 7, and 28 Days.

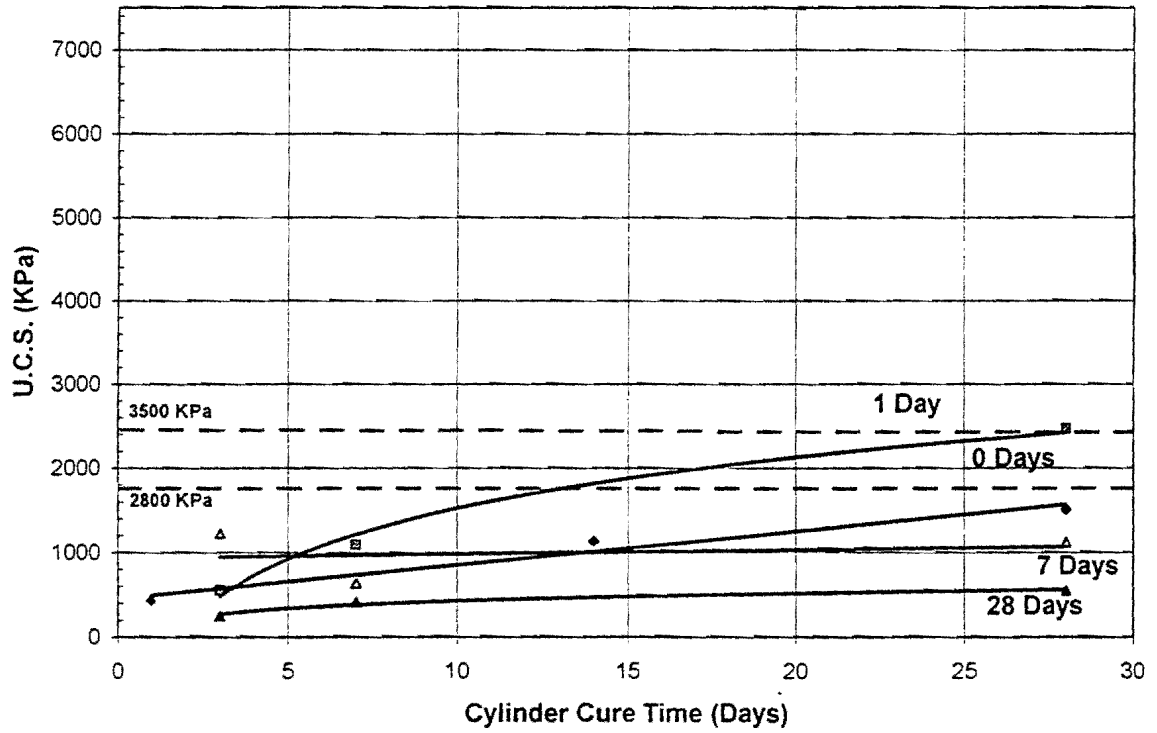


Figure 3-37. Comparison of Cylinder Strengths for 50/50 and 2% Cement from Stockpiles Aged at 0, 1, 7, and 28 Days.

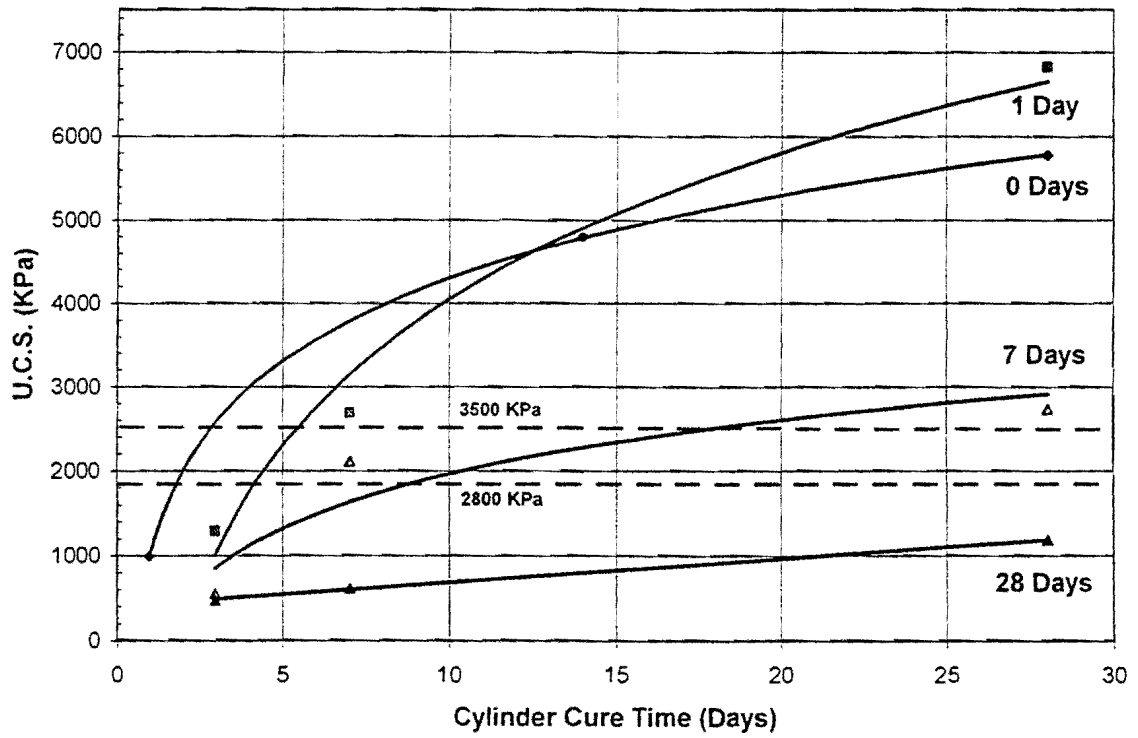


Figure 3-38: Comparison of Cylinder Strengths for 50/50 and 2% Cement from Stockpiles Aged of 0, 1, 7, and 28 days.



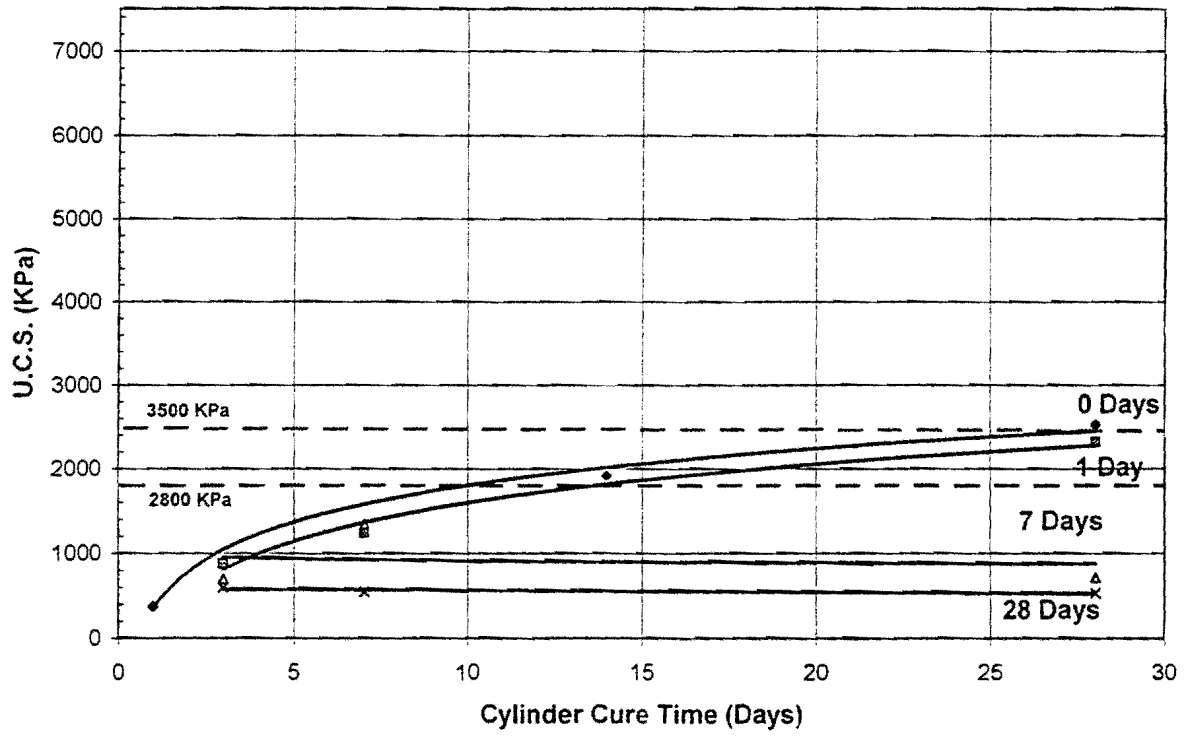


Figure 3-39. Comparison of Cylinder Strengths for 70/30 and 2% Cement from Stockpiles Aged at 0, 1, 7, and 28 days.

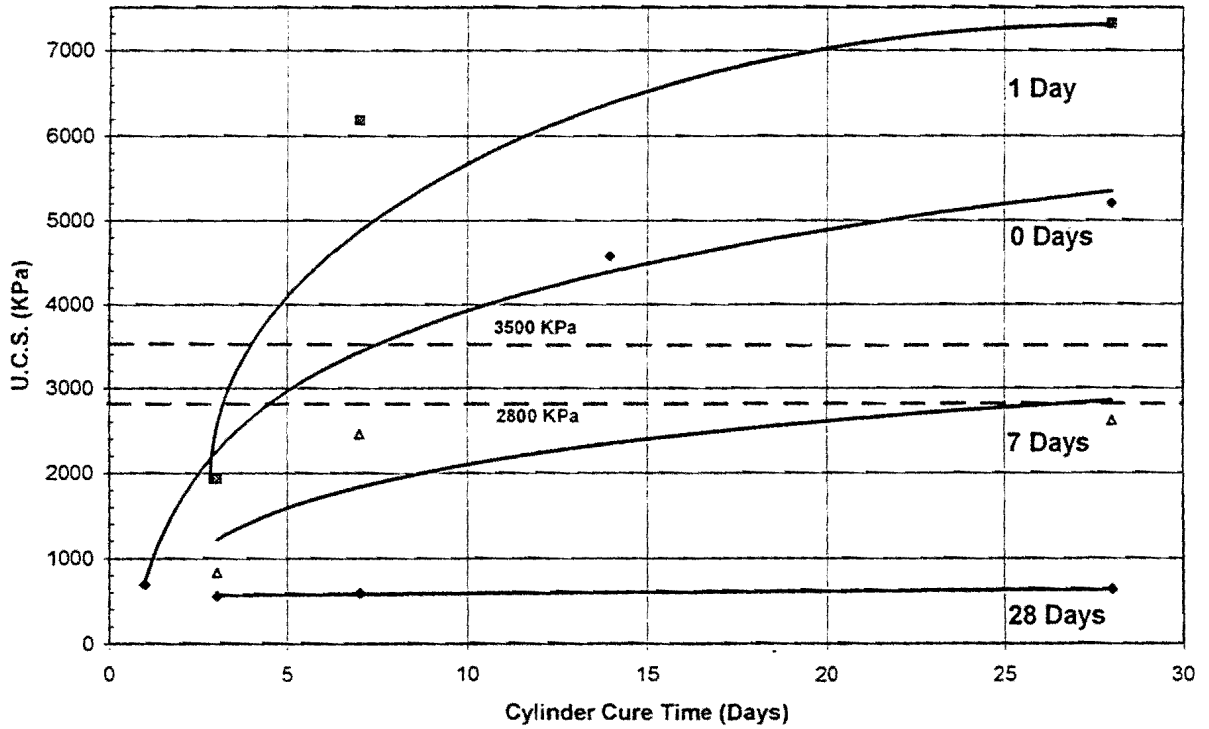


Figure 3-40. Comparison of Cylinder Strengths for 70/30 and 7% Cement from Stockpiles Aged at 0, 1, 7, and 28 days.

**28 - Day Compressive Strength Values  
Per Stockpile Age for R = 30/70, 50/50, and 70/30 with  
2% Cement**

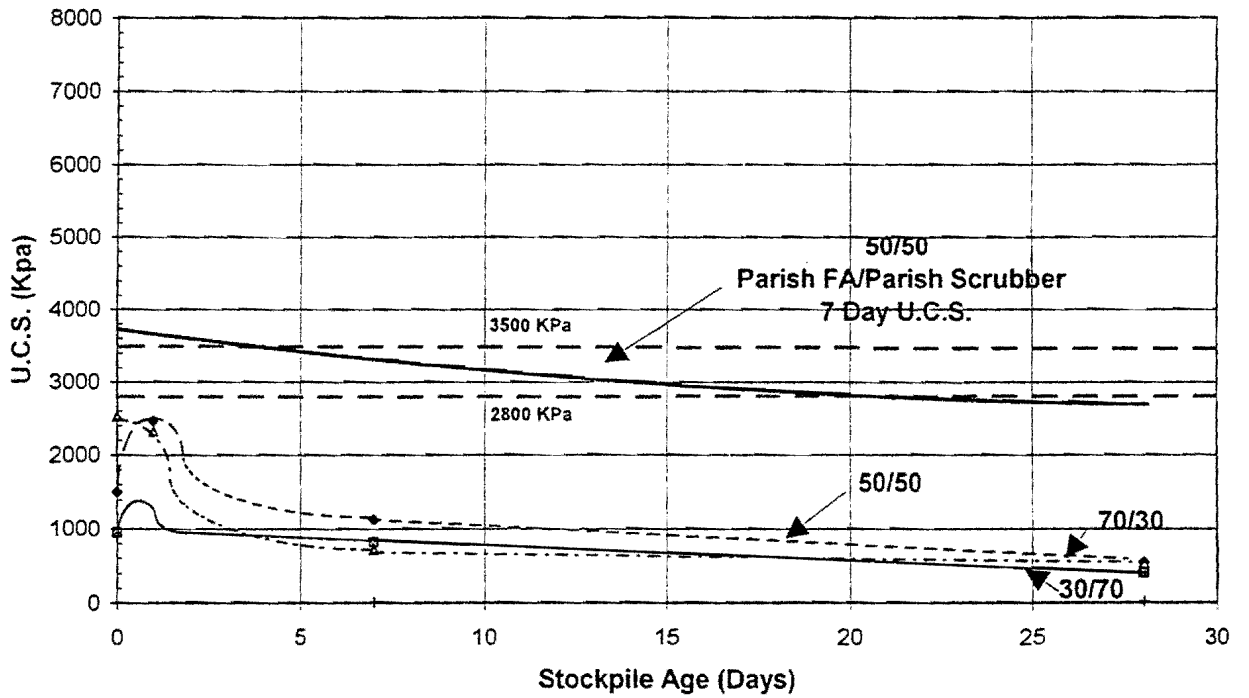


Figure 3-41. 28-Day Compressive Strength Values Per Stockpile Age for R = 30/70, 50/50, and 70/30 with 2% Cement.

**28 - Day Compressive Strength Values  
Per Stockpile Age for R = 30/70, 50/50, and 70/30 with  
7% Cement**

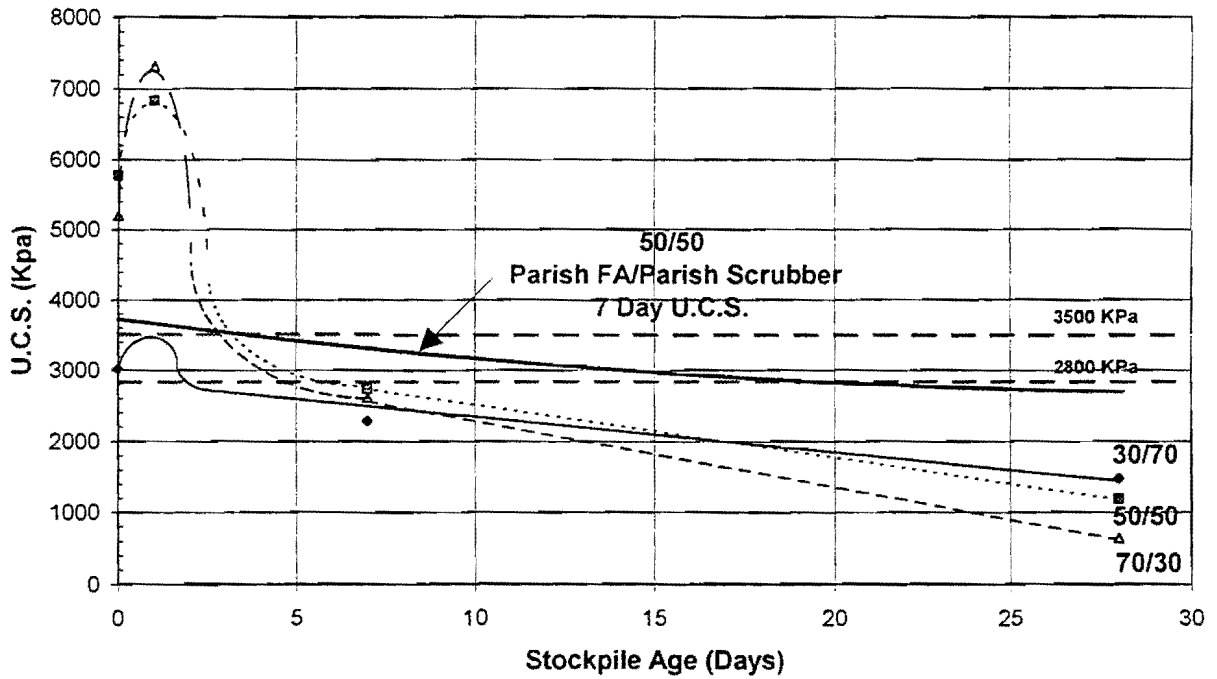


Figure 3-42. 28-Day Compressive Strength Values Per Stockpile Age for 30/70, 50/50, and 70/30 with 7% Cement.

### 28 - Day Unconfined Compressive Strength Plotted against Type "F" Fly Ash to FGD Ratio

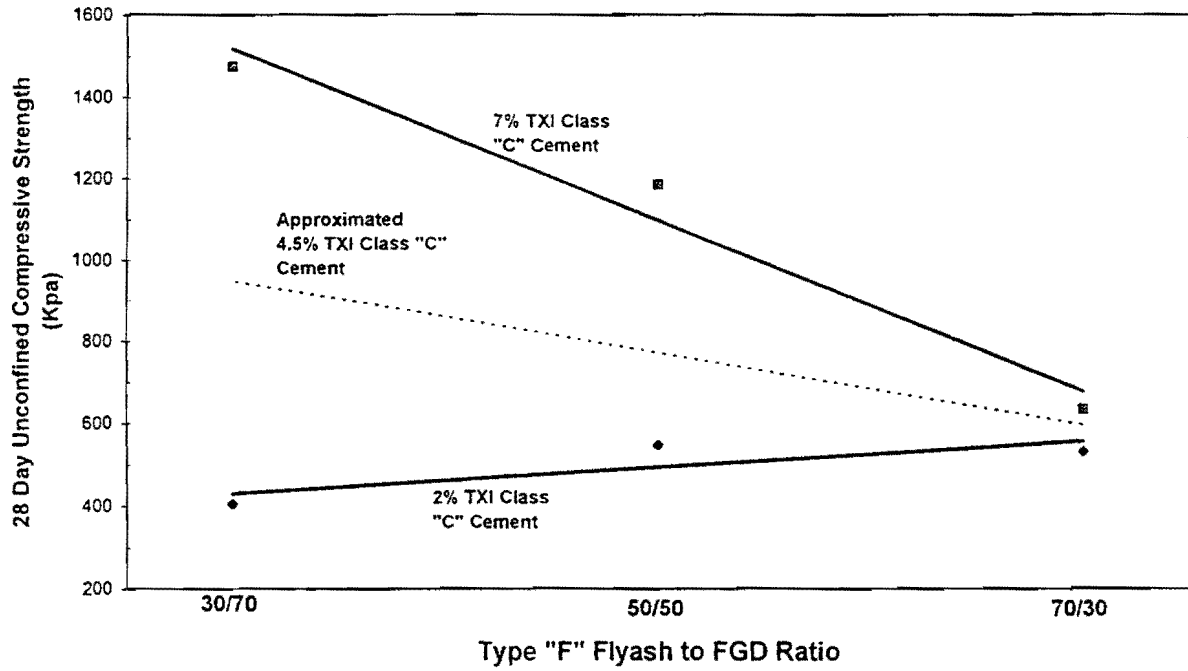


Figure 3-43. 28-Day Unconfined Compressive Strength Plotted against Type "F" Fly Ash to FGD Ratio.

## **Durability Testing**

The durability of the mixes were evaluated using the South African erosion test. The durability of a specimen was evaluated using the erosion index,  $L$ , which is defined as the average depth of erosion, measured at 15 different positions on the specimen, after 5000 erosion load repetitions in the erosion test (3.26). Durability tests were performed on the following mixes:

1. 50% Class 'F' fly ash and 50% FGD with 2% TXI cement,
2. 50% Class 'F' fly ash and 50% FGD with 4.5% TXI cement,
3. 50% Class 'F' fly ash and 50% FGD with 7% TXI cement,
4. 70% Class 'F' fly ash and 30% FGD with 2% TXI cement,
5. 70% Class 'F' fly ash and 30% FGD with 4.5% TXI cement, and
6. 70% Class 'F' fly ash and 30% FGD with 7% TXI cement.

The mass of fly ash, FGD, and cement required for each mix was calculated based on compacting to 95% of the maximum dry density (determined using ASTM D 1557). The fly ash, FGD, and cement were first dry mixed and then mixed thoroughly with the optimum water content. The mix was poured into the beam mold (450 mm x 75 mm x 75 mm) in three equal layers, and each layer was compacted with 56 blows using the modified compaction method (ASTM D 1557). A static load of 44 kN was applied and cycled five times to provide maximum density and a smooth finished surface. The beam was removed from the mold, and its mass and dimensions were recorded. The beam was then subject to accelerated curing by placing it in a sealed chamber (with about 50 mL of water) and storing it in a 70°C room for 7 days. After curing, the beam was cut, using a diamond blade saw, to a length of approximately 270 mm and moulded (sides and the bottom) with gypsum. The beam was then cut to a height of 50 mm and loose flakes on the surface were washed off using running water.

The moulded beam was soaked in water for about 1.5 hours after which the mass of the beam and the initial height of the sample (measured at 15 locations using a digital vernier caliper and a measuring jig) was recorded. The sample was then placed (in water) in the

erosion testing device and tested for its durability. The mass and height (at the same 15 locations) of the sample were recorded after predetermined number of erosion load repetitions, and the final readings were taken after 5000 erosion load repetitions. The details of the steps involved in sample preparation and testing are outlined in (3.26) and (3.27). The schematic diagram of the erosion test device is shown in Figure 3-44. Figures 3-45 to 3-49 show a few details of the test and the other details of the erosion test setup are summarized in Table 3-23.

### *Results*

The erosion depth for the test mixes after predetermined number of erosion repetitions is shown in Figures 3-50 and 3-51. The rate of erosion is observed to decrease with increasing number of erosion load repetitions due to a decrease in the contact stress resulting from an increase in the contact area with erosion. The test sample made with 50% fly ash, 50% FGD, and 2% cement disintegrated upon soaking in water. Hence, further testing on this mix was discontinued.

The influence of cement content on the erosion index,  $L$ , is seen in Figures 3-52 and 3-53. It can be seen that the erosion index decreases (as expected) with an increase in cement content. A comparison of the erosion index of the mixes with varying cement contents (Figure 3-54) indicates that the 70% fly ash and 30% FGD mix is, in general, more durable than the 50% fly ash and 50% FGD mix. The durability of the 70% fly ash, 30% FGD, and 4.5% cement mix is comparable to that of the 50% fly ash, 50% FGD, and 7% cement mix. The 50% fly ash, 50% FGD, and 4.5% cement mix is more durable than the 70% fly ash, 30% FGD, and 2% cement mix. Based on the proposed erodibility criteria for cement stabilized bases and subbases (3.26), a cement content of 4.5% is found to be the required minimum and the following mixes are suitable for bases and subbases:

#### Base :

50% fly ash and 50% FGD mix with 7% cement.

70% fly ash and 30% FGD mix with 4.5% or 7% cement.

#### Subbase:

50% fly ash and 50% FGD mix with 4.5% cement.

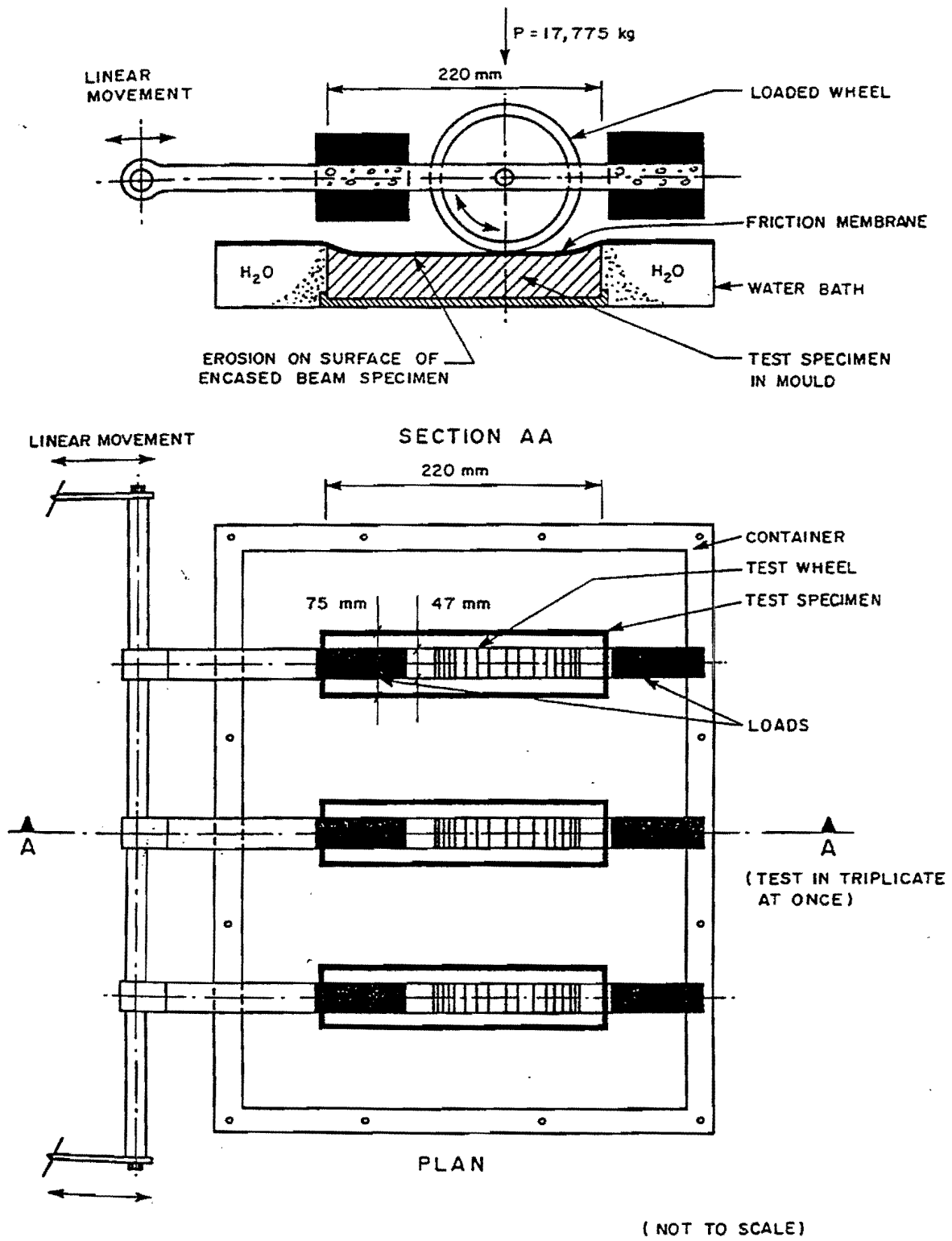


Figure 3-44. Schematic Diagram of the Erosion Test Device (3.26).



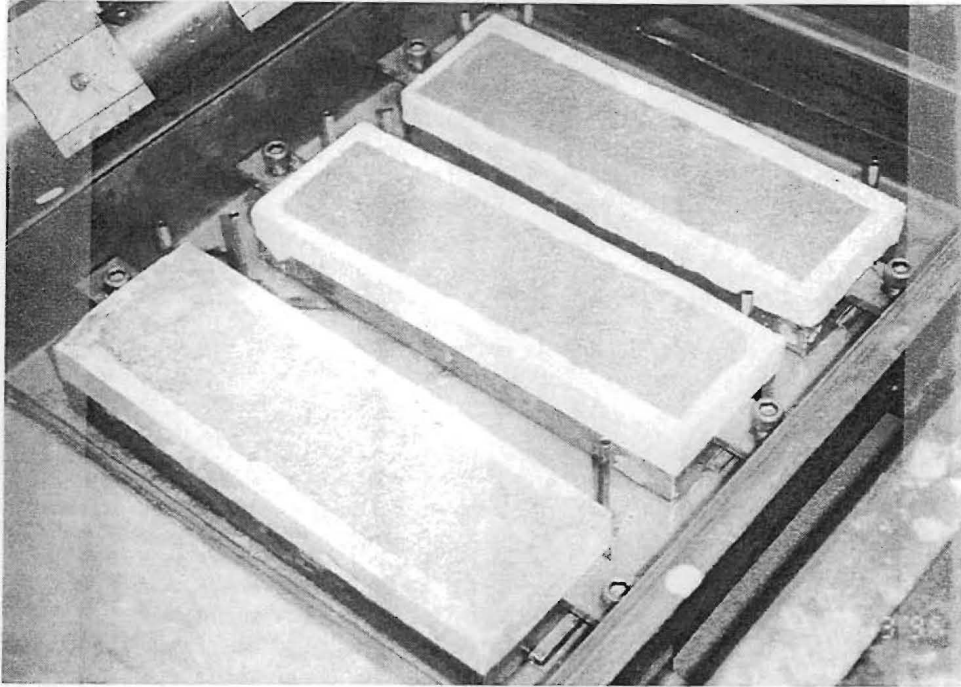


Figure 3-45. Erosion Test Specimens in Place in the Test Device.

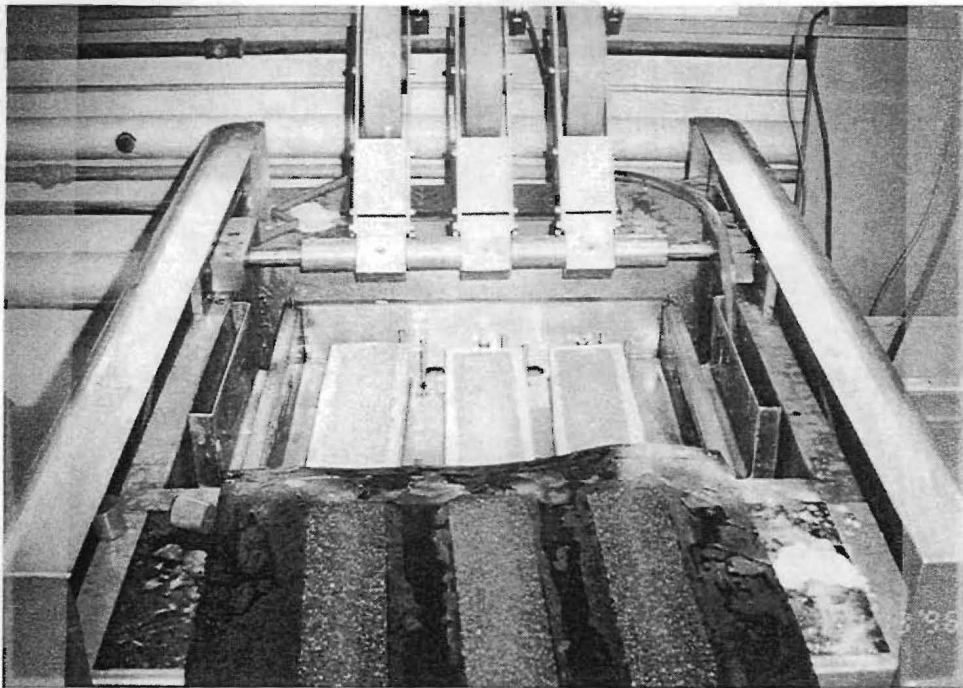


Figure 3-46. Erosion Test Device with Specimens in Place, Friction Pads, and Test Wheels.

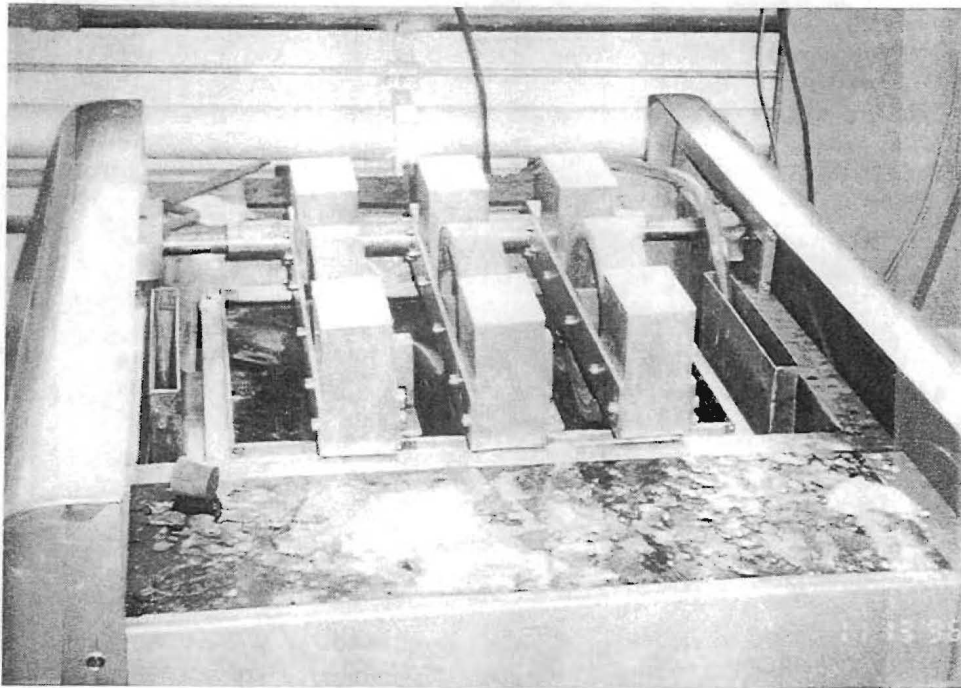


Figure 3-47. Erosion Test in Progress.

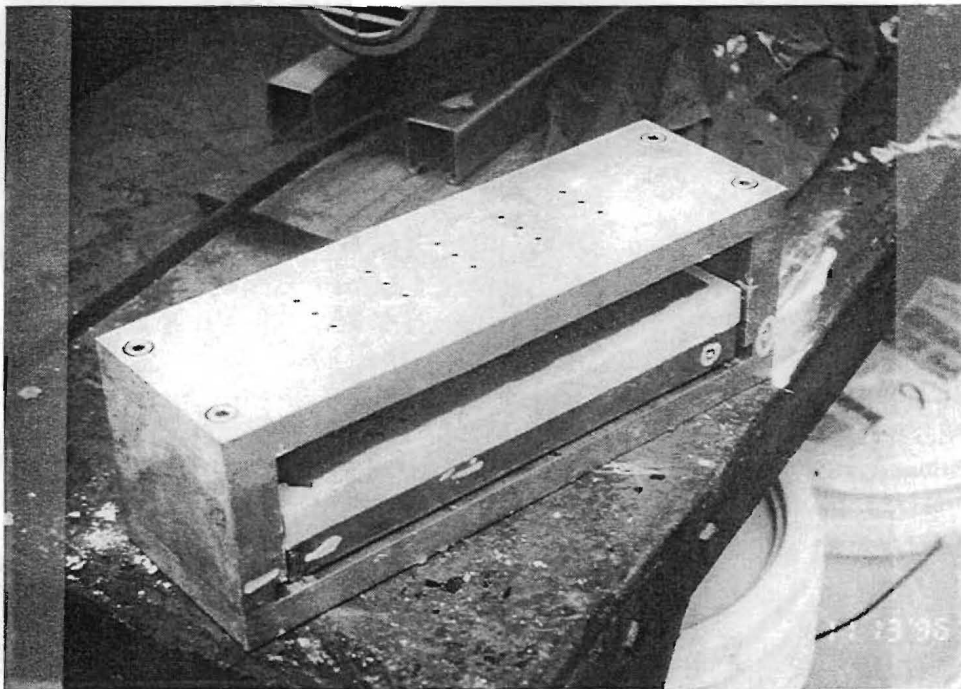


Figure 3-48. Erosion Specimen Placed in Jig to Record Erosion Depth.

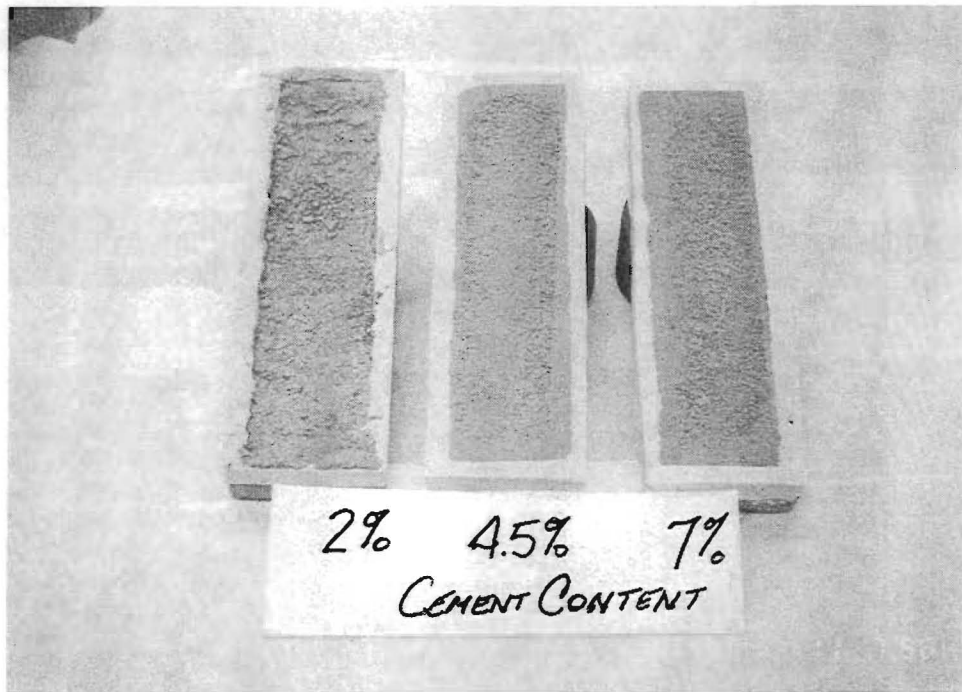


Figure 3-49. Test Specimens at the End of Erosion Test (Left to Right - 50% Fly Ash and 50% FGD mix with 2%, 4.5%, and 7% cement).

Table 3-23. Details of the Erosion Test Set Up (3.26).

Loading cycle frequency :	1 Hz
Testing length :	220 mm
Length of specimen :	≈ 270 mm
Height of specimen :	Variable, but normally fixed to the same height at the start of each test
Width of specimen :	75 mm
Maximum aggregate size :	≈ 19 mm
Total load per wheel :	17.755 kg
Contact stress (dry state) :	≈ 1.0 to 2.2 Mpa
Width of wheel :	47 mm
Diameter of wheel :	205 mm
Size of friction pad :	75 mm x 270 mm
Aggregate on friction pad :	1.0 mm (16 mesh) Silicon crystals
Number of cycles before measurement :	Variable, but normally 5000 for current criteria

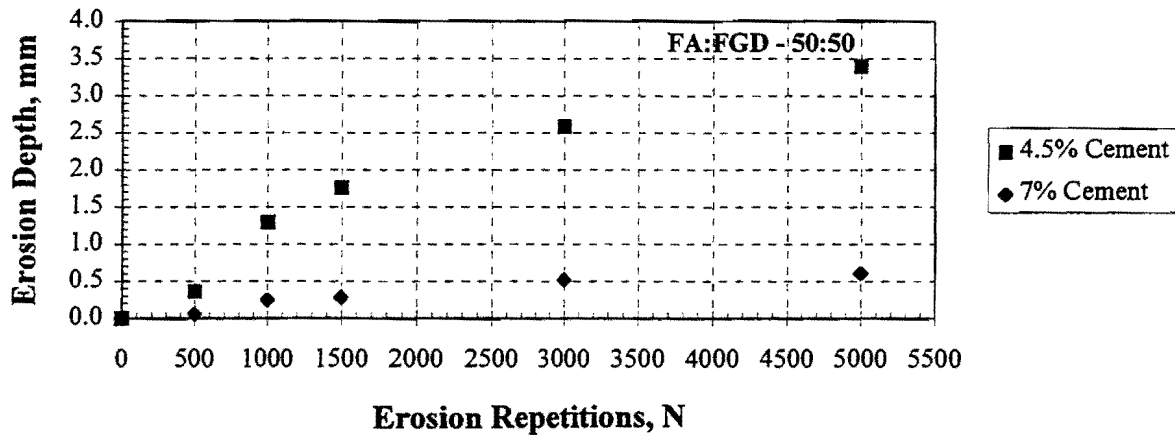


Figure 3-50. Erosion Depth of 50% Fly Ash and 50% FGD Mix for Varying Cement Contents.

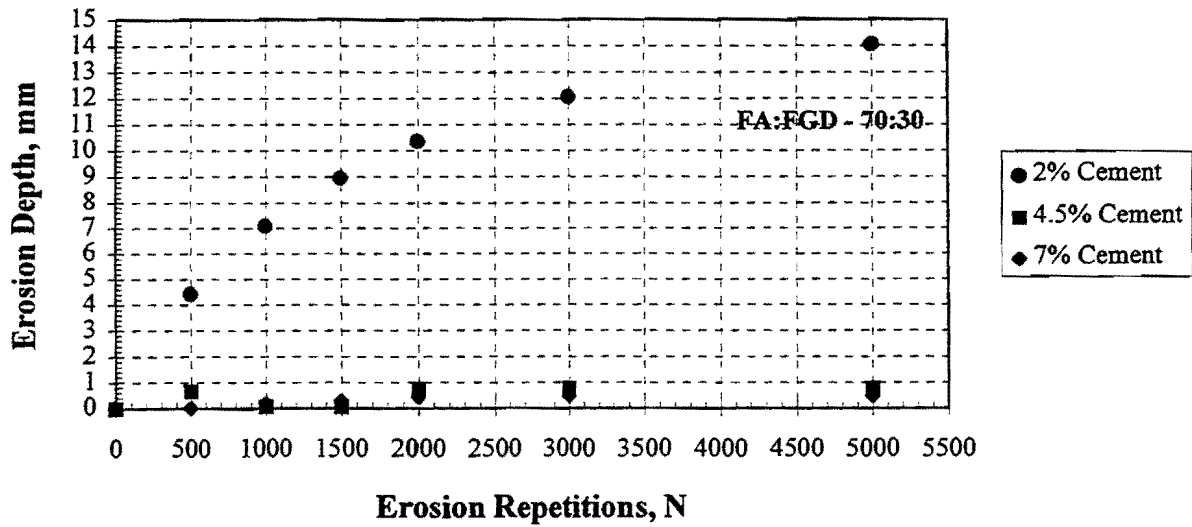


Figure 3-51. Erosion Depth of 70% Fly Ash and 30% FGD Mix for Varying Cement Contents.

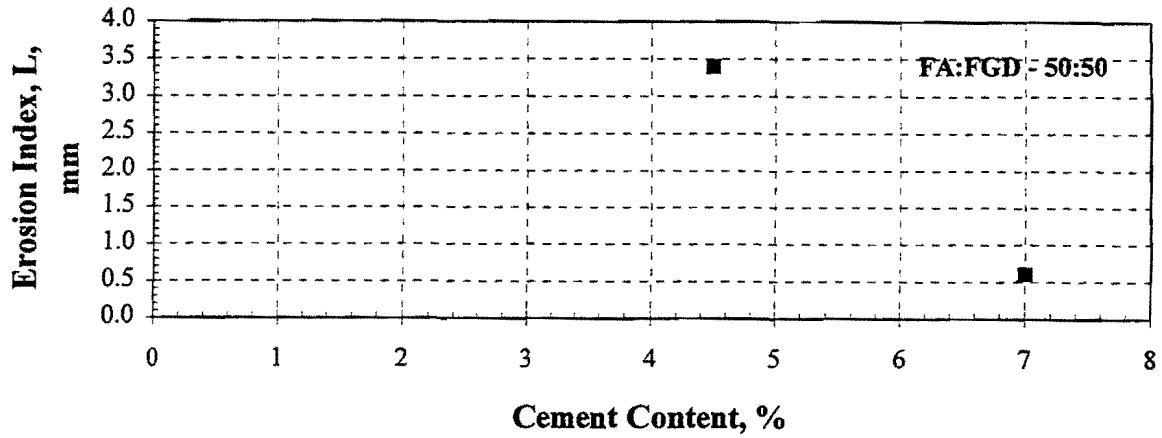


Figure 3-52. Influence of Cement Content on Erosion Index of 50% Fly Ash and 50% FGD Mix.

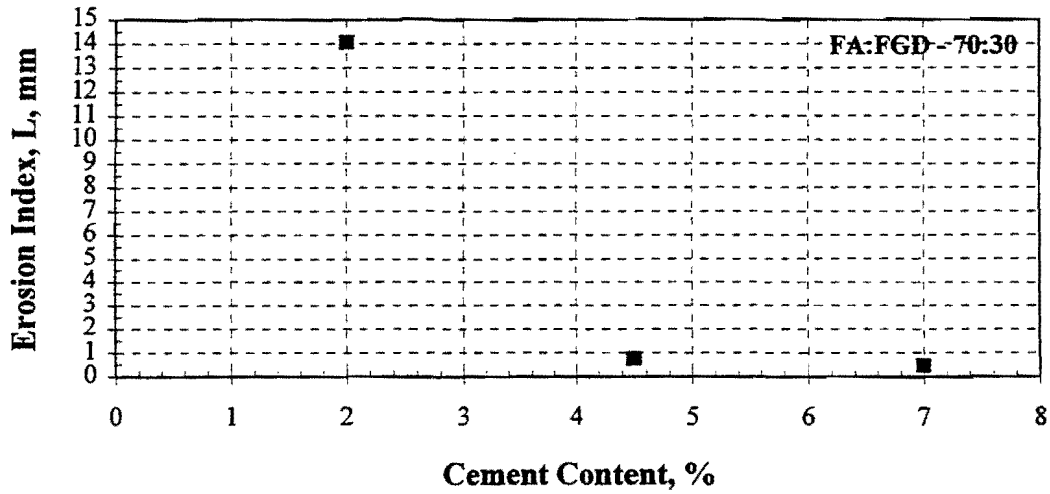


Figure 3-53. Influence of Cement Content on Erosion Index of 70% Fly Ash and 30% FGD Mix.

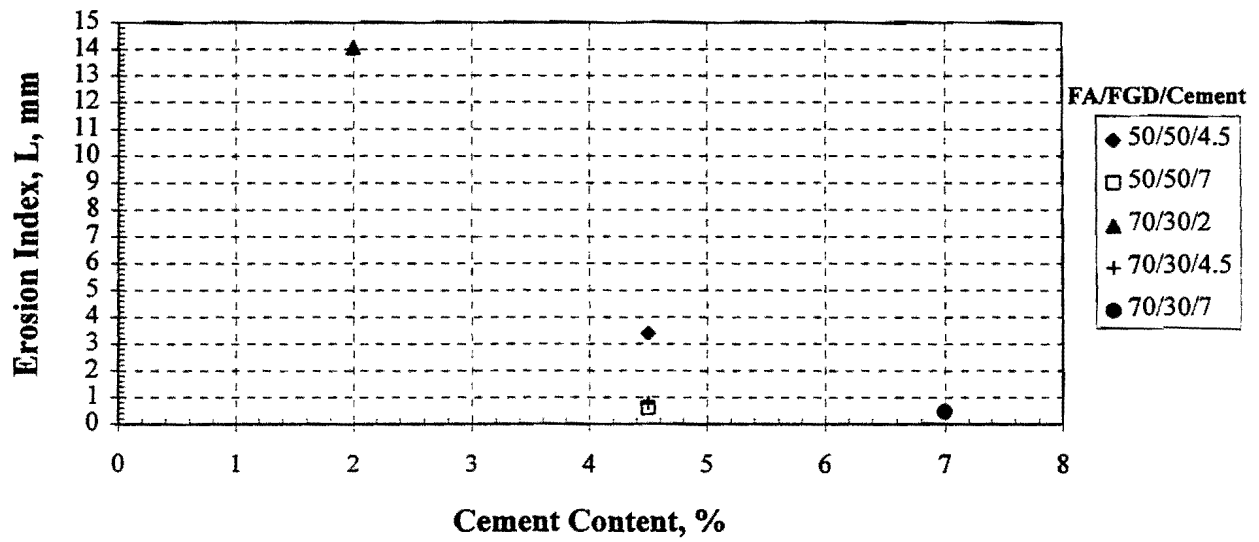


Figure 3-54. Erosion Index of Test Mixes with Varying Cement Contents.

## **Triaxial Testing**

Triaxial tests were performed on the following mixes in order to classify the material:

1. 50% Class 'F' fly ash and 50% FGD with 2% TXI cement,
2. 50% Class 'F' fly ash and 50% FGD with 4.5% TXI cement, and
3. 50% Class 'F' fly ash and 50% FGD with 7% TXI cement.

The test specimens were prepared by compacting the fly ash, FGD, and cement mix at optimum moisture content in cylindrical moulds of 101.5 mm diameter and 203 mm height. The material was compacted in ten equal lifts with each lift being subject to 25 blows from the modified hammer (ASTM D 1557). The mass, height, and circumference of the cylinders was recorded, and the specimens were cured in a damp room for 7 days in accordance with Tex-120-E (3.28). The test specimens were not subject to capillary wetting or a surcharge and immediately after the curing period the mass, height, and circumference of the specimens was recorded and the specimens were tested in accordance with Tex-117-E (3.28).

For each mix the cylinders were tested at 0, 20.7, 34.5, 69, 103.4, and 138 kPa lateral pressures. The triaxial tests were performed using an Instron machine, and the load and displacement readings were recorded by an automated data acquisition system. The failure envelope for each mix was obtained from the Mohr's circles and transferred onto the chart for classification of subgrade and flexible base materials. All of the mixes tested exceeded the requirements of a Class 1 (good flexible base material) material.

## **SUMMARY OF FINDINGS AND RECOMMENDATIONS**

A comprehensive literature review has revealed that FGD by-product calcium sulfate (gypsum) and sulfites (Gypsite) can be stabilized with either cement or fly ash and utilized as road bases or sub-bases. This research, along with work previously done at the Texas Transportation Institute, has shown that these procedures developed for FGD materials also apply to other forms of gypsums as well.



A set of test procedures and materials selection guidelines, which do not necessarily comply with TxDOT standard methods, have been generated and validated in the field using experimental test sections. Gypsum and Gypsites require high-sulfate-resistant cements for stabilization that exceed the protection provided by Type II cements. This protection is achieved with tri-calcium aluminate levels of less than 4 percent.

Post-construction evaluations of the structural integrity and environmental impact of a series of TTI-constructed test sections would classify stabilized FGD<sub>2</sub> road bases as excellent, with negligible effect on groundwater, surface runoff, and leachates.

Mix designs of FGD scrubber bases from two power generating stations indicate that, depending on the amount and type of fly ash relative to FGD material in the stockpile, sufficient stabilization could be accomplished with as low as 2 percent or as high as 7 percent cement. Due to the chemically active nature of some fly ashes, the age of the stockpile should be considered in the mix design. The best window of opportunity exists between a stockpile age of 1 to 3 days. Beyond this time, additional stabilizer may need to be provided.

In summary, FGD by-products would appear to offer a cost-effective alternative to natural aggregates currently in use. That sources for these materials coincide with regions in Texas where conventional aggregate supplies are non-existent or depleting represents an additional advantage for utilizing.

### **Recommendations**

Since many of TxDOT standard methods for evaluating base courses do not necessarily apply to FGD materials, they should be re-evaluated for use on Coal Combustion By-Products. The promising concepts revealed in this section of the report should be field tested to provide the confidence necessary to enhance their acceptance for use on Texas roadways. Another area for future considerations entails the use of high-volume fly ash cements (i.e., with Class 'F' fly ash contents up to 60 percent) as a stabilizer for FGD by-product. High-sulfate-resistant cements are usually more expensive and not always locally available. High-volume fly ash cements, in addition to incorporating a cheaper class of fly ash, also utilize the more readily available Type I cements and still achieve a sulfate resistance equal to if not better than those used in this study.



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## **SECTION IV**

### **SULFUR-MODIFIED BOTTOM ASH (SMBA) ASPHALT MIXTURES**

#### **BACKGROUND**

During the late 1960s, Shell Canada Limited began investigating asphaltic mixes involving low-grade aggregates (4.1, 4.2). This work involved the initial coating of the aggregate with bituminous binder and the subsequent addition of sulfur to fill the void spaces around the aggregate. The process was found to be particularly adaptable for use with inexpensive, ungraded sands such as beach sand, dune sand, and blow sands typical of those found along the Gulf Coast region of Texas. A number of compositions were developed by Shell under the trade name THERMOPAVE and by the Texas Transportation Institute (TTI) in the United States as sand-asphalt-sulfur (SAS) mixtures.

#### **SAS Development in Canada**

Shell Canada Limited has been investigating the use of sulfur as a means of producing asphaltic mixes with low-quality aggregates since 1963. This work produced a series of patents (4.3 - 4.11) and experimental field trials (4.12, 4.13). The results of these trials and an extensive research and development program (4.14 - 4.16) led to the development of a number of specialty items, two of which are:

1. THERMOPAVE\* a sand-asphalt-sulfur paving material (4.17) and
2. THERMOPATCH\* a remeltable material used for pavement repair (4.18).

\* Registered Trade Marks of Shell Canada Limited

Whereas the former was developed primarily for use with low-quality sands, THERMOPATCH mixtures use a variety of aggregates ranging from sand to graded crushed rock or gravel.

In 1964, Shell Canada conducted full-scale paving trials in Oakville, Ontario, and St. Boniface, Manitoba, in which the in-service performance of SAS systems were compared

with conventional asphaltic concrete using both well-graded aggregates and poorly graded sands.

Based on the success of these early trials and the additional data base provided by concurrent laboratory programs, Shell undertook a number of subsequent field trials in Richmond, British Columbia (1970) (4.12), Oakville, Ontario (1971), St. Antoine, Quebec (1972), Tillsonburg, Ontario (1972), and McLean, Saskatchewan (1974). In addition, a number of small-scale field trials were conducted to evaluate various paver modification schemes. The results of these trials are summarized below:

1. Both solid and liquid sulfur can be used to prepare SAS mixtures using batch-type asphalt hot mix facilities.
2. Mixes that were compacted by conventional rolling following placement experienced early damage, whereas those which were not compacted performed very well.
3. Mixtures prepared with one-sized sands performed equally to those prepared with the dense-graded systems.
4. Because of the pourability of SAS mixtures, whose slump characteristics can range from 50-150 mm (ASTM Test Method C-143), pavement thickness control could be a problem necessitating the use of forms as in concrete construction. Flowability of the mixes induced an adverse shoving effect on the surface of the mat due to the interaction with the paver screed. This required an alteration to be made to pavers on future SAS construction projects in which screed height was hydraulically controlled to balance upward lift and downward pressure so as to maintain a uniform pavement thickness.
5. Screed temperature control must be maintained within the suggested working range of the mixture (125° to 150°C) for optimum workability and minimum generation of noxious fumes.
6. SAS mixes with S:A ratios of 1.0 to 2.5:1.0 are recommended for use in flexible pavement mixture designs, whereas S:A ratios greater than 5:1 can be used in



situations requiring rigid pavement designs. A typical SAS formulation is 82 parts sand, 6 parts asphalt, and 12 parts sulfur by weight (4.19).

For a more detailed treatment of the historical background of the THERMOPAVE and THERMOPATCH concepts, the reader is referred to the excellent review by Rennie (4.2).

### **SAS Development in the United States**

Two factors generated interest in the United States in the SAS concept during the 1970s: (a) the decreasing availability or total absence of quality aggregates in a number of regions around the country such as the Gulf States and (b) the projected surplus of low-cost sulfur anticipated from secondary sources in connection with pollution control processes. TTI under the co-sponsorship of the Sulphur Institute and the Bureau of Mines, instituted a program to introduce the SAS concept to the United States (4.20). Volume III (revised 1978) of the final report was prepared as a user's manual and sets forth some preliminary specifications and recommended mix design, construction, and quality control procedures to be used on SAS highway systems. Saylak et al. (4.21, 4.22) conducted a preliminary structural analysis using layered-elastic theory on SAS systems that indicated excellent fatigue life compared to conventional asphalt concretes. The Bureau of Mines made independent studies of SAS mixes using blow sands and mine tailings. Patches of these materials placed within the city limits of Boulder City, Nevada, in 1975 are still performing well to date (4.23). The Bureau also used an SAS mix to repair a portion of McCarran International Airport in Las Vegas (4.24).

Two experimental field projects evolved out of the Shell and TTI studies: (a) LA. 108 in Sulphur, Louisiana January 1977 (4.25) and (b) U.S. 77 in Kenedy County, Texas April 1977 (4.26). A part of the Kenedy County construction report includes an extensive analysis of emissions generated during construction. By 1980 at least six SAS projects had been constructed in the United States, and seven in Canada. The results tend to verify the conclusions established earlier by Shell, that as long as mix temperatures are maintained

below 150°C, concentrations of H<sub>2</sub>S and SO<sub>2</sub> are well within the safety limits suggested by the American Conference of Governmental Industrial Hygienists (4.27). All of the field trials conducted both in the United States and Canada utilized conventional batch type hot mix plants and paving equipment. These field sections were subjected to post-construction evaluation for more than 4 years and performed satisfactorily. Most recently the 1977 Kenedy County pavement was replaced after more than 15 years of satisfactory service. For a more detailed treatment of the development a SAS mixtures, the reader is referred to the state-of-the-art review by Saylak (4.28).

### **The Use of CCBP Bottom Ash in Asphalt Pavements**

Prior to 1980 the state of the art for using bottom ash in black base and bituminous surfacing was defined by three FHWA documents (4.29, 4.30, 4.31). The Texas Department of Transportation (TxDOT) undertook a research program to evaluate bottom ash as an aggregate in asphaltic mixtures (4.32). The latter led to a 1980 project in cooperation with TxDOT's Material and Tests Division (D-9) and the FHWA to construct three test sections of asphaltic concrete in the Paris District in which bottom ash was one of the components.

The sections were on Farm-to-Market Road 1870, Interstate Highway 30 (4.33), and State Highway 11. The test sections were from 90 to 250 m in length. The aggregates used in the mix consisted of 55 percent crushed gravel from a source located near Frogville, Oklahoma, and 45 percent bottom ash from TU-Electric Monticello Plant near Mt. Pleasant, Texas. This mix was classified as a Type D surface mix. By selecting only two aggregate materials and using a relatively large amount of bottom ash in the mix, the asphalt requirements were high. The asphalt was varied slightly to match existing pavement deflection and conditions, with traffic also being a consideration. The asphalt contents selected for the test sections were 10, 11, and 12 percent, respectively.

After evaluation of the three sections for an extended period of time, several observations and conclusions were made as follows:

1. The optimum asphalt content is less critical when using bottom ash blends than when using natural aggregate blends.

2. The voids in the bottom ash will provide an increased safety factor against bleeding and flushing caused by too much asphalt and higher traffic density than expected.
3. There was no lateral displacement of this bottom ash-gravel mix during compaction, indicating a high internal friction of the mix.
4. The mix cools faster than a conventional mix containing only natural aggregates.
5. Even though the mix contained a large amount of crushed gravel having a low polish value and a high asphalt content, skid numbers were above 40 on all three test sections.
6. Compression tests performed on Hveem specimens after immersion indicate that bottom ash mixtures have a high degree of resistance to moisture damage.

As a direct result of this study, an experimental project was undertaken in 1985 in the form of a 23 km section of Interstate Highway 30 in Hopkins County, Texas ([4.32](#), [4.33](#)).

The primary difference in this research project and the three experimental test sections that were constructed in 1980 was the length of the project and a more conventional design approach. The project was 23 km in length and included both the east and westbound lanes. The pavement consisted of 50 mm of Type C level-up overlaid by 25 mm of Type D surface. Both courses contained approximately 20 percent bottom ash by weight of the total mixture. The other aggregates included crushed sandstone, sandstone screening and local field sand. The asphalt requirement was about 7 percent by weight. A 5-year evaluation study with an annual report to the Federal Highway Administration was a condition of the contract. The study included evaluation of the laboratory design and project test data. It also required field evaluations of construction procedures and pavement distress. Roadway cores and skid values were secured from a test section within the project during the study period. All designs and project tests appeared well within the specification requirements.

After 11 years there has been no apparent pavement failures or distress due to bottom ash mix characteristics. In 1994, 19 mm of plant mix seal was placed over the bottom ash

mix. The purpose of the seal was to correct grade problems only, as there was no failure of the bottom ash mix or loss of skid characteristics.

As a result of the satisfactory pavement performance of this project, the Paris District developed bottom ash mix designs and began using these mixes on new projects.

### **Development of SMBA Asphalt Concrete Mixtures**

Along with the use of bottom ash in the base course mix designs discussed in Section III, a new study maximized the use of this by-product in both lane and surface courses. The overall objective of this activity was to exploit the work done in Texas during the 1980s as discussed above. In addition, this study was directed toward increasing the use of bottom ash while decreasing the demand for conventional aggregates such as sand and crushed rock. The factors that limit the use of bottom ash in HMAC mixtures include the following:

1. It requires high asphalt demand due to the porosity of the ash.
2. Its low crush-resistance causes the bottom ash to break down during compaction.
3. It increases air void content.

Experience gained through TTI work on SAS mixtures would indicate that the use of sulfur should minimize if not alleviate these problem areas. By coating the surface of the ash particles, a significant amount of the voids will be filled thus minimizing the need for additional asphalt. Sulfur is well known for its structural properties and therefore can be expected to strengthen the ash particles and thus enhance its crush resistance. As was stated above, SAS mixtures do not require compaction in the field and therefore could allow for higher bottom ash fractions in SMBA mix designs. Another of the beneficial features of SAS mixtures is that their stability and integrity were maintained with air voids as high as 15 percent (4.28). These features led to the incorporation of SAS technology into the study of SMBA mixtures in this program for use in both highway and airport pavement construction.

## MATERIALS

### Aggregates

A wide range of bottom ashes were used typical of those being generated along the east and central regions of Texas (see Figure 4-1). Table 4-1 gives the materials used in this study along with their sources. Bottom ashes were selected on the basis of their approximate location within District 17 (Bryan, Texas). The chemical and physical analyses of these materials are given in Appendix A.

Table 4-1. Materials Used in SMBA Asphalt Mixtures.

Material	Coal Source	Designation	Specific Gravity
Bottom Ash			
HL&P Limestone Plant	Lignite	HLBA	2.193
HL&P Parish Plant	Powder River, Wyoming	HLBA	2.306
TU-Electric Big Brown	Lignite	TUBBA	1.886
TU-Electric Montecello	Lignite	TUMBA	1.897
TU-Electric/ALCOA	Lignite	ALCOA/BA	2.054
Boiler Slag			
TU-Electric/ALCOA	Lignite	ALCOA/BS	2.539
Natural Aggregate			
Crushed Limestone	Young Brothers of Bryan, Texas	Crushed Limestone	2.505
Field Sand	Young Brothers of Bryan, Texas	Field Sand	2.630
Washed Screening	Young Brothers of Bryan, Texas	Washed Screening	2.638
Pea Gravel	Young Brothers of Bryan, Texas	Pea Gravel	2.498
Manufactured Sand	Young Brothers of Bryan, Texas	Manufactured Sand	2.550
Asphalt	Young Brother of Bryan, Texas	AC-20	1.020
Sulfur	Stauffer Chemical Washington, D.C.	S	2.000

## GRADATION OF BOTTOM ASHES

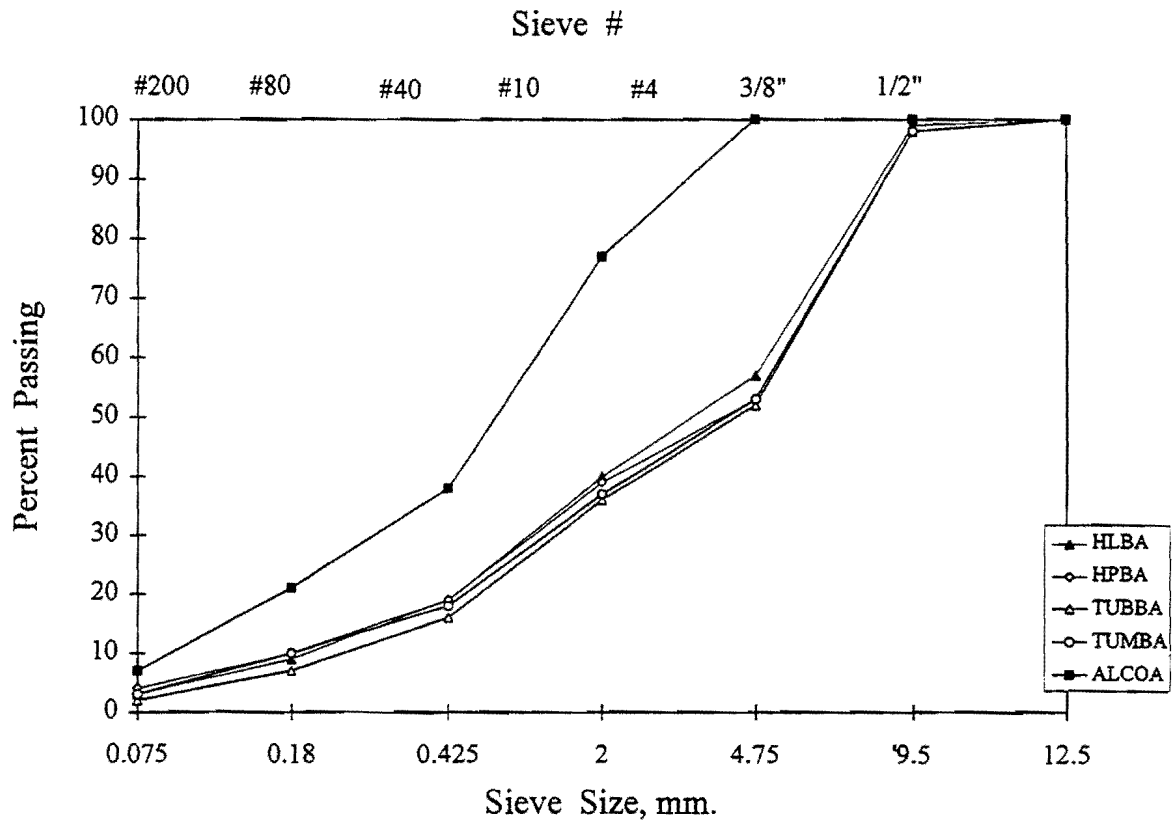


Figure 4-1. Gradation of Bottom Ashes.

A sieve analysis was run to compare the particle-size distributions of the five bottom ashes. Figure 4-1 shows the results. A comparison of the size distribution of ALCOA bottom ash and boiler slag is shown in Figure 4-2. Except for the ash generated at the TU-Electric/ALCOA plant in Rockdale, Texas, the rest of the bottom ashes were quite similar. The reason for this difference could be attributed to boiler temperature, particle size of the fuel, or a combination of the two.

Typical gradations of conventional aggregate systems used by Texas Department of Transportation are given in Table 4-2.

Table 4-2. Typical Gradations of Conventional Aggregate Systems.

Sieve size mm.(#)	Crushed Limestone	Field Sand	Manufactured Sand	TxDOT Type D Specifications
0.075 (#200)	0	4	6	2-8
0.180 (#80)	1	15	16	4-14
0.425 (#40)	1	20	33	11-26
2.000 (#10)	2	75	81	32-42
4.750 (#4)	14	100	100	50-70
9.500 (3/8")	97	100	100	85-100
12.5 (1/2")	100	100	100	100

Based on these gradations, various blends of ash and limestone were prepared and designed to determine the ratio that would most closely comply with the gradation of a Type D mix specification. The ratios which satisfied these requirements were as follows:

HLBA/limestone            50/50,  
 HPBA/limestone            45/55,  
 TUBBA/limestone          45/55, and  
 TUMBA/limestone          42/58.

## GRADATION OF ALCOA BOTTOM ASH AND BOILER SLAG

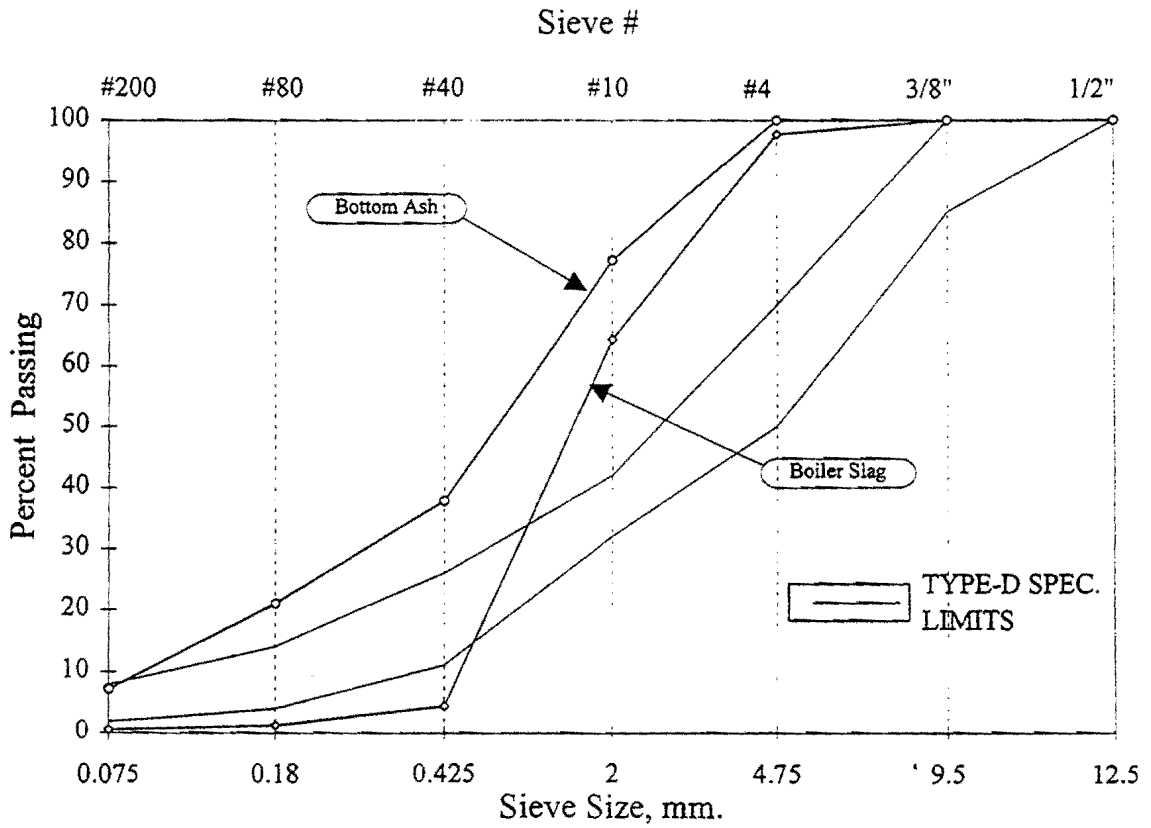


Figure 4-2. Gradation of ALCOA Bottom Ash and Boiler Slag.



## MODIFIED LOS ANGLES (LA) ABRASION TEST

As stated above, bottom ashes are friable and therefore highly susceptible to crushing during handling and mixing operations at the plant, transporting to the job site, or while under compacting by construction equipment or traffic. The LA Abrasion Test is normally used to assess interparticulate abrasion resistance by subjecting the aggregate to the pounding action of spherical steel balls in a rotating drum. This test would be too severe for bottom ash and therefore was modified to better simulate the interaction of ash and crushed stone in a drum plant. The blends of ash and limestone given above were placed into the metallic cylinder (without the steel balls) and rotated for 10 minutes. The gradations before and after the test are shown in Figure 4-3 through 4-6. Since neither the ALCOA bottom ash nor boiler slag alone conformed to a Type D specification, a 75/25 blend of ash to slag was prepared with the latter replacing the limestone. Figure 4-7 shows the gradation for this blend.

## MIX DESIGNS

### Paris District Mix Designs

A program to establish the optimum mix design for the various bottom ash blends was performed. The results were compared with two mix designs utilized in the Paris District (4-32, 4-33): designated I-30 and control mixtures, respectively.

Table 4-3. I-30 Mix Design Using Fly Ash.

<u>Aggregate</u>	<u>I-30</u>	<u>Weight Percent Control (w/o)</u>
Bottom Ash	36	0
Limestone	53	59.4
Field Sand	0	13.6
Washed Screening	0	20.8
AC-20	11	6.2
Air Void Content, %	0	0
Hveem Stability, %	0	0
Unit Weight, kg/m <sup>3</sup>	0	0

# HLBA/LIMESTONE TYPE D GRADATION (BEFORE AND AFTER "MODIFIED" LA ABRASION)

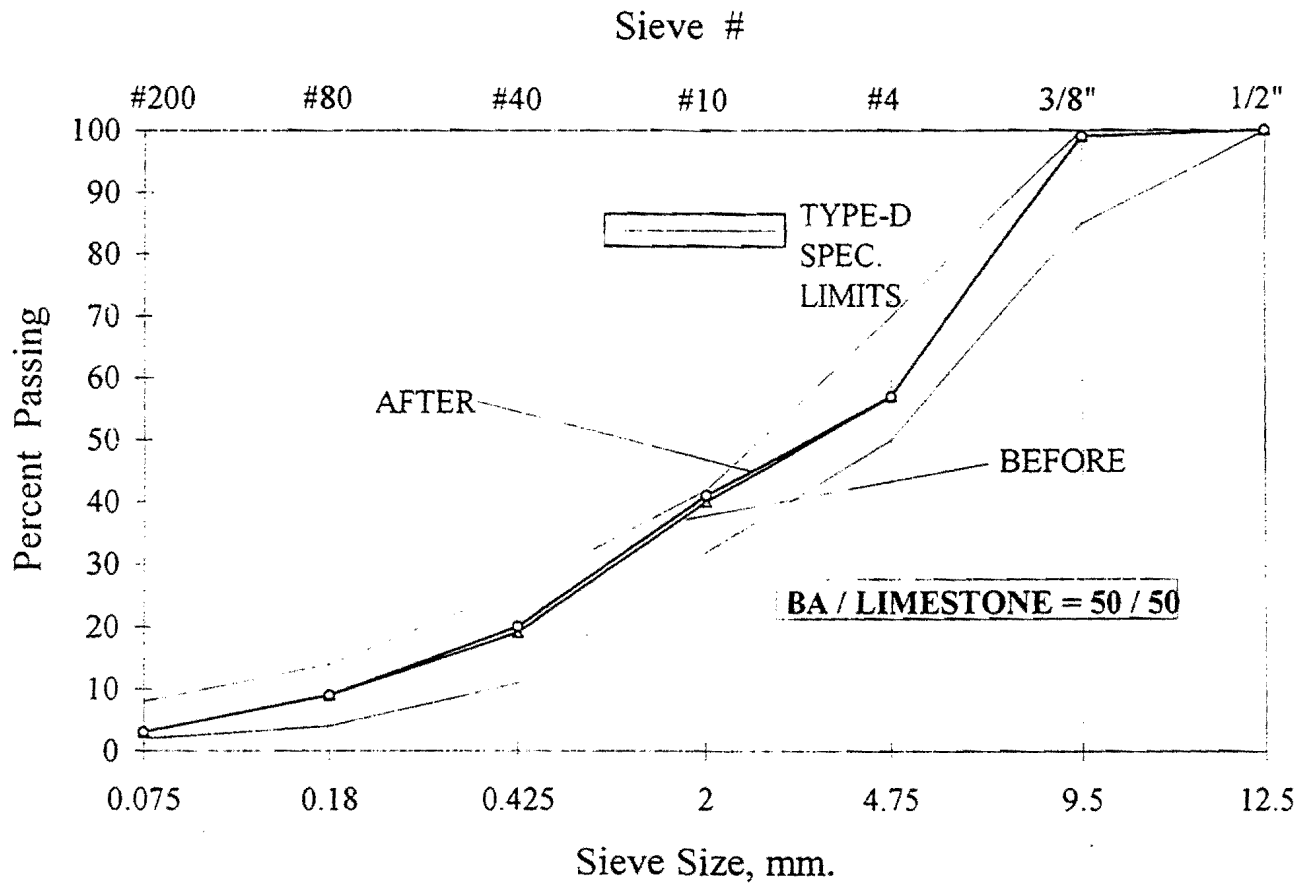


Figure 4-3. HLBA/Limestone Type D Gradation.

# HPBA/LIMESTONE TYPE D GRADATION (BEFORE AND AFTER "MODIFIED" LA ABRASION)

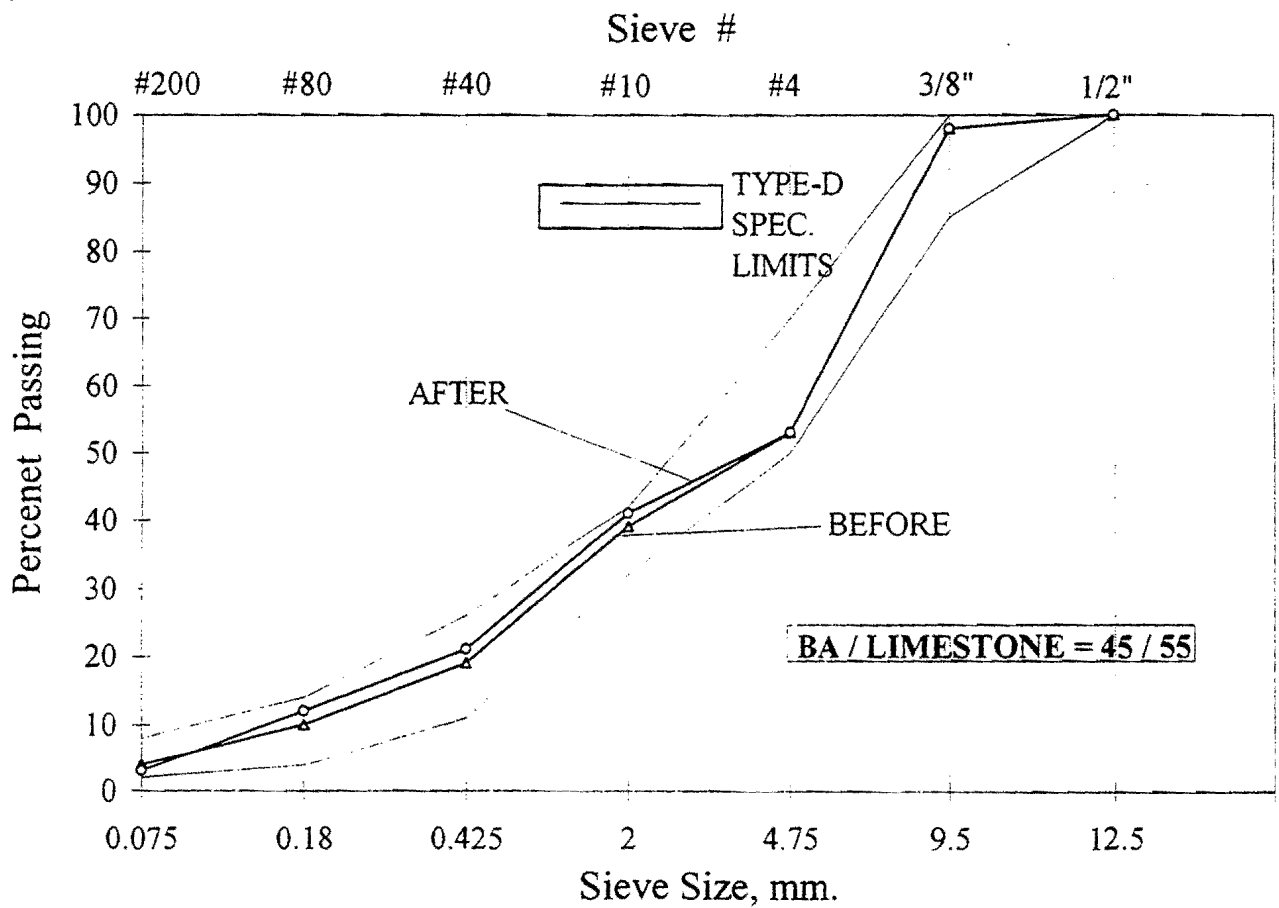


Figure 4-4. HPBA/Limestone Type D Gradation.

## TUBBA/LIMESTONE TYPE D GRADATION (BEFORE AND AFTER "MODIFIED" LA ABRASION)

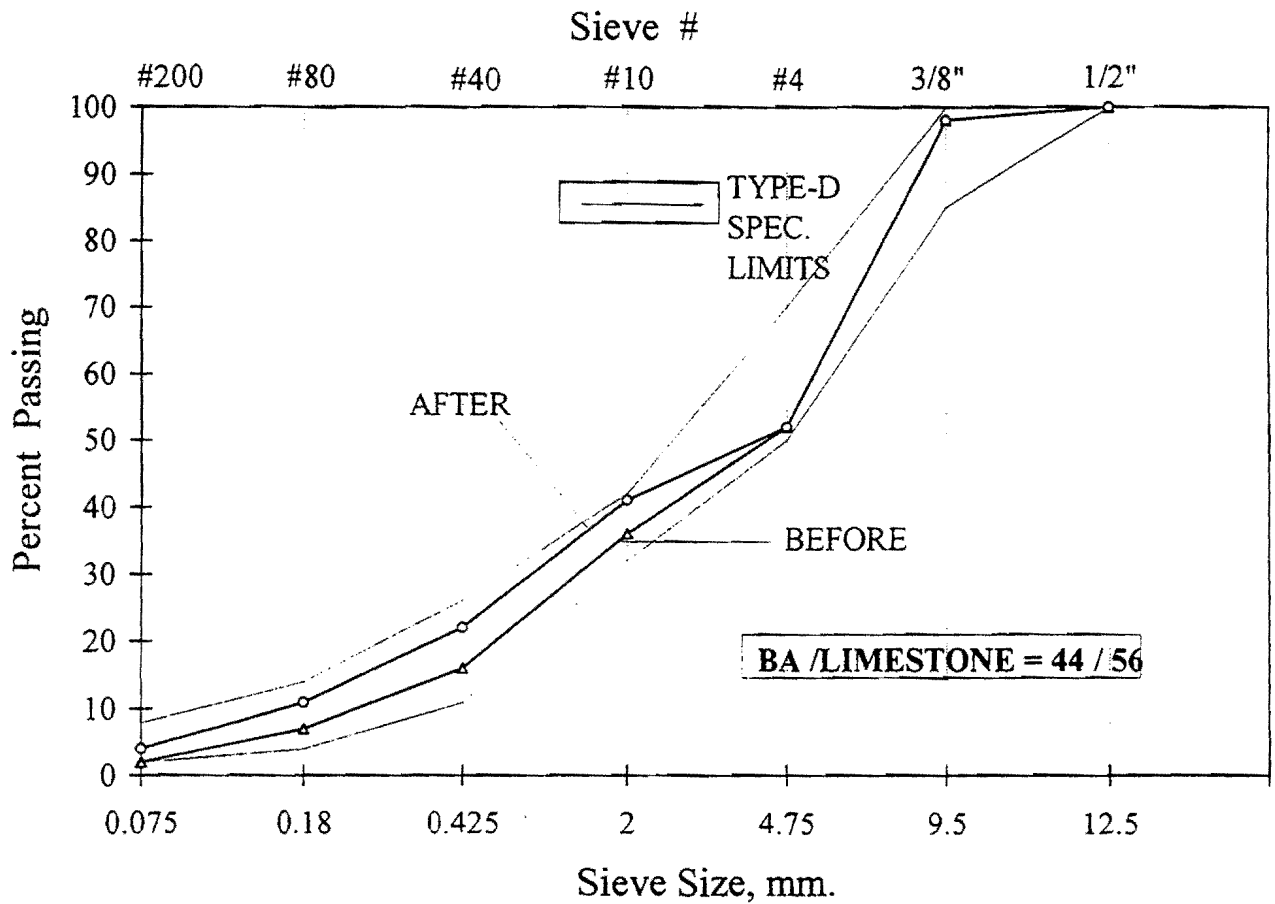


Figure 4-5. TUBBA/Limestone Type D Gradation.

# TUMBA/LIMESTONE TYPE D GRADATION (BEFORE AND AFTER "MODIFIED" LA ABRASION)

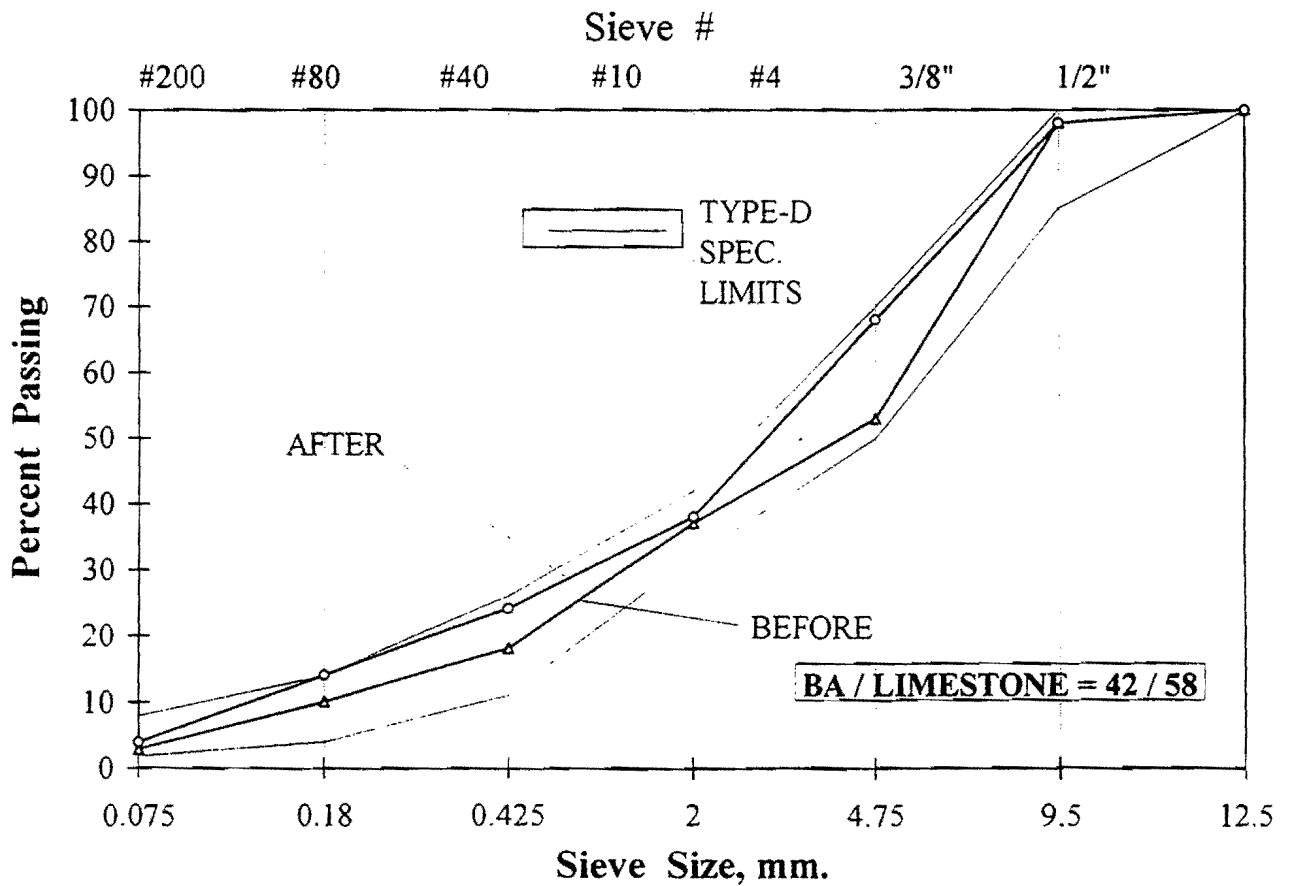


Figure 4-6. TUMBA/Limestone Type D Gradation.

**GRADATION OF ALCOA MIXTURE  
(75% BOTTOM ASH AND 25% BOILER SLAG)**

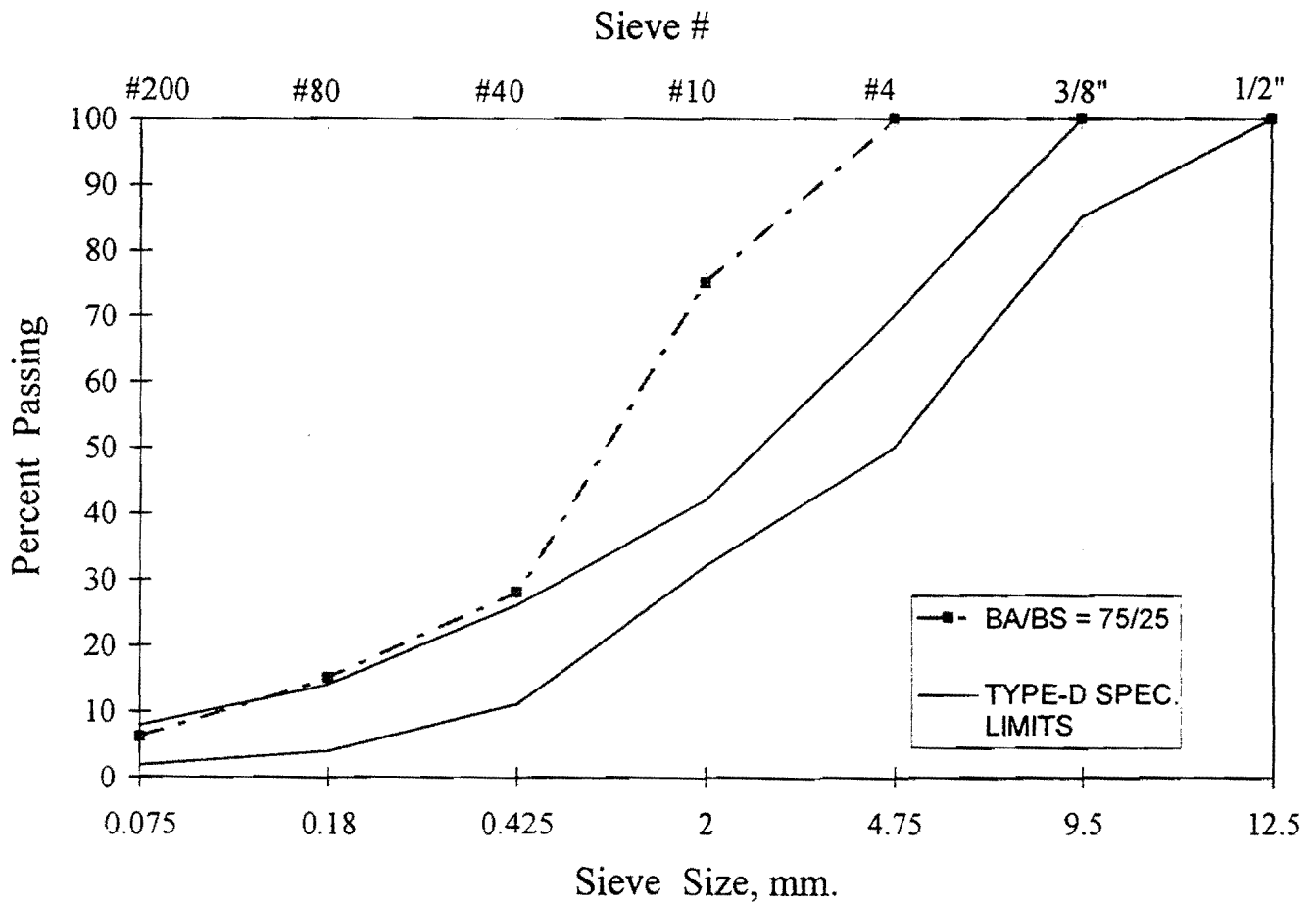


Figure 4-7. Gradation of ALCOA Mixture.

### **Sulfur Modified Bottom Ash (SMBA) Mix Design**

The preparation of SMBA asphalt concrete materials involved a two-cycle process as shown in Figure 4-8. The operation begins with all three ingredients: aggregate, asphalt, and sulfur preheated to a temperature above the melting point of sulfur (116°C) and below 150°C. The upper limit is the temperature above which sulfur undergoes an abrupt and very large increase in viscosity that could adversely affect mix workability. Although acceptable mixes have been prepared at temperatures as high as 193°C, 150°C is considered to be the maximum mix processing temperature for averting toxic fumes in the emission (4.14, 4.16). The optimum mix design for SMBA mixtures was established by preparing mixtures over a range of asphalt contents at the BA/Limestone ratios given above. Initially, these mixtures were prepared without sulfur. A second series of mixtures were prepared with 12 weight percent of sulfur added. The mix designs were compared for their compliance with Type D specifications as to air void content (2 to 4 percent) and Hveem Stability (32 percent minimum). A comparison was also made on the basis of unit weight.

A comparison of the air voids as a function of asphalt content for the various mixtures is shown in Figures 4-4 through 4-9. They illustrate the relatively high air voids inherent in bottom ash mixtures. Only the HL&P Parish material approached Type D criterion at 9 percent asphalt.

The reduction in air voids achieved by the addition of 12 weight percent sulfur is given in Figure 4-10 for HL&P ashes and in Figure 4-11 for TU-Electric bottom ashes. All mixtures appear to be capable of achieving acceptable Type D specified limits for air void content with about 7 weight percent asphalt.

Hveem Stability with and without sulfur modification are shown in Figures 4-12 and 4-13. All mixtures achieved stability values above 32 percent as required by Type D specifications. However, the introduction of sulfur tended to further increase Hveem Stability.

Figures 4-14 and 4-15 reflect the increase in unit weight achieved with SMBA asphalt concrete mixtures. This was to be expected due to the high specific gravity of sulfur as it fills the voids in the ash.

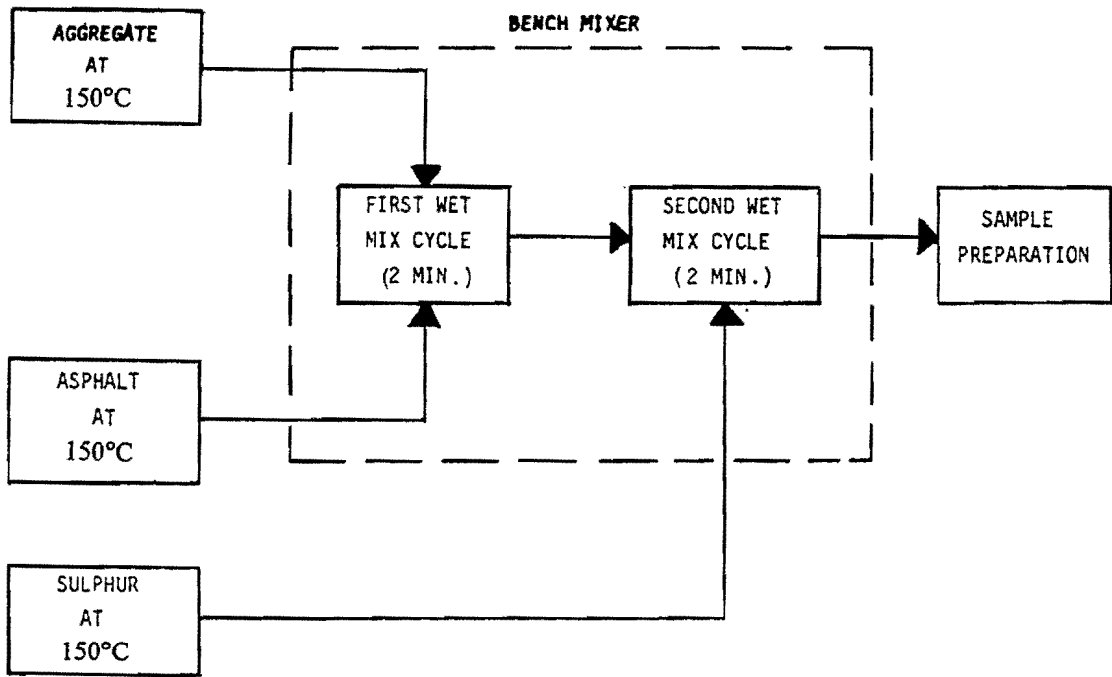
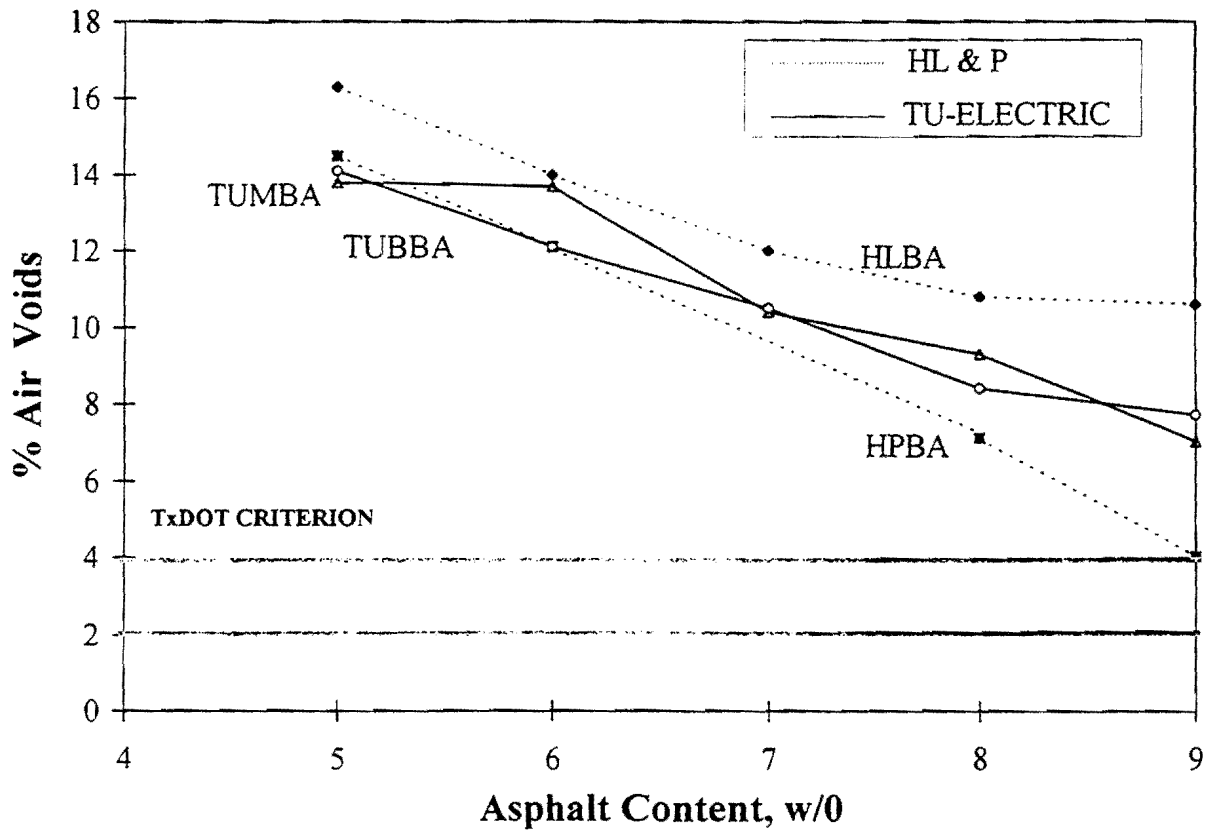


Figure 4-8. Schematic of SMBA Asphalt Concrete Mixture Preparation.



## TYPE D MIX DESIGN (0% SULFUR)



4-9. Type D Mix Design.

# HL&P TYPE D MIX DESIGN (0% AND 12% SULFUR)

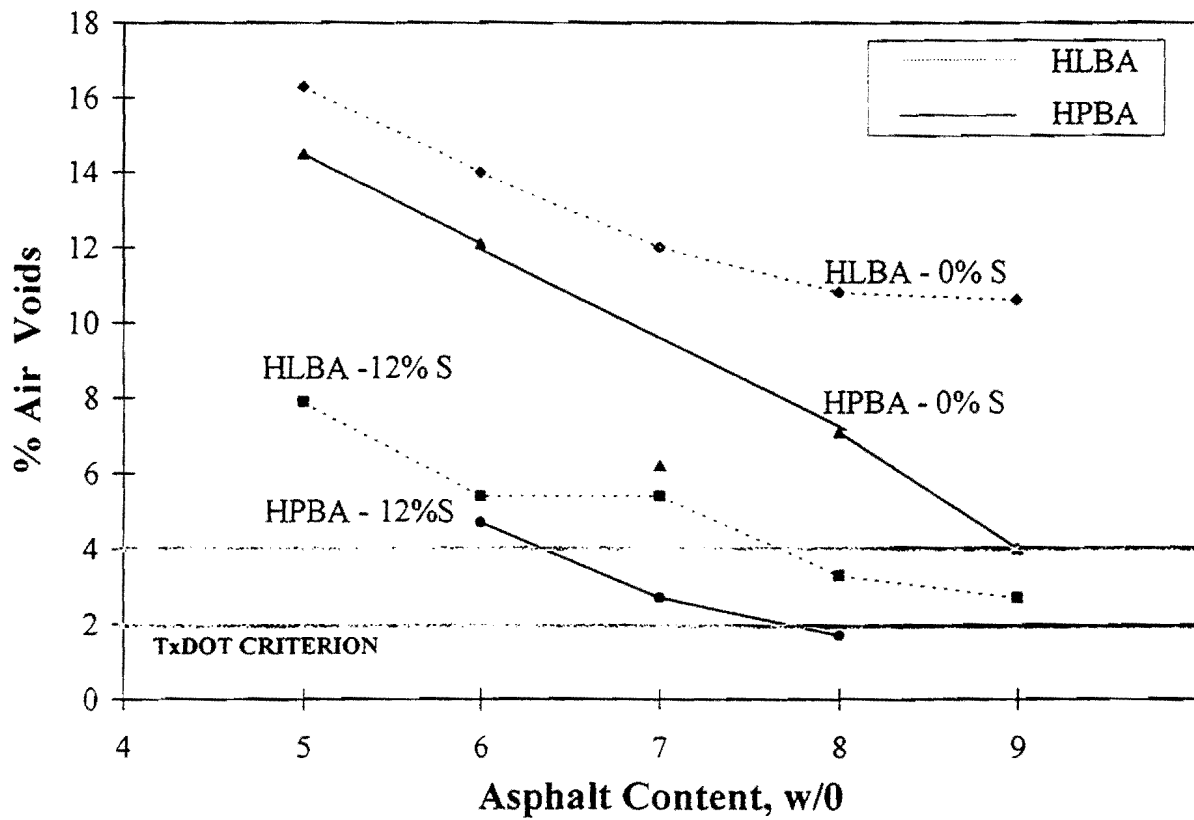


Figure 4-10. HL&P Type D Mix Design.

# TU ELECTRIC TYPE D MIX DESIGNS (0% AND 12% SULFUR)

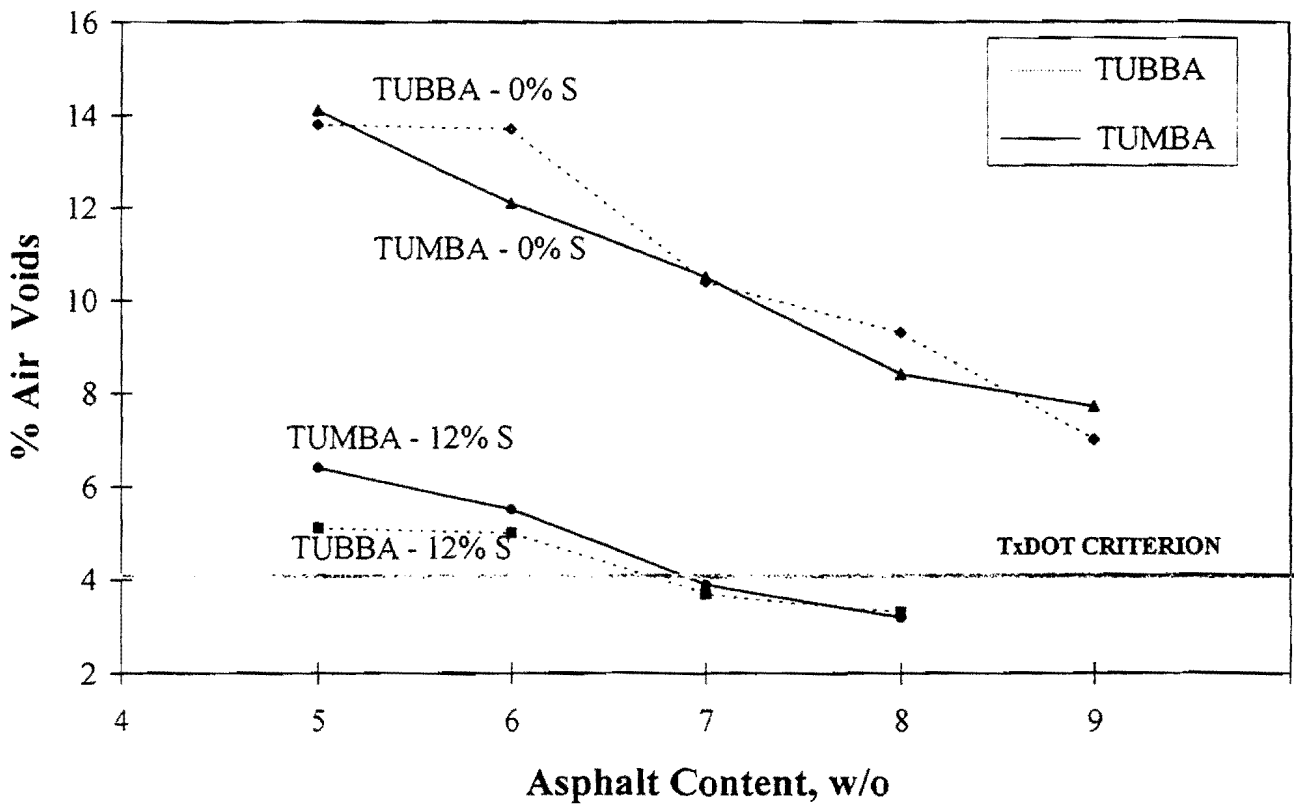


Figure 4-11. TU Electric Type D Mix Designs.

## HL&P TYPE D MIX DESIGNS (0% AND 12% SULFUR)

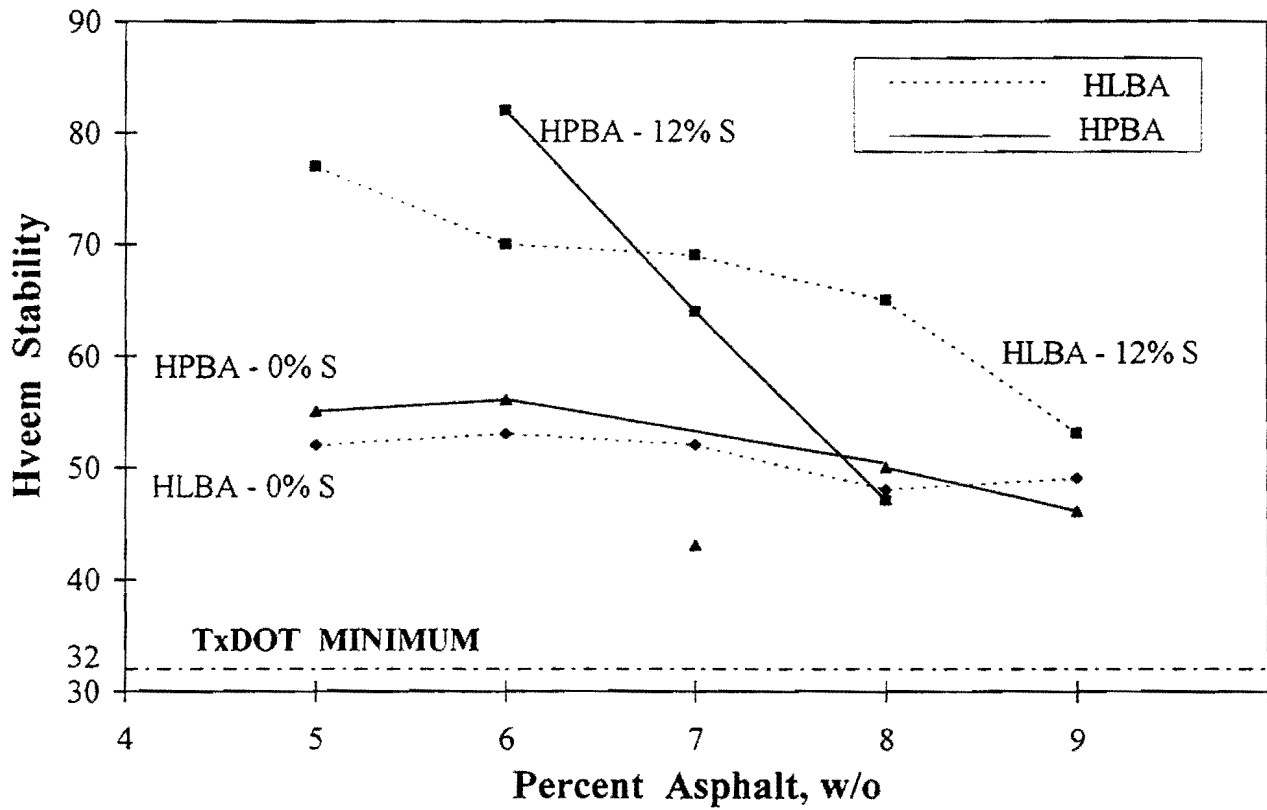


Figure 4-12. HL&P Type D Mix Designs.

# TU ELECTRIC TYPE D MIX DESIGNS (0% AND 12% SULFUR)

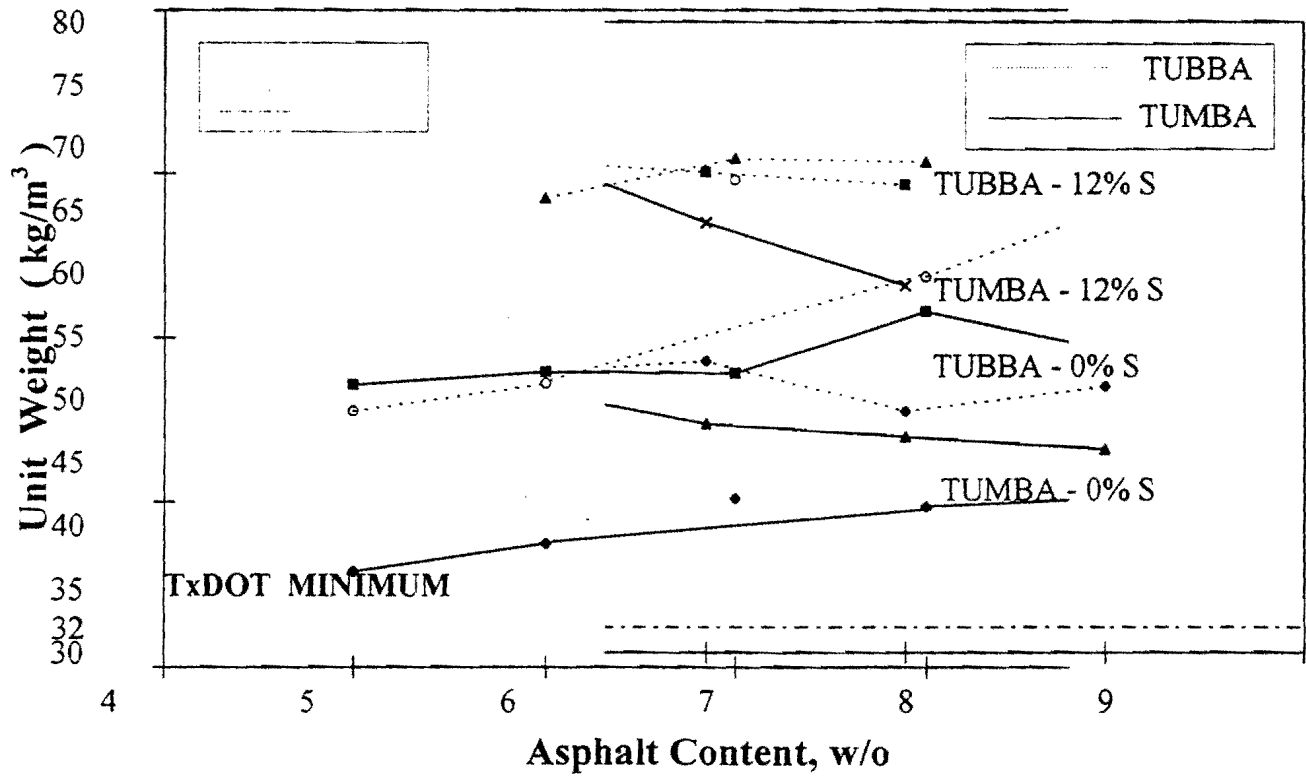


Figure 4-13. TU Electric Type D Mix Designs.

# HL&P TYPE D MIX DESIGNS (0% AND 12% SULFUR)

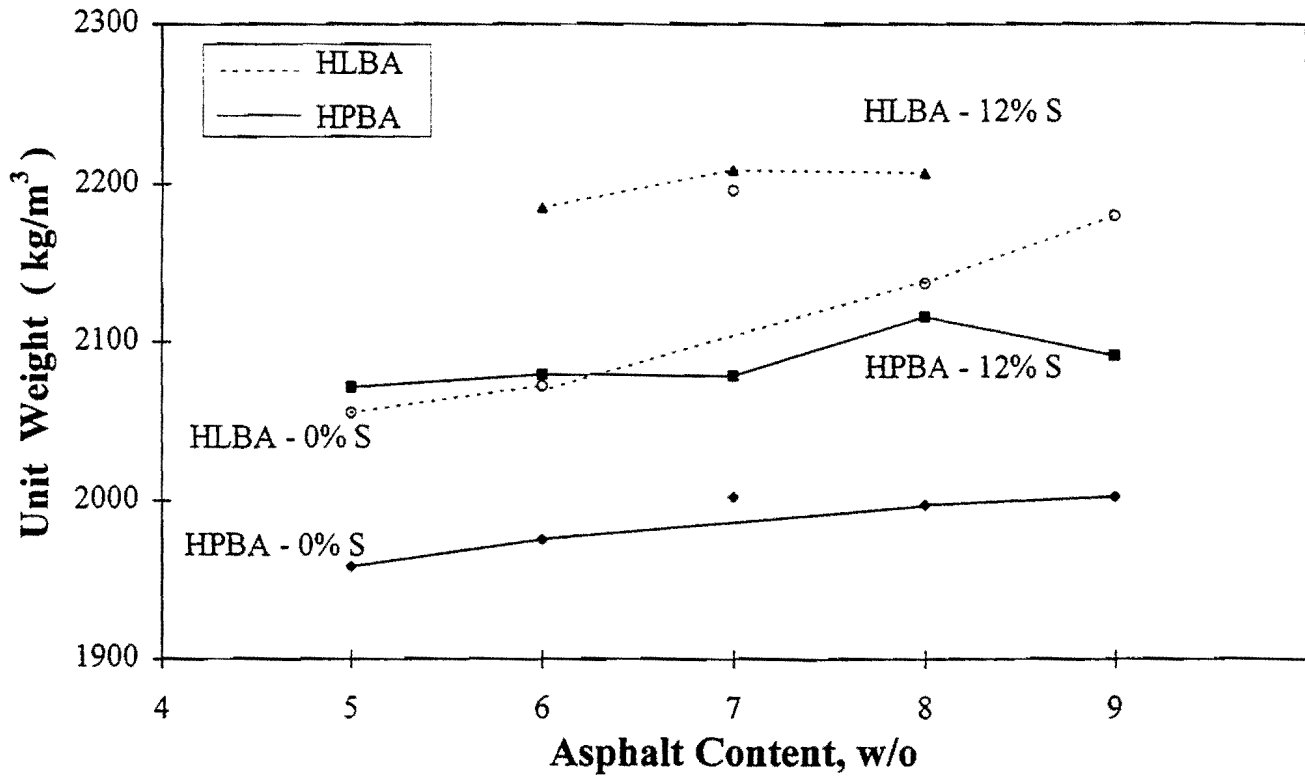


Figure 4-14. HL&P Type D Mix Designs.

## TU ELECTRIC TYPE D MIX DESIGNS (0% AND 12% SULFUR)

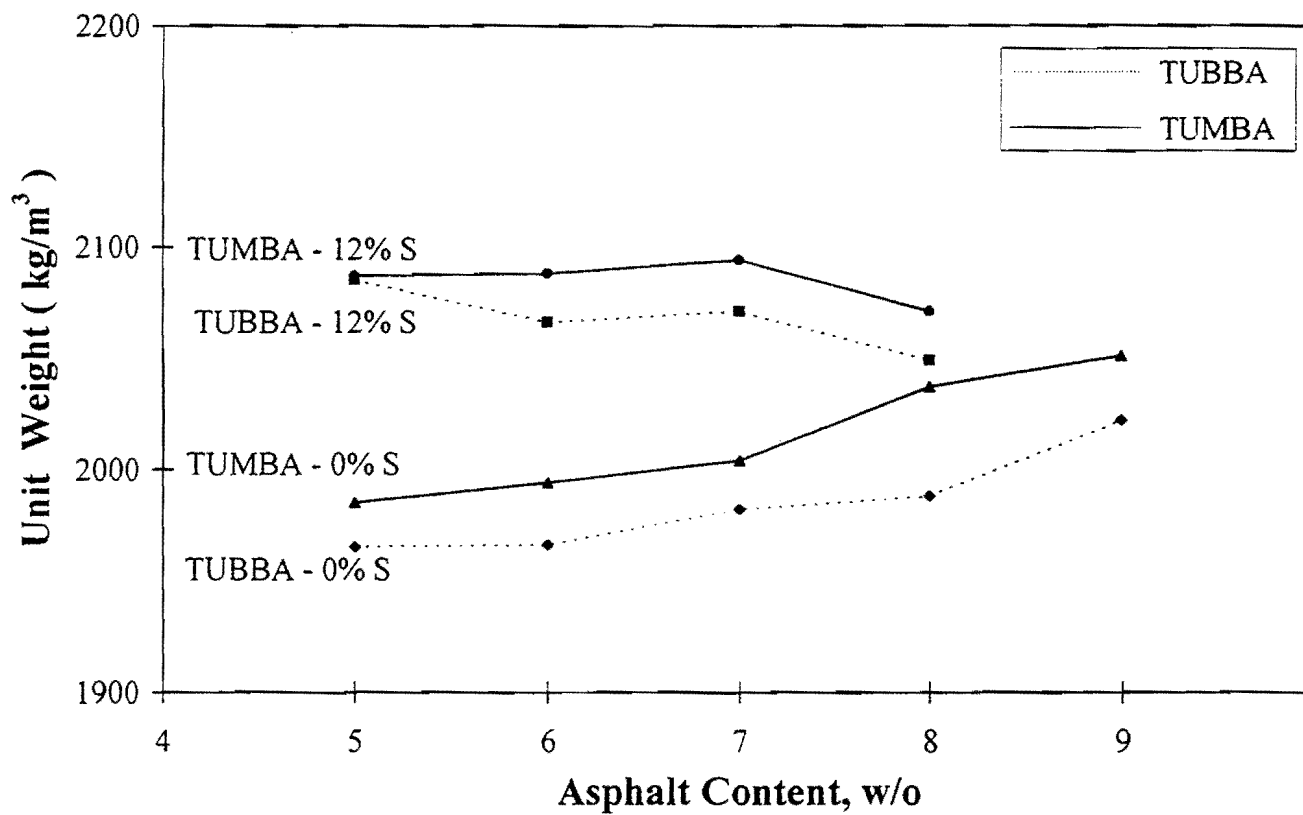


Figure 4-15. TU Electric Type D Mix Designs.

In the mixtures just discussed, the maximum bottom ash content ranged from 42 to 50 percent. An attempt was made to extend this fraction to 100 percent bottom ash using a 75/25 blend of ALCOA bottom ash to boiler slag as the sole aggregate component. To minimize air voids, the sulfur content of the mix was raised from 12 to 20 percent and the asphalt content was set at 8 percent. The properties generated are given below and compared with the control mixture.

	ALCOA	Control
Hveem Stability, %	89	30
Air Voids, %	14.2	3.5
Unit Weight, kg/m <sup>3</sup>	1807	2800

Figures 4-16 and 4-17 give the above properties. The optimum asphalt content for the I-30 and control mixtures and for the SMBA mixtures are given in Figure 4-18. The asphalt contents are shown in both weight (w/o) and volume (v/o) percents. A significant savings in asphalt over the bottom ash/asphalt mixture placed on I-10 was realized by sulfur modification. Because of the lower unit weight of SMBA mixtures, their asphalt demands were about the same as for the control.

#### **CHARACTERIZATION TESTING OF SMBA ASPHALT CONCRETE MIXTURES**

Four tests were used to better reflect the performance characteristic of SMBA mixtures:

1. Resilient modulus temperature susceptibility,
2. Indirect tension fatigue resistance,
3. Lottman freeze-thaw durability resistance to thermal and fatigue fracture, and
4. Static creep moisture susceptibility, freeze-thaw durability, resistance to rutting.



**ALCOA**  
**% ASPHALT versus % AIR VOIDS**  
**(BA/BS = 75/25 @ 20% SULFUR)**

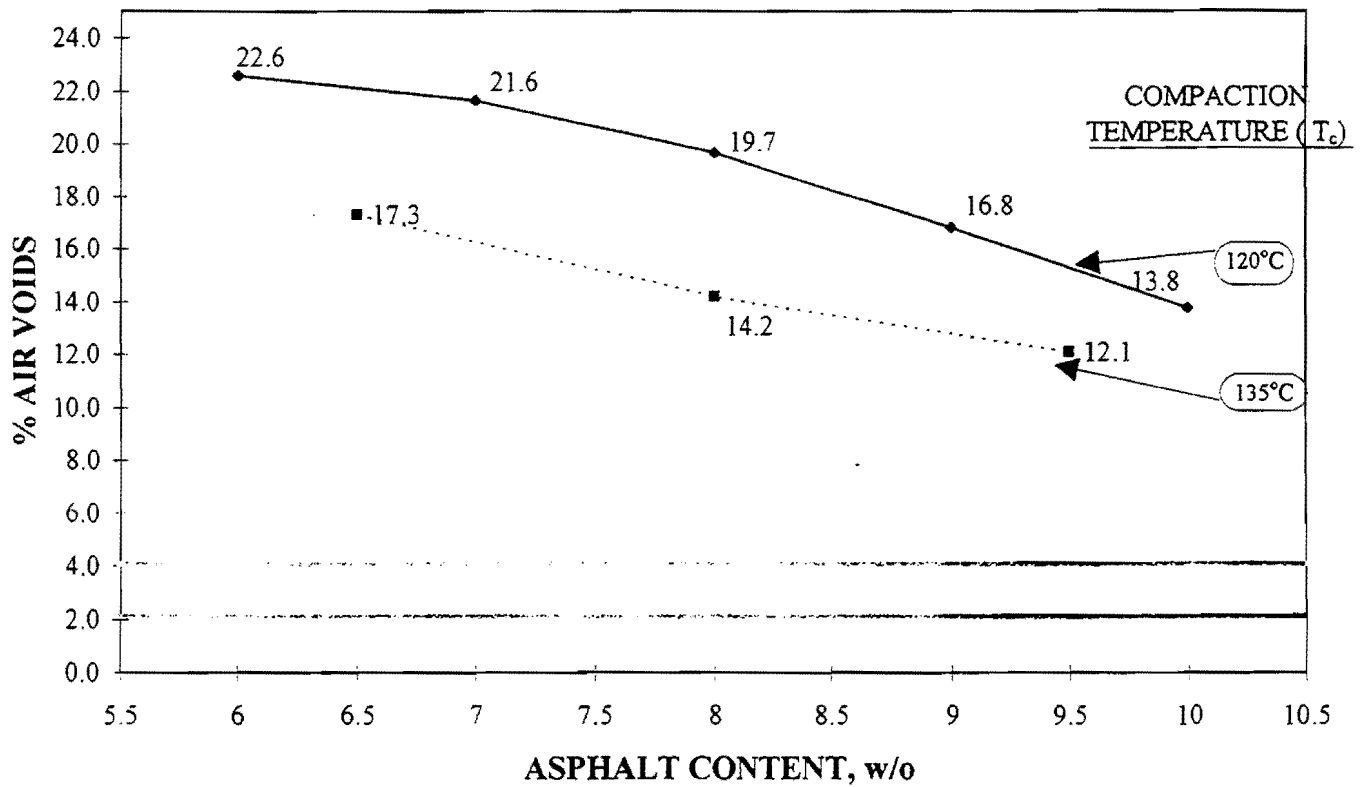


Figure 4-16. Effect of Compaction Temperature on Air Void Content of ALCOA SMBA Mixtures.

ALCOA  
UNIT WEIGHT versus % ASPHALT  
(BA/BS = 75/25 @ 20% S, T<sub>c</sub> = 135 °C)

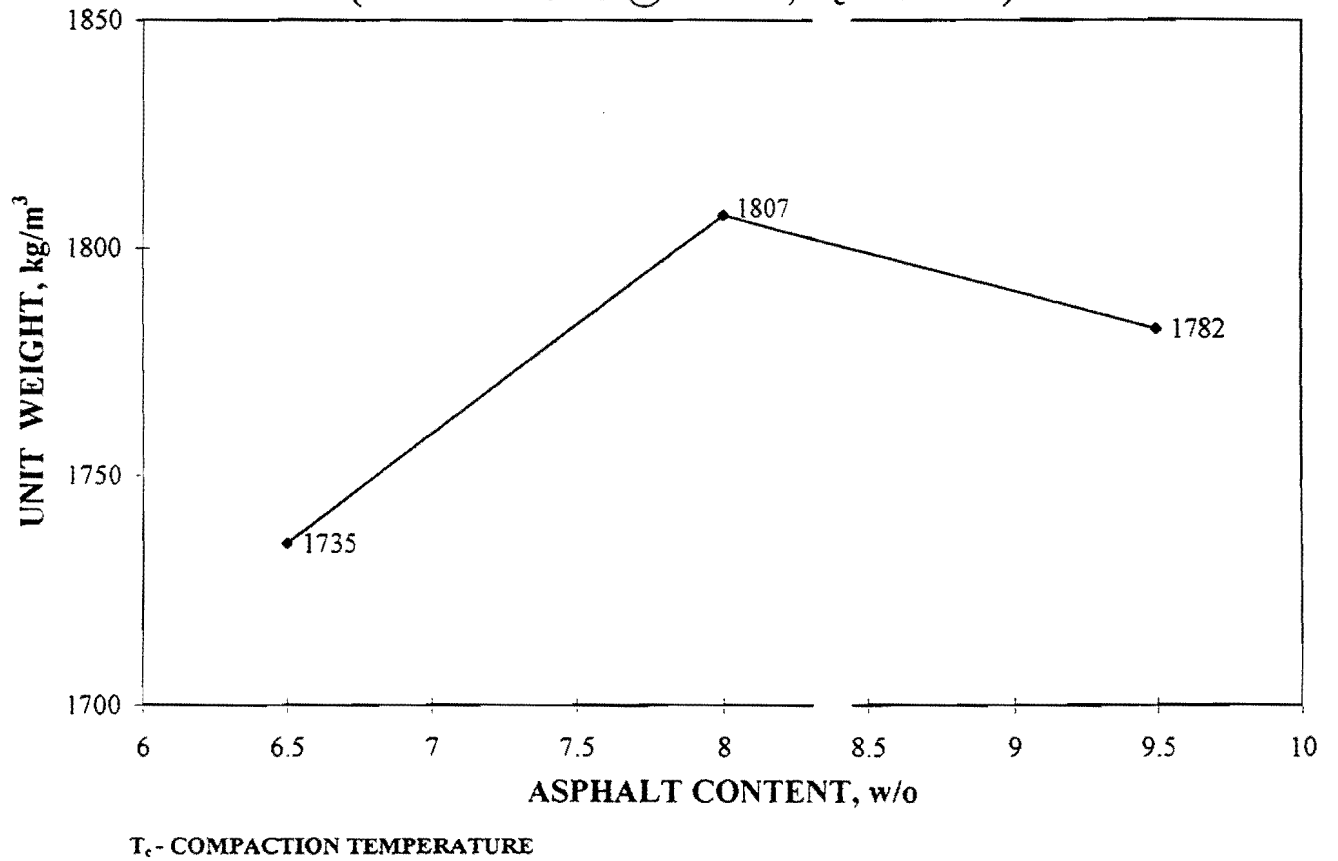
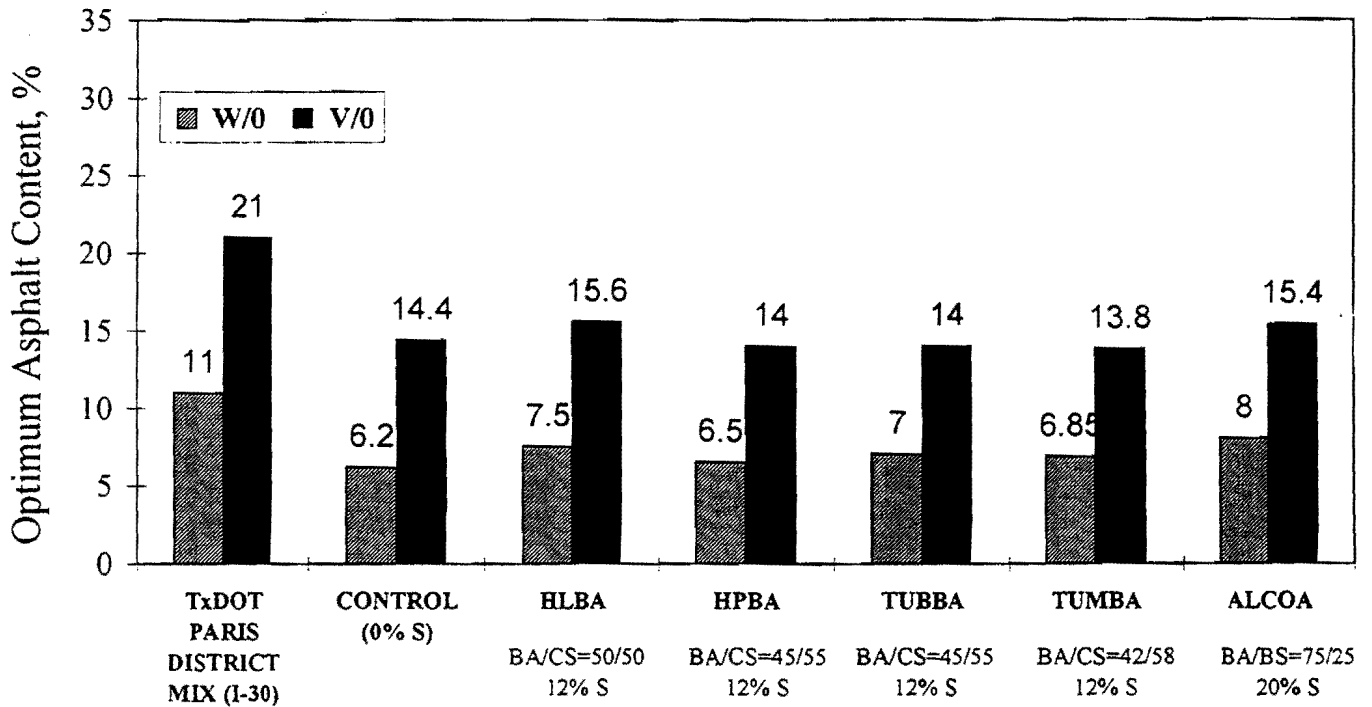


Figure 4-17. Unit Weight of ALCOA SMBA Mixtures.

## OPTIMUM MIX DESIGNS OF SMBA MIXTURES (w/o AND v/o)\*



w/o - Weight %  
v/o - Volume %

Figure 4-18. Optimum Asphalt Content for SMBA Mixtures.

### **Resilient Modulus ( $M_R$ ) Test**

The resilient modulus ( $M_R$ ) test was conducted according to ASTM D 4123. Three specimens were tested at each of three temperatures (3.3°, 25°, and 40°C). A diametral load of 334 N was applied for 0.1 seconds along two diametral axes during which the lateral deformations of the specimens were recorded. The total resilient moduli values were evaluated using the National Cooperative Highway Research Program (NCHRP) asphalt-aggregate mixture analysis system (AAMAS) criteria (4.34).

Specimens for this test were prepared using the Texas gyratory compactor (Texas Test Method: Texas 206F). The average height of the specimen was 50.8 mm with a diameter of 101.6 mm. The specimens were compacted at a stress level of 1,035 kPa with a final compaction load of 11.25 kN.

The resulting  $M_R$  values for the five SMBA mixtures were plotted with respect to their test temperatures and are shown in Figures 4-19 through 4-21. These data were compared with allowables established by AAMAS (4.34). The area between the two lines represents acceptable values for temperature susceptibility and resistance to fatigue.

The results indicate compliance with AAMAS criteria at low and moderate temperatures but too stiff at higher temperatures. It should be noted that the control mixtures did not meet the criteria at high temperatures. If the generally accepted hypothesis that lower moduli are preferred at low temperatures and higher moduli are desired at high temperature is applied, the SMBA mixtures would offer the better choice over the control. It should also be noted that the ALCOA (i.e., 100 percent bottom ash) SMBA mixture would best satisfy this rationale.

### **Indirect Tension Test**

Indirect tension test is a method used to indicate the resistance of an asphaltic concrete mixture to thermal and fatigue cracking. Indirect tension tests were conducted in accordance with AAMAS procedures (4.34). The samples were tested at 25°C and loaded to failure. In this test, a constant-rate, compressive load was applied along the diametral axis of a preconditioned specimen. The diametral horizontal deformation during the entire loading

### Temperature Susceptibility Curves for HL&P SMBA Mixtures - HLBA and HPBA

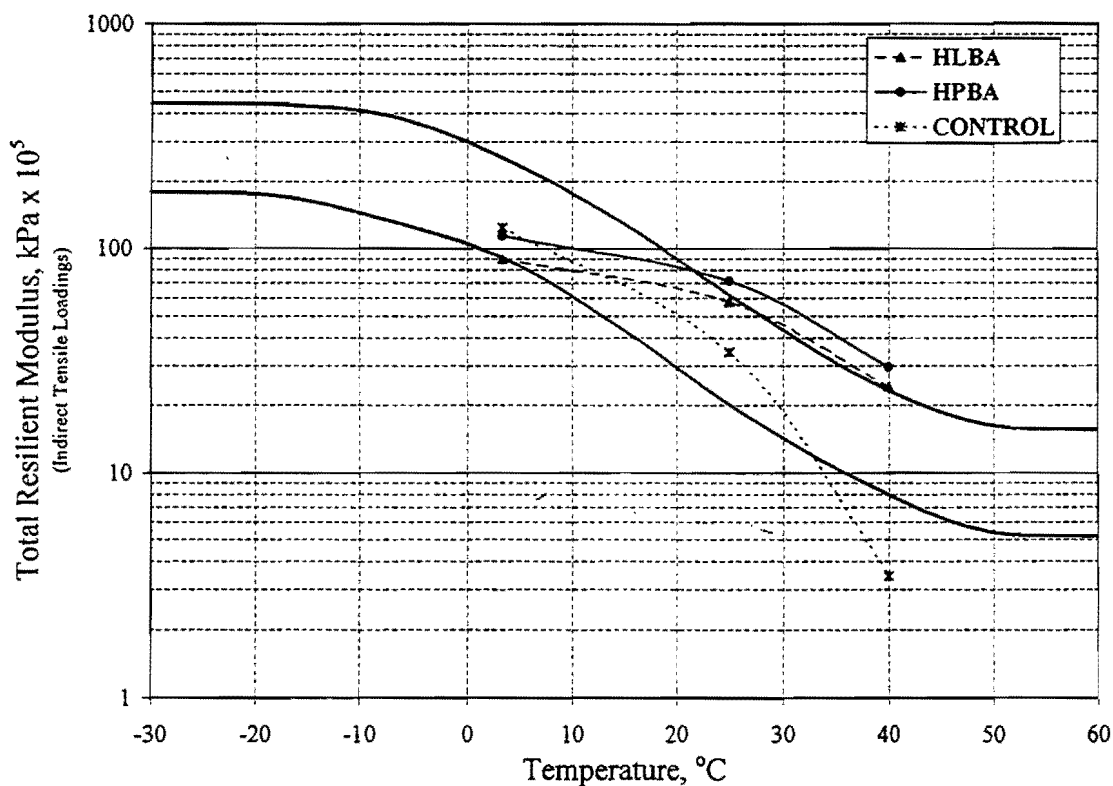


Figure 4-19. Temperature Susceptibility Curves for HL&P SMBA Mixtures.

**Temperature Susceptibility Curves for  
TU Electric SMBA Mixtures - TUBBA and TUMBA**

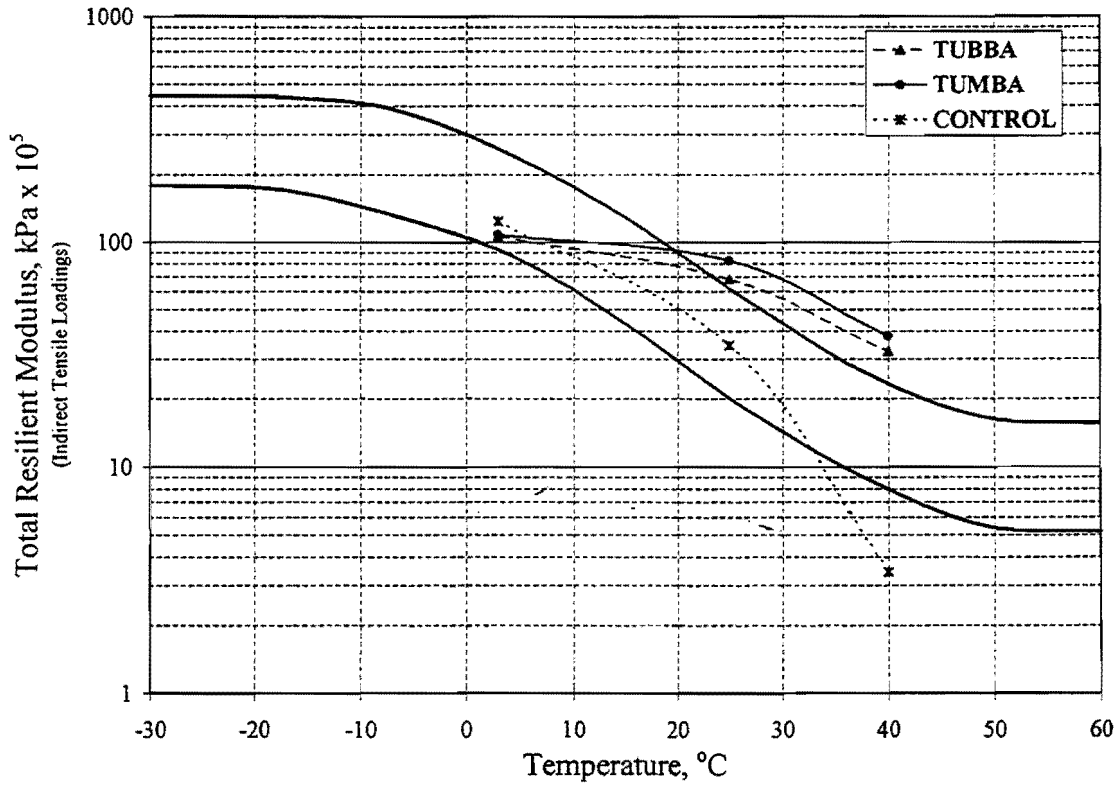


Figure 4-20. Temperature Susceptibility Curves for TU-Electric SMBA Mixtures.

### Temperature Susceptibility Curves for ALCOA SMBA Mixtures

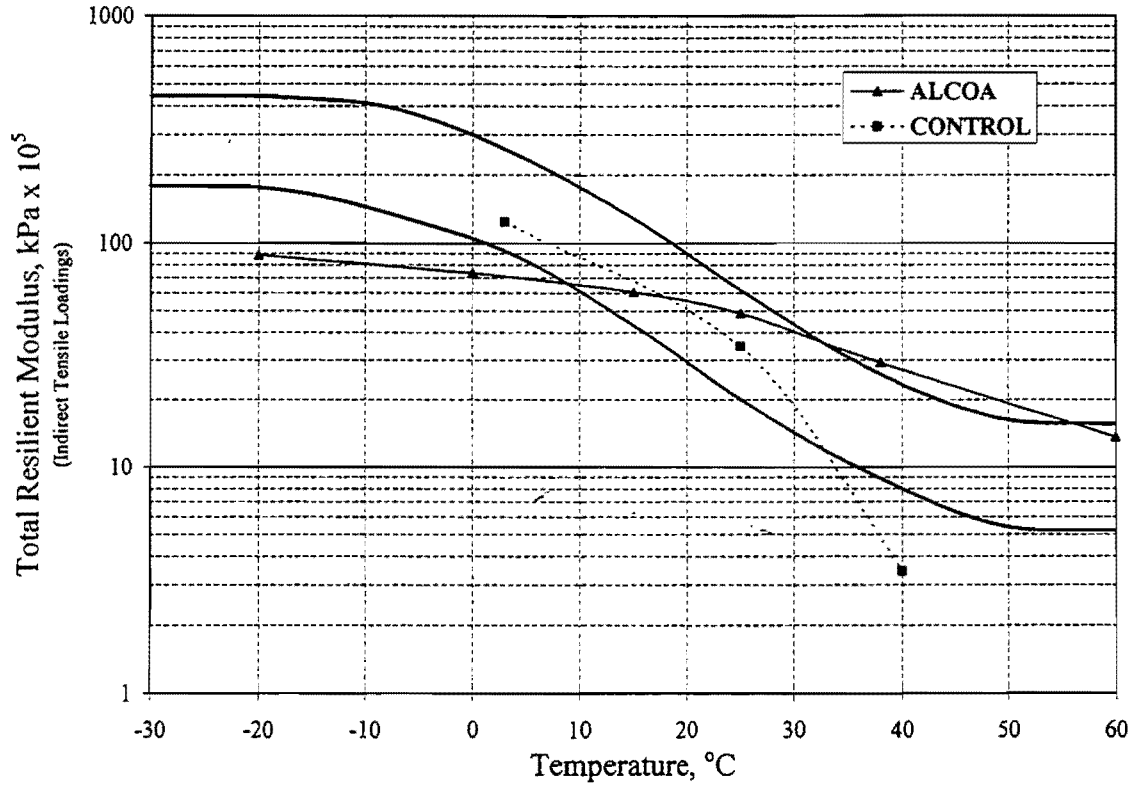


Figure 4-21. Temperature Susceptibility Curves for ALCOA SMBA Mixtures.

time, or until the load sustained by the specimen begins to decrease, was recorded. The maximum load and the load at failure were also recorded. The tensile stress and strain at the breaking point were calculated. The applied rate of deformation was 5018 mm/min at a temperature of 25°C.

Specimens for this test were prepared using the Texas gyratory compactor (Texas Test Method: Texas 206F). The average height of the specimen was 50.8 mm with a diameter of 101.6 mm. The specimens were compacted at a stress level of 1035 kPa with a final compaction load of 11.25 kN.

Figures 4-22 and 4-23 show the indirect tensile strengths of the five SMBA mixtures and the control. Except for the HLBA mixtures, strengths were either equal to or greater than the control. However, when plotted against FHWA (4.35) and NCHRP (4.36) criteria for acceptable fatigue resistance, none of the SMBA mixtures complied. The control mixture came the closest to meeting both criteria.

### **Lottman Freeze-Thaw Moisture Conditioning Test**

The Lottman freeze-thaw procedure exposes asphaltic concrete mixtures to vacuum saturated freeze-thaw conditions. This method simulates accelerated highway field conditions. It examines moisture effect on compacted asphaltic mixtures.

The Lottom freeze-thaw procedure was done according to TxDOT Texas Test Method Tex-531C. In this test the specimen was vacuum saturated between 60 to 80 percent. The specimen was frozen and then thawed in a 60°C water bath for 24 hours. The moisture-conditioned specimen was then tested in indirect tension, and the tensile strength and strain were recorded. From the recorded data, the tensile strength ratio of the moisture-conditioned and unconditioned specimens (i.e., ratio of strength before (dry) and after (wet) testing) was computed. A ratio about 75 percent is required to pass this test.

Specimens for this test were prepared using the Texas gyratory method (Texas Method 206 F), but with lower compaction effort than required for regular gyratory specimens. The average specimen height was 50.8 mm with a minimum diameter of 4 inches. The specimens were compacted at a stress level of 345 kPa with a final seating load of 4450 N. The gyrations were continued until air voids between 6 and 8 percent were achieved.



## TENSILE STRENGTH VALUES OF SMBA MIXTURES

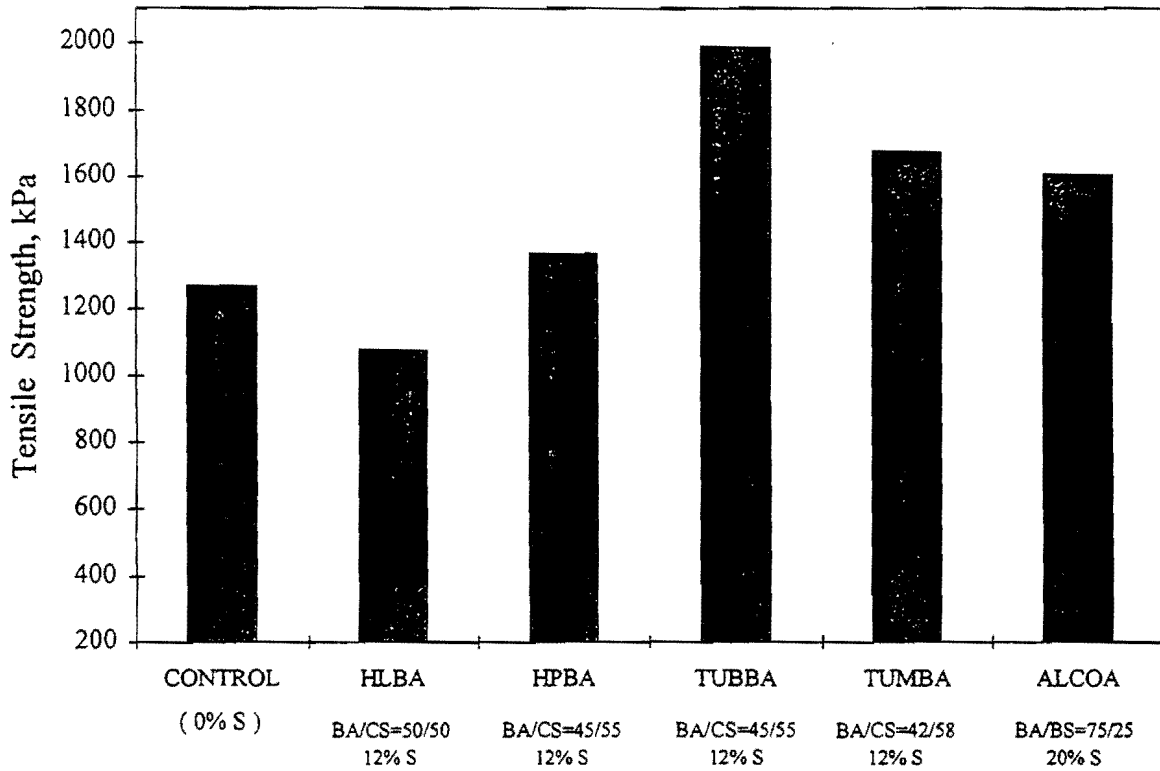


Figure 4-22. Tensile Strength Values of SMBA Mixtures.

## TENSILE STRAINS OF SMBA MIXTURES AT 25°C

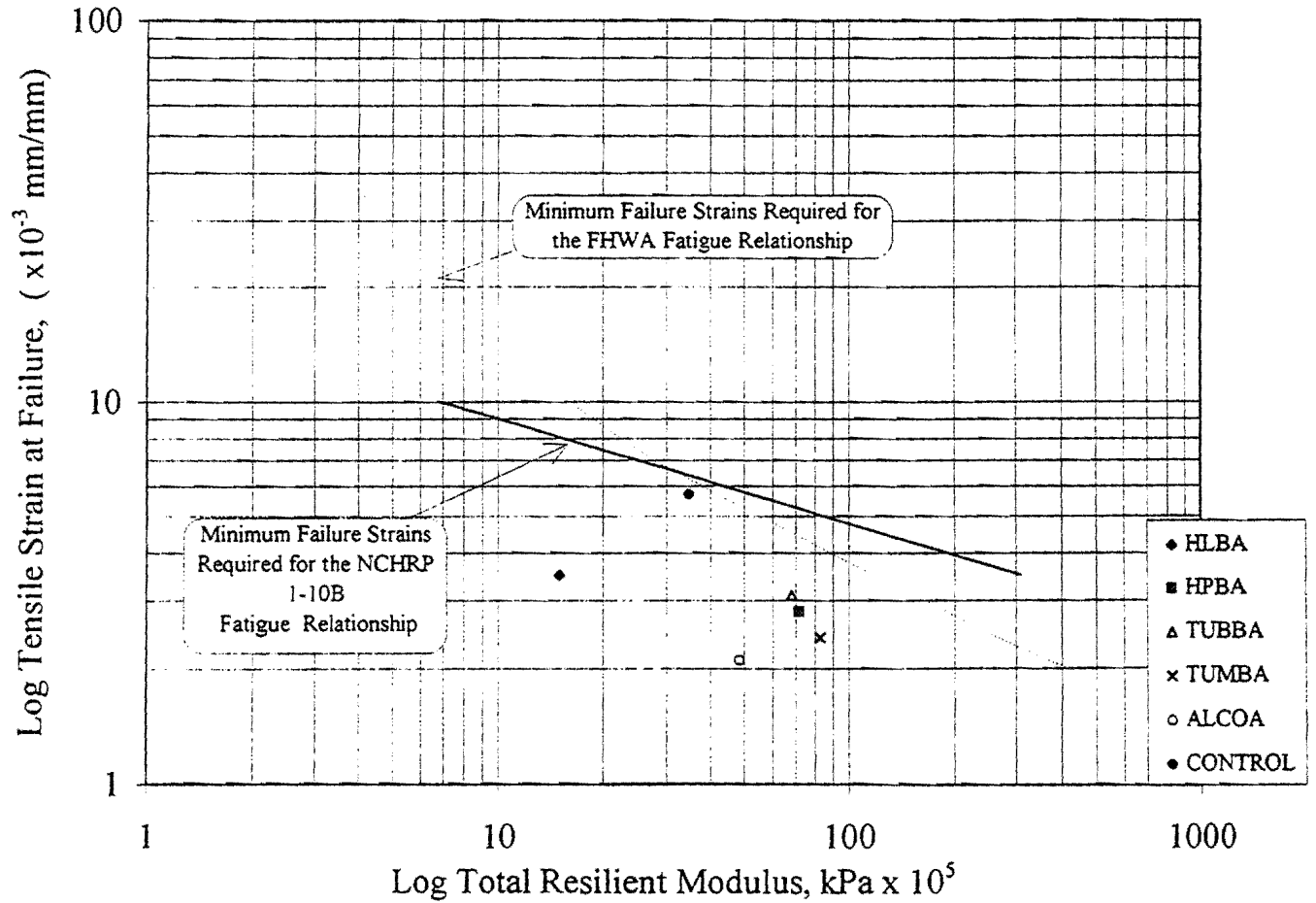


Figure 4-23. Tensile Strains of SMBA Mixtures at 25°C .

Figures 4-24 and 4-25 show the results of the Lottman test. Not all mixtures had the same degree of recovery from the freeze-thaw experience of this test. Figure 4-25 indicated the best performance was achieved with the HLBA and TUBBA mixtures. The TUMBA and ALCOA mixtures were only slightly below the TxDOT minimum ratio of 75 percent. Using this same criterion, the control mixture would not have passed. The poorest showing came from the HPBA mixture. It should be noted that this was the only SMBA mixture whose ash was generated from Wyoming Powder River Basin coal. The others were produced from burning lignite. The near compliance of TUMBA and ALCO mixtures suggests the need for more sulfur or a more viscous asphalt to correct this deficiency.

### **Static Creep Test**

The static creep test was done according to Test Method Tex-450-A. Three specimens of each mixture design were tested at a constant stress level of 69 kPa and temperatures of 40°C. In this test, a static load was applied for a fixed duration along the centric longitudinal axis of a preconditioned specimen. The total axial deformation of the specimen was measured over a period of 1 hour. After 1 hour, the load was released and the specimen was allowed to recover for 10 minutes. A typical creep curve resulting from this type of loading is shown in Figure 4-26. From the recorded data, creep stiffness, permanent strain, and slope of the strain versus time plot were computed. Using these properties, bituminous mixtures can be evaluated for their resistance to rutting.

Specimens for this test were prepared using the Texas gyratory compactor (Texas Test Method: Tex 206F). The average height of the specimen was 50.8 mm with a diameter of 101.6 mm. The specimens were compacted at a stress level of 1035 kPa with a final compaction load of 11 kN. The following relationship was used to calculate strain and stiffness:

$$\begin{aligned}\text{Strain} &= \text{Deformation}/\text{Specimen thickness,} \\ \text{Creep Stiffness} &= 69 \text{ kPa}/\text{total strain, and} \\ \text{Slope} &= \text{Strain during secondary creep}/\text{time.}\end{aligned}$$

## TENSILE STRENGTH OF SMBA MIXTURES

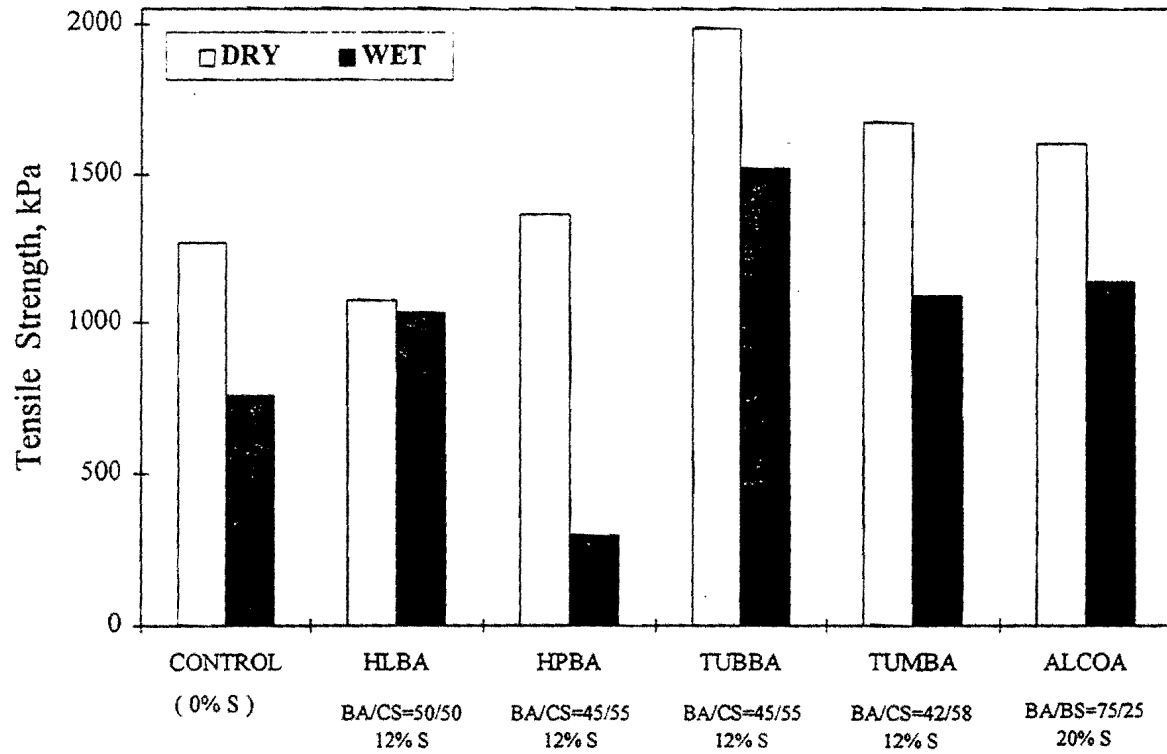


Figure 4-24. Tensile Strength of SMBA Mixtures.

## TENSILE STRENGTH RATIO OF SMBA MIXTURES

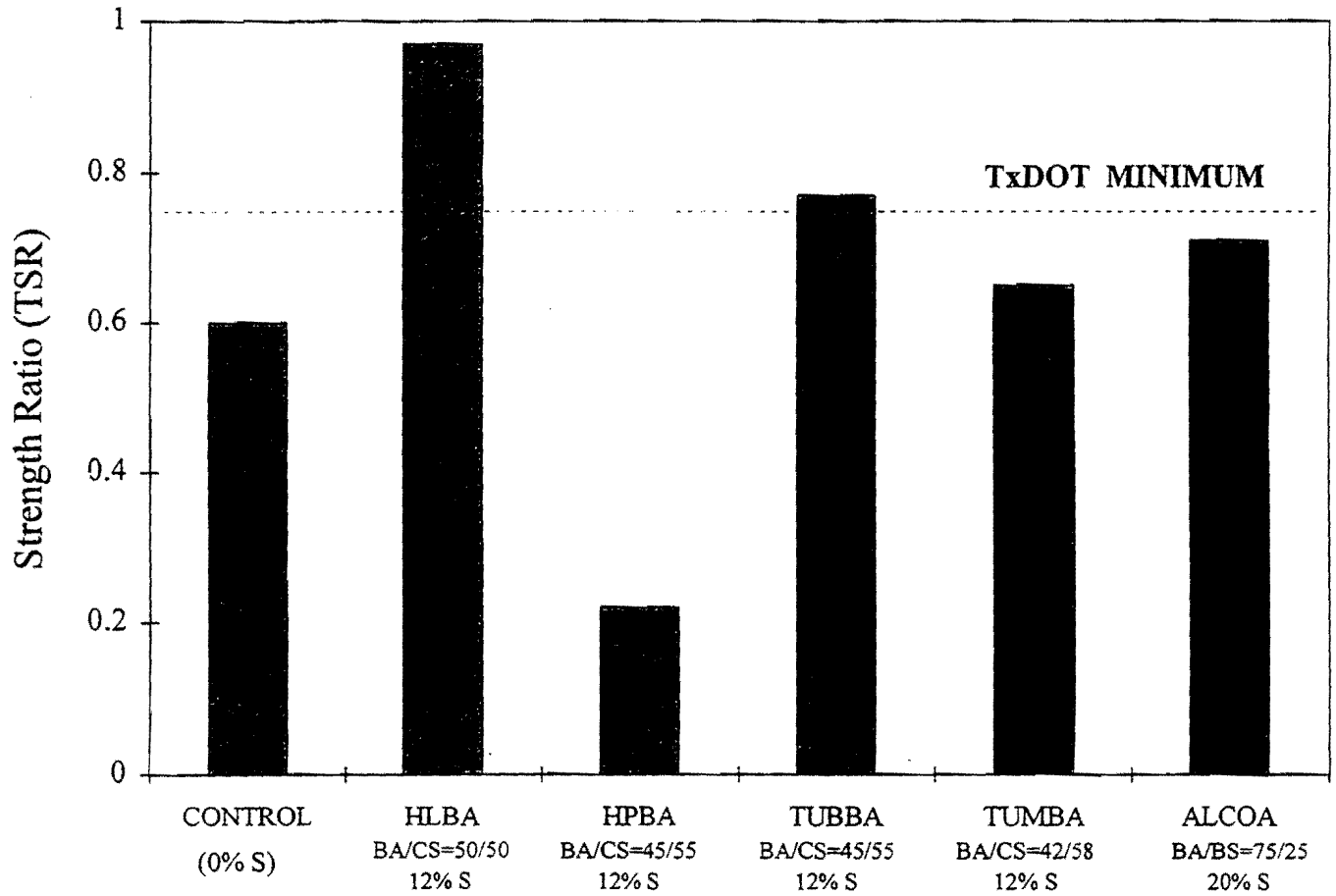


Figure 4-25. Tensile Strength Ratio of SMBA Mixtures.

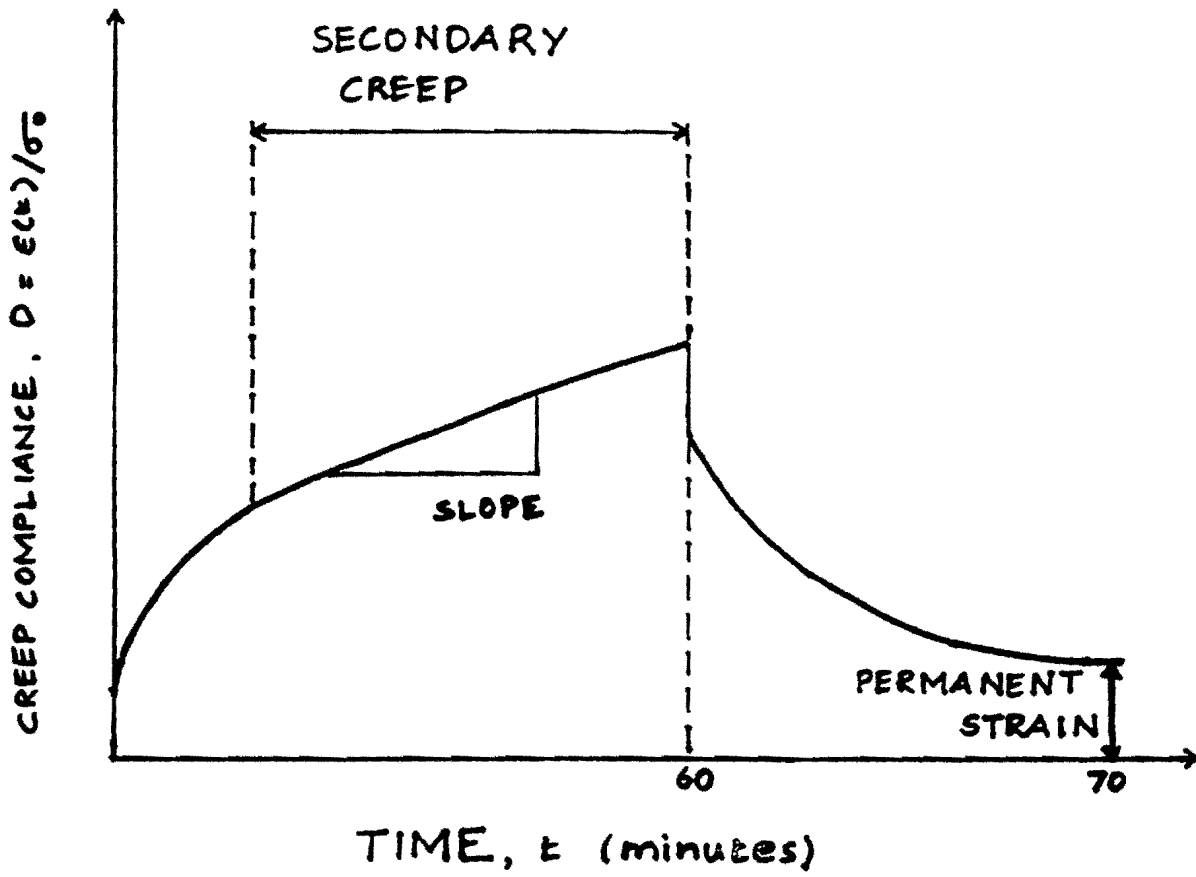


Figure 4-26. Typical Creep Curve.

Figures 4-27 through 4-29 give the results of the creep test conducted on the five SMBA and the control mixtures. Comparisons of the three creep parameters are also shown with allowable TxDOT criteria given below.

	TxDOT Allowable
Creep stiffness	41,400 kPa (min)
Permanent strain	0.05 percent
Creep slope	$8.9 \times 10^{-7}$ mm/sec

All SMBA mixtures and the control satisfied the above criteria, indicating their good resistance to rutting.

### PERMANENT STRAINS OF SMBA MIXTURES

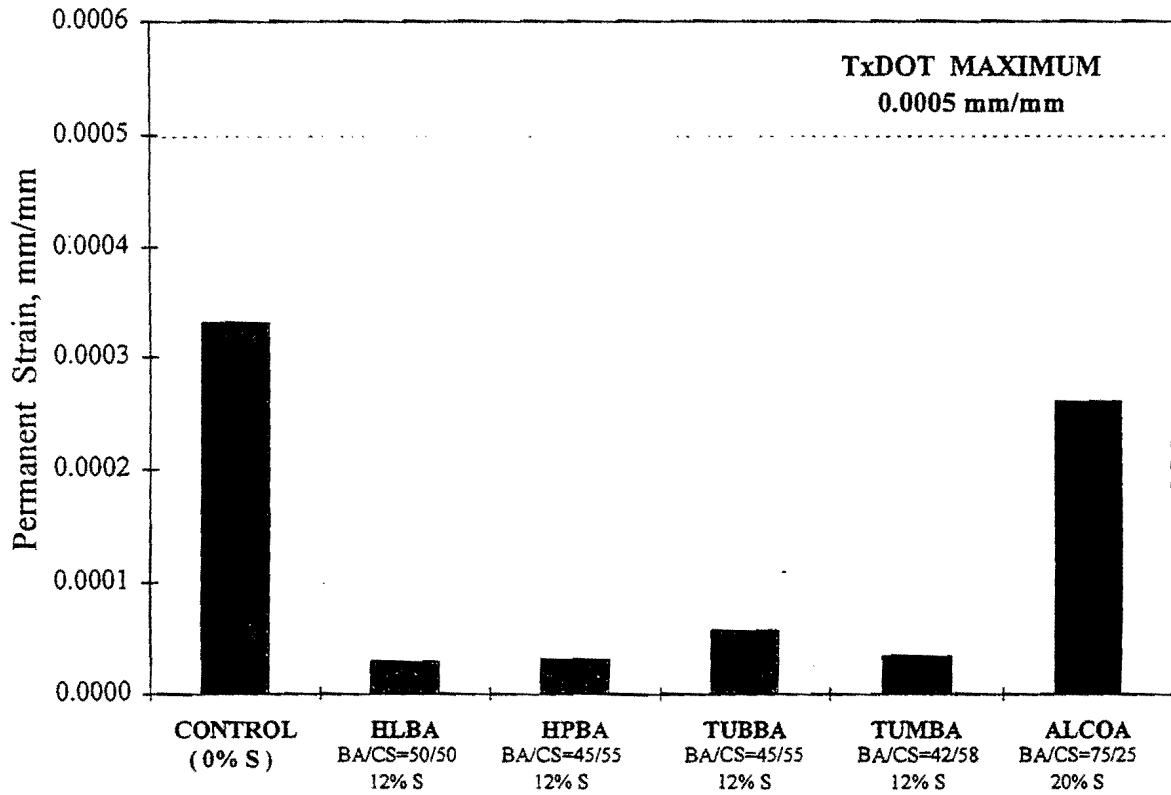


Figure 4-27. Permanent Strains of SMBA Mixtures.



### CREEP SLOPE OF SMBA MIXTURES

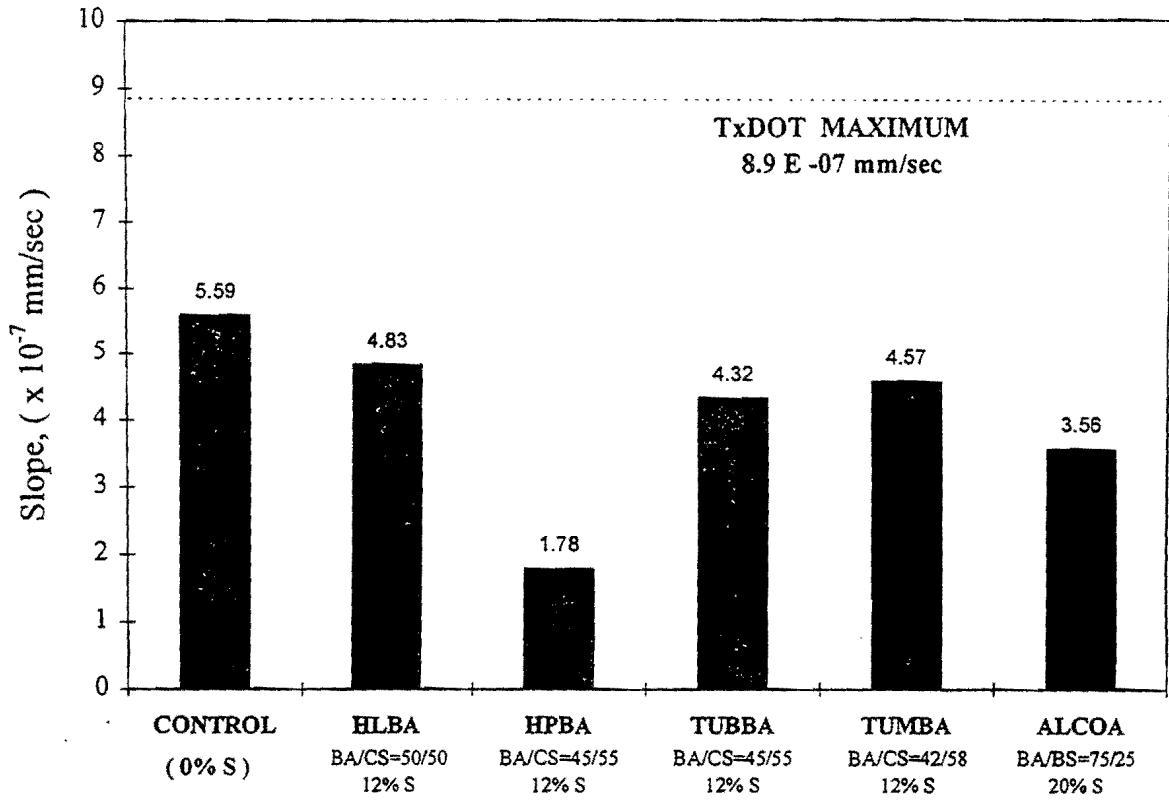


Figure 4-28. Creep Slope of SMBA Mixtures.

## CREEP STIFFNESS OF SMBA MIXTURES

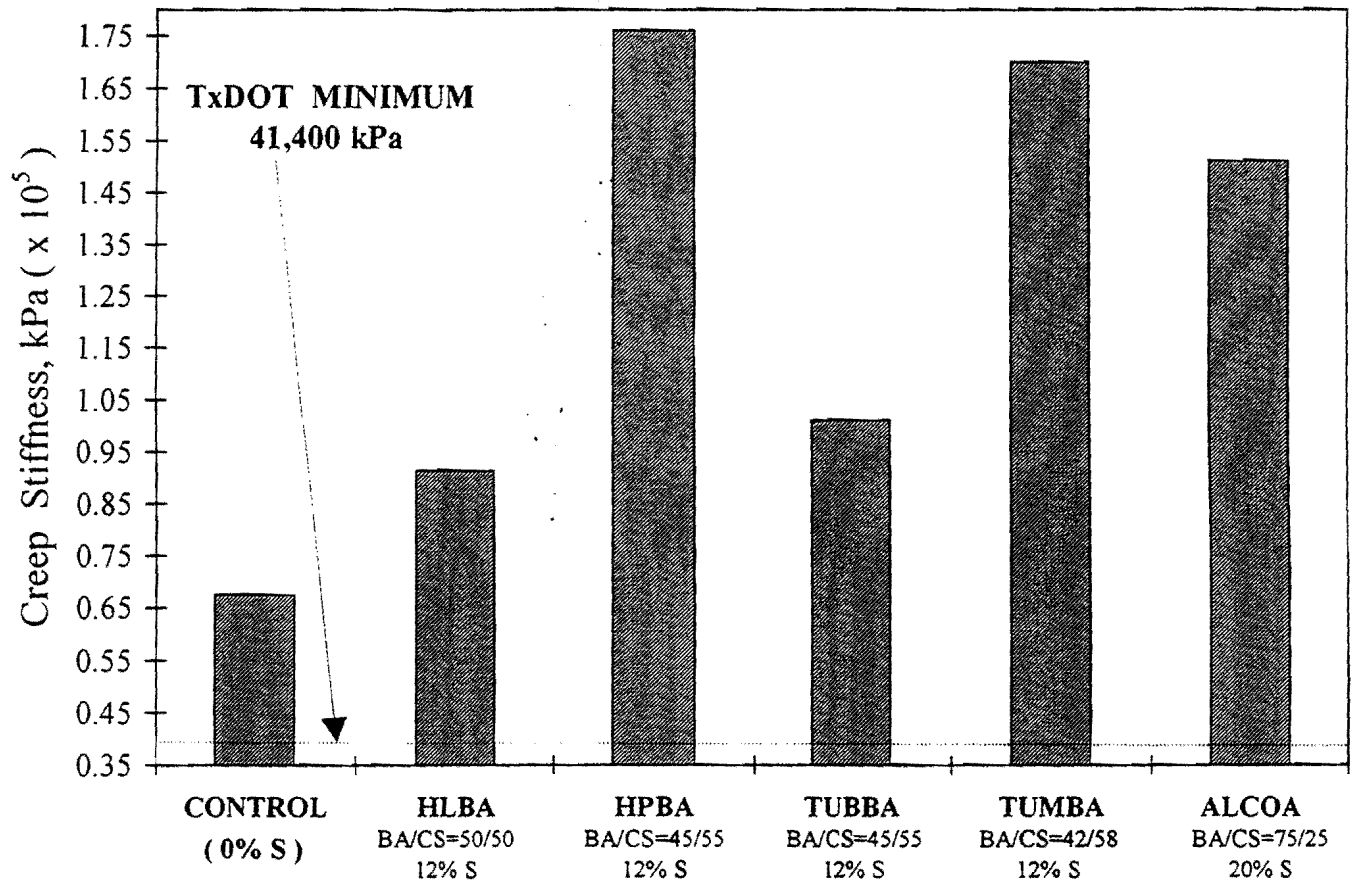


Figure 4-29. Creep Stiffness of SMBA Mixtures.

## **SUMMARY OF FINDING AND RECOMMENDATIONS**

The technology developed during the energy crises of the 1970s for preparing sand-asphalt-sulfur mixtures was exploited for upgrading bottom ash for use in base and surface course applications. The role of sulfuric in these mixtures was to enhance crush resistance of the ash, reduce asphalt demand, and air voids. Sulfur-modified bottom ash (SMBA) mixtures were designed to meet the requirement for TxDOT Type D mixtures while minimizing the need for conventional aggregates.

The need to meet Type D specifications on gradation and air voids required at least 50 percent crushed stone in the mixtures. One mixture with a 25/75 blend of boiler slag/bottom ash as the sole aggregate fraction performed very well in characterizations testing even though its gradation and air void content were not in accordance with Type D specifications. This suggests that, as was the case for SAS mixtures developed in the 1970s, SMBA mixtures should not be restricted to specifications developed for mixtures containing conventional aggregates. Sulfur-to-asphalt ratios of 2 to 3 should be a good starting point for designing these mixtures.

The results of this phase of the program indicate that SMBA mixtures should perform very well as both base and surface course materials. Except for ash produced from Wyoming coal, SMBA mixtures exhibit good durability. All SMBA mixtures reflect good fracture resistance and excellent resistance to rutting. The low unit weights of SMBA mixtures allow 20 to 35 percent more roadway to be built per ton of mix at approximately the same values demand of asphalt.

Two recommendations are suggested for additional work with SMBA mixtures. First is to explore the potential for minimizing field compactions. One of the limitations uncovered in field applications of SMBA mixtures was their vulnerability to compaction during construction and subsequently under heavy traffic loads. If, as was the case with the SAS concept, acceptable SMBA mixtures can be designed with minimal compaction and air voids much higher than those specified for Type D mixtures, this would offer a significant cost savings in the field.

Finally, an experimental project should be planned to demonstrate the constructability and the long-term integrity of SMBA mixtures under actual traffic conditions. Preliminary specifications contained in this report could be the basis for designing the mixture for this project.

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## **SECTION V SUMMARY OF FINDINGS AND RECOMMENDATIONS**

The objective of this study was to exploit the beneficiation and utilization of coal combustion by-products (CCBPs) as low-cost alternate aggregates and stabilizers in roadway and airfield construction. A literature review and laboratory investigation were performed to investigate the use of CCBPs in three highway construction applications:

1. Fly ash for subgrade stabilization,
2. Flue gas desulfurized (FGD) gypsum and gypsite for base and subbase construction materials, and
3. Bottom ash (with sulfur modification) in asphaltic mixtures.

The following is a discussion of the findings and recommendations regarding the above three highway construction applications.

### **FLY ASH FOR SUBGRADE STABILIZATION**

#### **Summary of Findings**

To evaluate the effectiveness of fly ash as a stabilizer, two very different subgrade soils that are commonly encountered in the Bryan District were chosen for the study: an A-3 soil, which is a poorly graded sand, and an A-7-6 soil, which can be described as a highly plastic clay. Two fly ashes (a Class 'C' and Class 'F') were chosen from power plants close to the district. Findings are listed below.

- The Class 'C' fly ash is a more effective stabilizer than the Class 'F' fly ash.
- Class 'F' fly ash/lime blends are more effective stabilizers than Class 'F' fly ash alone.
- Moisture requirements of lime and lime/fly ash blends are greater than that of fly ash mixes alone.

- For very highly plastic clays (PI = 60 or more) excavation and disposal may be a more cost-effective alternative to stabilization.

Conclusions related to stabilization of the poorly graded fine sand (A-3) are listed below.

- Class 'C' fly ash or lime/fly ash blends are more effective stabilizers than lime alone for the A-3 soil.
- The maximum dry density for the A-3 soil increases with the addition of fly ash.
- Optimum moisture content for the A-3 soil decreases with the addition of lime and/or fly ash.
- The unconfined compressive strength of the A-3 soil increases with increasing Class 'C' fly ash content.
- The unconfined compressive strength of the A-3 soil is greater with quick lime/Class 'F' than with either quick lime or Class 'F' fly ash only.
- For the A-3 soil, a delay between mixing the stabilizer and molding has greater influence on the unconfined compressive strength than on the maximum dry density.

Conclusions related to the stabilization of the highly plastic clay (A-7-6) soil are listed below.

- Lime was found to be the most effective stabilizer for the A-7-6 soil.
- Class 'F' fly ash blended with lime is an effective alternative stabilizer.
- The addition of lime and/or fly ash to the clay soil makes the soil more workable but reduces the maximum dry density achieved.
- The optimum moisture content increases with the addition of lime and/or fly ash to the clay soil.
- An increase in Class 'C' fly ash content reduces the PI and increases the strength of the clay soil. However, with increasing Class 'C' fly ash contents, there is a decrease in the reduction of swell.

## Preliminary Specifications

TxDOT currently has a specification governing the use of lime/fly ash blends as stabilizers for subgrade soils. No changes are recommended to Item 265, "Lime-Fly Ash (LFA) Treatment for Materials Used as Subgrade."

The following is a preliminary specification recommendation for the use of fly ash alone as a stabilizer for subgrade soils and is a modification of Item 265.

### FLY ASH TREATMENT FOR MATERIALS USED AS SUBGRADE

- 1.0 **Description.** This Item shall govern for treating new or existing subgrade, existing pavement structure or combination thereof to be used as subgrade by pulverizing, adding fly ash, mixing and compacting the mixed material as specified in this item.
- 2.0 **Materials.**
  - 2.1 **Fly Ash.** Fly ash shall be ASTM Class 'C' fly ash and shall meet the requirements of "Departmental Materials Specification: D-9-8900, Fly Ash."
  - 2.2 **Water.** Water shall meet the materials requirements of Item 204, "Sprinkling."
- 3.0 **Equipment.**
  - 3.1 **General.** The machinery, tools and equipment necessary for proper prosecution of the work on this Item shall be on the project and approved by the Engineer prior to beginning this Item.
  - 3.2 **Material Storage.** Fly ash shall be suitably stored in closed, weatherproof containers until immediately before use. Storage bins, when used, shall be completely enclosed.
  - 3.3 **Material Mass Verification.** When fly ash is furnished in trucks, the mass of fly ash shall be determined on certified scales or the Contractor shall provide a set of standard platform truck scales at a location approved by the Engineer. Scales shall conform to the requirements of Item 520, "Weighing and Measuring Equipment."

When fly ash is furnished in bags, each bag shall bear the manufacturer's certified mass. Bags varying more than five (5) percent from that mass may be rejected and the average mass of bags in any shipment, as shown by weighing 10 bags taken at random, shall not be less than the manufacturer's certified mass.

#### 4.0 Construction Methods.

- 4.1 **General.** The completed course shall be uniformly treated, free from loose or segregated areas, of uniform density and moisture content, well bound for its full depth and shall have a smooth surface.
- 4.2 **Preparation of Subgrade or Existing Base.** Preparation of subgrade or existing base shall be in accordance with Item 265.4, "Preparation of Subgrade or Existing Base."
- 4.3 **Pulverization.** The existing pavement or base material shall be pulverized or scarified so that 100 percent shall pass the 63 millimeter sieve.
- 4.4 **Application of Fly Ash.** The percentage by mass or kilograms per square meter of fly ash to be added will be shown on the plans and may be varied by the Engineer if conditions warrant.

Fly ash shall be spread only on that area where the mixing operation can be completed during the same working day.

Unless otherwise approved in writing by the Engineer, the fly-ash mixing operation shall not be started when the air temperature is below 5°C and falling, but may be started when the air temperature is above 2°C and rising. The temperature will be taken in the shade and away from artificial heat. Fly ash shall not be placed when the weather conditions, in the opinion of the Engineer, are unsuitable.

Unless otherwise approved by the Engineer, fly ash shall be distributed in the dry form only by a distributor approved by the Engineer. Fly ash shall not be applied when wind conditions, in the opinion of the Engineer, are such that blowing fly ash becomes objectionable to adjacent property owners or dangerous to traffic. The mixture shall be sprinkled as approved by the Engineer. Fly ash shall be uniformly spread only on that area where the mixing and compacting operations can be completed during the same working day. A motor grader shall not be used to spread fly ash. Initial mixing after the addition of fly ash shall be accomplished dry or with a minimum of water to prevent fly ash balls.

- 4.5 **Mixing of Fly Ash.** The mixing shall be continued until, in the opinion of the Engineer, a homogeneous mixture is obtained.
- 4.6 **Compaction Methods.** Prior to compaction, the material shall be aerated or sprinkled as necessary to provide the optimum moisture. Compaction shall begin immediately after mixing of the last stabilizing agent. All compaction operations shall be completed within six (6) hours.

Compaction shall continue until the entire depth of mixture is uniformly compacted by "Ordinary Compaction" or "Density Control" as shown on the plans. Throughout this entire operation the shape of the course shall be maintained by balding, and the surface upon completion shall be smooth and in conformity with the typical sections, lines and grades as shown on the plans or as established by the Engineer.

When shown on the plans or approved by the Engineer, multiple lifts will be permitted.

When "Ordinary Compaction" is shown on the plans, the following provisions shall apply:

The material shall be sprinkled and rolled as directed by the Engineer. All irregularities, depressions or weak spots which develop shall be corrected immediately by scarifying the areas affected, adding or removing material as required and reshaping and recompacting by sprinkling and rolling. The surface of the course shall be maintained in a smooth condition, free from undulations and ruts, until the next course is placed.

Should the material lose the required stability, compaction or finish before the next course is placed, or the project is accepted, it shall be reworked in accordance with Subarticle 4.7. However, compaction shall be in accordance with "Ordinary Compaction."

When "Density Control" is shown on the plans, the following provisions shall apply:

Unless otherwise shown on the plans, each course shall be sprinkled as required and compacted to the extent necessary to provide not less than 95 percent of the optimum density as determined by Test Method Tex-121-E, Part II. Roadway density testing will be as outlined in Test Method Tex-115-E.

When the material fails to meet the density requirements, or should the material lose the required stability, density or finish before the next course is placed or the project is accepted, it shall be reworked in accordance with Subarticle 4.7.

**4.7 Reworking a Section.** When a section is reworked with 72 hours after placement, the Contractor shall rework the section to provide the required compaction. When a section is reworked more than 72 hours after placement, the Contractor shall add 25 percent of the specified rate of fly ash. Reworking shall include loosening, road mixing as approved by the Engineer, compacting and finishing. When a section is reworked, a new optimum density will be determined from the reworked material in accordance with Test Method Tex-121-E, Part II.

**4.8 Finishing and Curing.** After the final layer or course of the fly ash-treated material has been compacted, it shall be brought to the required lines and grades in accordance with the typical sections, within two (2) hours.

The completed section shall then be finished by rolling with a pneumatic tire or other suitable roller approved by the Engineer. The completed section shall be moist-cured or prevented from drying by addition of an asphalt material at the rate of 0.2 to 0.9 liter per square meter as determined by the Engineer. This material shall be the type shown on the plans. Curing shall continue for seven (7) days before further courses are added or traffic is permitted, unless otherwise approved by the Engineer.

However, the fly ash-treated material may be covered by other courses, the day following finishing when approved by the Engineer. When the plans provide for the treated material to be covered by other courses of material, the next course shall be applied within 14 calendar days after final compaction is completed, unless otherwise approved by the Engineer.

**5.0 Tolerances.**

- 5.1 Density Tolerances.** The Engineer may accept the work providing not more than one (1) out of the most recent five (5) density tests performed is below the specified density, provided the failing test is not more than 50 kilograms per cubic meter below the specified density.
- 5.2 Grade Tolerances.** Finished grade tolerances shall be in accordance with Subarticle 132.4 (1).

**6.0 Measurement.**

- 6.1 Fly Ash.** Fly ash will be measured by the megagram, dry mass as delivered on the road.
- 6.2 Fly Ash Treatment.** The fly ash treatment will be measured by the square meter of the dept specified to the line and grades shown on the typical sections.

**7.0 Payment.** The work performed and materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for as follows:

- 7.1 Fly Ash.** Fly ash will be paid for at the unit price bid for "Fly Ash" which will be full compensation for furnishing all fly ash.

Fly ash for reworking a section in accordance with Subarticle 4.7 will not be paid for directly but will be subsidiary to this Item.

- 7.2 Fly Ash Treatment.** "Fly Ash Treated Subgrade" of the compaction method and depth specified will be paid for at the unit price bid per square meter. This price shall be full compensation for shaping existing material, loosening, mixing, pulverizing, spreading, drying, applying fly ash, compacting, curing including curing materials, shaping and maintaining, processing, hauling, reworking if required, preparing secondary subgrade and for all mixing water, tools, equipment, labor and incidentals necessary to complete the work.

When proof rolling is shown on the plans and directed by the Engineer, it will be paid for in accordance with Item 216, "Rolling (Proof)."

When "Ordinary Compaction" is shown on the plans, all sprinkling and rolling, except proof rolling, will not be paid for directly, but will be considered subsidiary to this Item, unless otherwise shown on the plans.

When "Density Control" is shown on the plans, all sprinkling and rolling, except proof rolling, will not be paid for directly but will be considered subsidiary to this Item.

When subgrade is constructed under this project, correction of soft spots in the subgrade will be at the Contractor's expense. When subgrade is not constructed under this project, correction of soft spots in the subgrade will be in accordance with Article 9.3.

### **Recommendations for Field Testing**

Based on the results of the laboratory study, the following materials are recommended for use as stabilizers for subgrade soils as field trials:

#### ***A-3 Soil:***

<i>Test Section 1.</i>	<i>15% Class 'C' fly ash.</i>
<i>Test Section 2.</i>	<i>20% Class 'C' fly ash.</i>
<i>Test Section 3.</i>	<i>3% Quicklime and 8% Class 'F' fly ash.</i>

#### ***A-7-6 Soil:***

<i>Test Section 1.</i>	<i>Control - 4% Quicklime.</i>
<i>Test Section 2.</i>	<i>2% Quicklime and 5% Class 'F' fly ash.</i>
<i>Test Section 3.</i>	<i>10% Class 'C' fly ash.</i>
<i>Test Section 4.</i>	<i>15% Class 'C' fly ash.</i>

### **FLUE GAS DESULFURIZED (FGD) GYPSUM AND GYPSITE FOR BASE AND SUBBASE CONSTRUCTION MATERIALS**

#### **Summary of Findings**

A comprehensive literature review revealed that FGD by-product calcium sulfate (gypsum) and sulfites (Gypsite) can be stabilized with either cement or fly ash and utilized as road bases or sub-bases. This research, along with work previously done at the Texas Transportation Institute, has shown that these procedures developed for FGD material also apply to other forms of gypsum as well.

A set of test procedures and materials selection guidelines, which do not necessarily comply with TxDOT standard methods, have been generated and validated in the field using

experimental test sections. Gypsum and Gypsites require high sulfate-resistant cements for stabilization that exceed the protection provided by Type II cements. This protection is achieved with tri-calcium aluminate levels of less than 4 percent.

Post-construction evaluations of the structural integrity and environmental impact of a series of TTI-constructed test sections would classify stabilized FGD road bases as excellent, with negligible effect on groundwater, surface runoff, and leachates.

Mix designs of FGD scrubber bases from two power generating stations indicate that, depending on the amount and type of fly ash relative to FGD material in the stockpile, sufficient stabilization could be accomplished with as low as 2 percent or as high as 7 percent cement. Due to the chemically active nature of some fly ashes, the age of the stockpile should be considered in the mix design. The best window of opportunity exists between a stockpile age of 1 to 3 days. Beyond this time, additional stabilizer may need to be provided.

In summary, FGD by-products would appear to offer a cost-effective alternative to natural aggregates currently in use. That sources for these materials coincide with regions in Texas where conventional aggregate supplies are non-existent or depleting represents an additional advantage.

### **Preliminary Specifications**

The following is a preliminary specification recommendation for the use of FGD material and fly ash as road base. This specification was used to construct the road base on FM 1512 in the Bryan District near Jewett, Texas.

#### **STABILIZED SCRUBBER BASE (ROAD MIXED)**

- 1.0 Description.** This Item shall govern the placement and treating of new scrubber base material by adding cement, mixing and compacting the treated material to the required density as specified herein and in conformity with the typical sections, lines, grades and depths as shown on the plans or as established by the Engineer.
- 2.0 Materials.**
  - 2.1 Cement.** Cement shall conform with Item 524, "Hydraulic Cement," Type III, except the content of tricalcium aluminate shall be 5% or less.



- 2.2 **Scrubber Base.** Scrubber base material shall consist of a mixture of flue gas desulfurization (FGD) material and fly ash, with a minimum fly ash to FGD material ratio of 1/1 by weight.
- 2.3 **Fly Ash.** On-site fly ash (if needed for moisture control) shall conform to Type A fly ash and shall meet the requirements of Departmental Materials Specification D-9-890-0, except that the fineness requirement shall be increased to allow up to 45% passing a 325 sieve.
- 2.4 **Water.** Water shall meet the material requirements of TxDOT Item 204, "Sprinkling."

### 3.0 **Equipment.**

- 3.1 **General.** The machinery, tools and equipment necessary for proper prosecution of the work on stabilized scrubber base shall be on the project and approved by the Engineer prior to beginning the work.
- 3.2 **Material Storage.** Cement and fly ash (if needed) shall be suitably stored in closed, weatherproof containers until immediately before use. Storage bins, when used, shall be completely enclosed.
- 3.3 **Material Weight Verification.** When cement and fly ash (if needed) is furnished in trucks, their weight shall be determined on certified scales or the Contractor shall provide a set of standard platform truck scales at a location approved by the Engineer. Scales shall conform to the requirements of Item 520, "Weighing and Measuring Equipment."

- 4.0 **Mix Proportions.** Cement content will be selected by the Engineer based on unconfined compressive strength tests which achieve a minimum of 4485 kPa at 14 days. Laboratory specimens shall be prepared in accordance with Test Method Tex-120-E, except unconfined compressive strength shall be determined after 14 days.

### 5.0 **Construction Methods.**

- 5.1 **General.** The completed course shall be uniformly treated, free from loose or segregated areas, of uniform density and moisture content, well bound for its full depth and shall have a smooth surface.
- 5.2 **Preparation of Subgrade.** Subgrade preparation shall be in accordance with Item 260, "Lime Treatment for Materials Used as Subgrade (Road Mixed)."
- 5.3 **Application of Scrubber Base and Cement.** Scrubber base shall be spread uniformly over prepared subgrade.

Cement shall be spread only in that area where the mixing, compacting, and finishing operations can be completed within 3 hours.

Unless otherwise approved by the Engineer, the cement treatment operation shall not be started when the air temperature is below 7°C and falling, but may be placed when the air temperature is above 4°C and rising. The temperature will be taken in the shade and away from artificial heat. Cement shall not be placed when weather conditions in the opinion of the Engineer are unsuitable.

Cement (and fly ash, if required) shall be spread by an approved spreader or by bag distribution. It shall be distributed at a uniform rate and in such a manner as to reduce to a minimum the scattering of cement by wind. Cement (and fly ash) shall not be applied when wind conditions, in the opinion of the Engineer, are such that blowing cement or fly ash become objectionable to adjacent property owners or dangerous to traffic.

**5.4 Mixing.** Only single or multiple soil stabilizer mixers shall be used.

If additional fly ash is required to reduce moisture content of scrubber base at the job site, it shall be mixed prior to the addition of cement.

After any required mixing of scrubber base and fly ash, the cement shall be mixed with the materials prior to the addition of water. Immediately after mixing, water shall be uniformly applied, if required to increase moisture content to acceptable levels. Moisture content tolerances are specified in Article 6.2. The material shall be mixed again to evenly distribute the moisture throughout the mixture. After mixing, the mixture shall be in a loose, evenly spread state ready for compaction. The mixture shall be mixed and compacted in one (1) lift not to exceed 300 mm.

**5.5 Compaction.** Compaction shall continue until the entire thickness of the mixture is uniformly compacted as described below. Compaction shall be completed within three hours of the addition of cement to the material.

The treated material shall be sprinkled and rolled as directed by the Engineer. All irregularities, depressions or weak spots which develop shall be corrected immediately by scarifying the areas affected, adding or removing treated material as required, reshaping, and recompacting.

The stabilized scrubber base shall be compacted to 97% of the density as determined by Test Method Tex-120-E. Roadway Density will be determined by Test Method Tex-115-E, Part II.

Should the material lose the required stability, compaction or finish before the next course is placed or the project is accepted, it shall be removed and replaced, unless otherwise approved by the Engineer.

**5.6 Finishing.** When initial compaction of the mixture is nearing completion, the surface shall be shaped to the required lines, grades and cross sections, and compaction continued until uniform and adequate density is obtained. The moisture content of the surface of the base course mixture shall be maintained within zero (0) to minus three (3) percentage points of optimum as determined by Test Method Tex-103-E during all finishing operations, unless otherwise approved by the Engineer. The surface shall then be rolled with a pneumatic tire roller, adding small increments of moisture as needed during rolling. Final grading must occur before the cementing reaction takes place.

- 5.7 Curing and Preparation for Prime Coat.** The completed section shall be moist cured for a minimum of three (3) days, or prevented from drying by addition of an asphalt material at the rate of 0.2 to 0.9 liters per square meter or as determined by the Engineer. The asphalt used shall be of the type and grade shown on the plans and approved by the Engineer.

Loose material remaining on the base surface after final grading operation shall be removed from the surface prior to placing of the prime coat. An additional pass of the compactor may be required to seal the final graded surface. Traffic on the scrubber base must be limited to prime coat application equipment after final grading. Prior to prime coat application, the finished surface shall be moistened to maintain appropriate surface moisture content.

**6.0 Tolerances.** Tolerances shall conform to the following:

- 6.1 Density Tolerances.** The Engineer may accept the work providing not more than one (1) out of the most recent five (5) density tests performed is below the specified density, provided the failing test is no more than 50 kilograms per cubic meter below the specified density.
- 6.2 Moisture Tolerances.** The percentage of moisture in the mixture at the beginning of compaction shall be within zero (0) to minus three (3) percentage points of optimum as determined by Test Method Tex-120-E, unless otherwise approved by the Engineer. The percent moisture will be determined in accordance with Test Method Tex-103-E. If the percentage of moisture is outside the allowable tolerance, the Contractor shall adjust operations to meet this requirement.
- 6.3 Grade Tolerances.** In areas on which pavement is to be placed, any deviation in excess of 6 mm in cross section and 6 mm in 4.9 m measured longitudinally shall be corrected by loosening, adding or removing material, reshaping and compacting by sprinkling and rolling.

**7.0 Measurement.** This item will be measured as follows:

- 7.1** Stabilized scrubber base will be measured by the square meter of surface area to lines and grades shown on the typical sections.
- 7.2** Cement, scrubber base and fly ash will be measured by the megagram, dry weight. The dry mass will be determined by deducting the mass of the moisture in the material at the time of measurement from the gross mass of the material. The moisture in the material will be determined in accordance with Test Method Tex-103-E at least once each day and more often if conditions warrant.

**8.0 Payment.** The work performed and materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for as follows:

- 8.1** Cement will be paid for at the unit price bid for "Cement," which price will be full compensation for furnishing all cement.

- 8.2 Scrubber base will be paid for at the unit price bid for "Scrubber Base," which price will be full compensation for furnishing all scrubber base.
- 8.3 Fly ash will be paid for at the unit price bid for "Fly Ash," which price will be full compensation for furnishing all fly ash.
- 8.4 Stabilized scrubber base of the type, compaction and depth specified will be paid for at the unit price bid per square meter. This price shall be full compensation for applying fly ash (if required), pulverizing, applying cement, spreading, road mixing, compacting, grading, shaping, finishing, curing, including curing materials, replacing if required, and for all mixing water, labor tools and incidentals necessary to complete the work except as otherwise provided for in this specification.
- 8.5 When proof rolling is shown on the plans, and when directed by the Engineer, it will be paid for in accordance with Item 216 "Rolling (Proof)."
- 8.6 When subgrade is constructed under this project, correction of soft spots will be at the Contractors expense.

### **Recommendations for Field Testing**

Based on the results of this study, the following test sections are recommended for evaluation as potential road base materials:

- Test Section 1. Cement Stabilized or HVFA Cement Stabilized FGD.*
- Test Section 2. Fly Ash (Class 'C') Stabilized FGD.*

### **SULFUR-MODIFIED BOTTOM ASH IN ASPHALTIC MIXTURES**

#### **Summary of Findings**

The technology developed during the energy crises of the 1970s for preparing sand-asphalt-sulfur mixtures was exploited for upgrading bottom ash for use in base and surface course applications. The role of sulfur in these mixtures was to enhance crush resistance of the ash, reduce asphalt demand and air voids. Sulfur-modified bottom ash (SMBA) mixtures were designed to meet the requirement for TxDOT Type D mixtures while minimizing the need for conventional aggregates.

The need to meet Type D specifications on gradation and air voids required at least 50 percent crushed stone in the mixtures. One mixture with a 25/75 blend of boiler slag/bottom ash as the sole aggregate fraction performed very well in characterization testing even though its gradation and air void content were not in accordance with TxDOT specifications. This suggests that, as was the case for SAS mixtures developed in the 1970s, SMBA mixtures should not be restricted to specifications developed for mixtures containing conventional aggregates. Sulfur-to-asphalt ratios of 2 to 3 should be a good starting point for designing these mixtures.

The results of this phase of the program indicate that SMBA mixtures should perform very well as both base and surface course materials. Except for ash produced from Wyoming coal, SMBA mixtures exhibit good durability. All SMBA mixtures reflect good fracture resistance and excellent resistance to rutting. The low unit weights of SMBA mixtures allow 20 to 35 percent more roadway to be built per ton of mix.

Two recommendations are suggested for additional work with SMBA mixtures. First, explore the potential for minimizing field compaction. One of the limitations uncovered in field applications of SMBA mixtures was their vulnerability to compaction during construction and subsequently under heavy traffic loads. If, as was the case with the SAS concept, acceptable SMBA mixtures can be designed with minimal compaction and air voids much higher than those specified for Type D mixtures, this would offer a significant cost savings in the field.

### **Specifications**

The concept of SMBA asphaltic mixtures was developed in this study and was evaluated in the laboratory. Since the concept is very new and very different from the current way of designing and constructing asphaltic mixtures, insufficient information was available to develop specifications. It is recommended that specifications be formulated during the field experimentation phase described below.

### **Recommendations for Field Testing**

Researchers believe that SMBA asphaltic mixtures have potential for use on general aviation runways and that this would be an excellent field test; however, it is recommended that

an even smaller scale field experiment be developed first. It is recommended that a field experiment (100 meters in length) be placed at a location such as Texas A&M's Riverside Campus. The aggregate fraction of this mixture could be either 100% bottom ash or 50% bottom ash as determined by the Project Director. Any of the mixture designs presented in Section IV would be a good starting point for development of a field trial.

## **APPENDIX A**





# JTM JTM INDUSTRIES, INC.

POST OFFICE BOX 38  
 THOMPSONS, TEXAS 77481-0038  
 (713) 343-0077  
 FAX (713) 545-9205

Report of Fly Ash W.A. Parish Unit #8

DATE: September 9, 1996 LABORATORY NUMBER: WAP-100-8

COMPOSITE MONTH  
 July 1996

ASTM C-618-94a  
 SPECIFICATIONS

CLASS C CLASS F

CHEMICAL ANALYSIS

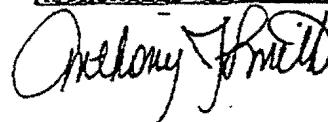
Silicon Dioxide (SiO <sub>2</sub> )	33.39		
Alumium Oxide (Al <sub>2</sub> O <sub>3</sub> )	19.57		
Iron Oxide (Fe <sub>2</sub> O <sub>3</sub> )	6.27		
Sum of SiO <sub>2</sub> , Al <sub>2</sub> O <sub>3</sub> , & Fe <sub>2</sub> O <sub>3</sub>	59.23	50 Min.	70 Min.
Magnesium Oxide (MgO)	5.27		
Sulfur Trioxide (SO <sub>3</sub> )	2.24	5.0 Max.	5.0 Max.
Moisture Content	0.06	3.0 Max.	3.0 Max.
Loss On Ignition	0.23	6.0 Max.	6.0 Max.
Available Alkalies as Na <sub>2</sub> O	1.29	A1.5 Max.	A1.5 Max.
Calcium Oxide (CaO)	27.45		

PHYSICAL ANALYSIS

Fineness: Amount retained on 325 sieve †	10.40	34% Max.	34% Max.
Water Requirement, % Control	94%	105% Max.	105% Max.
Specific Gravity	2.63		
Autoclave Expansion, %	-0.01	0.8% Max.	0.8% Max.
Strength Activity Index With Portland Cement, 7 Day	93%	75% Min.	75% Min.

<sup>A</sup>Applicable only when required by purchaser.

AUTHORIZED SIGNATURE:



Report of Fly Ash Limestone Generating Station Jewett, Texas

DATE: August 14, 1995 LABORATORY NUMBER: LEGS-100-FA

COMPOSITE MONTH  
 July 1995

ASTM C-618-85  
 SPECIFICATIONS  
 CLASS C CLASS F

CHEMICAL ANALYSIS

Silicon Dioxide (SiO <sub>2</sub> )	57.03		
Alumium Oxide (Al <sub>2</sub> O <sub>3</sub> )	19.08		
Iron Oxide (Fe <sub>2</sub> O <sub>3</sub> )	10.03		
Sum of SiO <sub>2</sub> , Al <sub>2</sub> O <sub>3</sub> , & Fe <sub>2</sub> O <sub>3</sub>	86.14	50 Min.	70 Min.
Magnesium Oxide (MgO)	1.89		
Sulfur Trioxide (SO <sub>3</sub> )	0.67	5.0 Max.	5.0 Max.
Moisture Content	0.01	3.0 Max.	3.0 Max.
Loss On Ignition	0.01	6.0 Max.	6.0 Max.
Available Alkalies as Na <sub>2</sub> O	0.19	A1.5 Max.	A1.5 Max.
Calcium Oxide (CaO)	7.66		

PHYSICAL ANALYSIS

Fineness: Amount retained on 325 sieve %	34.17	34% Max.	34% Max.
Water Requirement, % Control	95%	105% Max.	105% Max.
Specific Gravity	2.38		
Autoclave Expansion, %	-0.03	0.8% Max.	0.8% Max.
Strength Activity Index With Portland Cement, 28 Day	80%	75% Min.	75% Min.

<sup>A</sup>Applicable only when required by purchaser.

AUTHORIZED SIGNATURE:



**REPORT OF  
ASH ANALYSIS**

409-880-8374

Engineers, Geologists, Chemists, Water Planners, Hygienists and Environmental Scientists



12821 W. Golden Lane  
P.O. Box 690287, San Antonio, TX 78269-0287  
(512) 696-0060 FAX (512) 696-6426

MONEX Resources, Inc.  
Attn: Mr. Gary Shelton

Project No.: ASD93-030-05

Date: 3-22-93

Assignment No.(s): 4-11950

Project Name: Rockdale Plant Fly Ash Quality Assurance

Sample Received: 3-4-93

Month of February, 19 93

MONTHLY COMPOSITE	Results	* Spec Class F/C
Silicon Dioxide (SiO <sub>2</sub> ), %	56.2	
Aluminum Oxide (Al <sub>2</sub> O <sub>3</sub> ), %	24.4	
Iron Oxide (Fe <sub>2</sub> O <sub>3</sub> ), %	3.7	
Sum of SiO <sub>2</sub> , Al <sub>2</sub> O <sub>3</sub> , Fe <sub>2</sub> O <sub>3</sub> , %	84.3	70.0/90.0 min.
Calcium Oxide (CaO), %	9.5	
Magnesium Oxide (MgO), %	2.0	
Sulfur Trioxide (SO <sub>3</sub> ), %	0.5	5.0 max.
Moisture Content, %	0.2	3.0 max.
Loss on Ignition, %	0.9	6.0 max.
Amount Retained on No. 325 Sieve, %	14.3	34 max.
Specific Gravity	2.27	

\* ASTM C 618, Class F

Copies to: Above (1)

dgp 3-24

RABA-KISTNER CONSULTANTS, INC.

By: [Signature]  
Date B. Martin, P.E.

San Antonio / El Paso / Austin

# Report of Chemical Analysis

Consulting Geotechnical, Materials and Environmental Engineers  
Geologists, Scientists and Chemists



**Raba-Kistner  
Consultants, Inc.**

To: Monier Resources, Inc.  
2030 Powers Fairy Road  
Suite 206  
Atlanta, GA 30339

P.O. Box 690287, San Antonio, TX 78269-0287  
12821 W. Golden Lane, San Antonio, TX 78249  
(512) 699-9090

Attn: Mr. Jim Riggs

Project No: 686-027  
Date Received: 10/01/86  
Date Reported: 10/07/86  
Submitted By: MRI

Sample Description/Code: Bottom Ash

## SUMMARY OF ANALYSIS

Unit 4 Bottom  
ASH

Wt, %



<u>Composition</u>	<u>Yates Plant</u>	<u>McDonough Plant</u>	<u>Rockdale Plant</u>
Silicon Dioxide (SiO <sub>2</sub> )	46.0	42.8	50.4
Aluminum Oxide (Al <sub>2</sub> O <sub>3</sub> )	21.0	21.2	24.4
Iron Oxide (Fe <sub>2</sub> O <sub>3</sub> )	18.5	16.8	5.36
Calcium Oxide (CaO)	3.52	1.52	12.0
Magnesium Oxide (MgO)	0.86	0.78	2.14
Sodium Oxide (Na <sub>2</sub> O)	0.45	0.26	0.21
Potassium Oxide (K <sub>2</sub> O)	1.81	1.91	0.80
Sulfur Trioxide (SO <sub>3</sub> )	0.05	0.40	0.18

C.C. Mr. Buddy Briscoe, MRI, San Antonio

Raba-Kistner Consultants, Inc.

by *Francis Y. Huang*  
Francis Y. Huang, Ph.D., CPC

Austin / El Paso / San Antonio

**J<sup>T</sup>M** JTM INDUSTRIES, INC.

POST OFFICE BOX 38  
THOMPSONS, TEXAS 77481-0038  
(713) 343-0077  
FAX (713) 545-9205

Report of Raw Bottom Ash W.A. Parish Generating Station

DATE: January 17, 1996 LABORATORY NUMBER: WAP-100-BA

COMPOSITE MONTH  
January 1996

CHEMICAL ANALYSIS

Silicon Dioxide (SiO <sub>2</sub> )	42.12
Alumium Oxide (Al <sub>2</sub> O <sub>3</sub> )	18.20
Iron Oxide (Fe <sub>2</sub> O <sub>3</sub> )	8.68
Sum of SiO <sub>2</sub> , Al <sub>2</sub> O <sub>3</sub> , & Fe <sub>2</sub> O <sub>3</sub>	69.00
Magnesium Oxide (MgO)	3.83
Sulfur Trioxide (SO <sub>3</sub> )	0.70
Moisture Content	9.48
Loss On Ignition	2.82
Calcium Oxide (CaO)	21.53

AUTHORIZED SIGNATURE:



**JTM** JTM INDUSTRIES, INC.

POST OFFICE BOX 38  
THOMPSONS, TEXAS 77481-0038  
(713) 343-0077  
FAX (713) 545-9205

Report of Raw Bottom Ash Limestone Generating Station Jewett, Texas

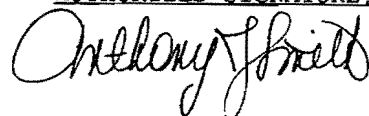
DATE: February 21, 1996      LABORATORY NUMBER: LEGS-100-BA

COMPOSITE MONTH  
January 1996

CHEMICAL ANALYSIS

Silicon Dioxide (SiO <sub>2</sub> )	56.01
Alumium Oxide (Al <sub>2</sub> O <sub>3</sub> )	14.27
Iron Oxide (Fe <sub>2</sub> O <sub>3</sub> )	18.88
Sum of SiO <sub>2</sub> , Al <sub>2</sub> O <sub>3</sub> , & Fe <sub>2</sub> O <sub>3</sub>	89.16
Magnesium Oxide (MgO)	1.24
Sulfur Trioxide (SO <sub>3</sub> )	1.58
Moisture Content	11.11
Loss On Ignition	3.02
Calcium Oxide (CaO)	5.25

AUTHORIZED SIGNATURE:



**JTM** JTM INDUSTRIES, INC.

POST OFFICE BOX 38  
THOMPSONS, TEXAS 77481-0038  
(713) 343-0077  
FAX (713) 545-9205

Report of Scrubber Base Limestone Generating Station Jewett, Texas

DATE: February 21, 1996

LABORATORY NUMBER: LEGS-100-SB

COMPOSITE MONTH  
January 1996

CHEMICAL ANALYSIS

Silicon Dioxide (SiO <sub>2</sub> )	52.49
Alumium Oxide (Al <sub>2</sub> O <sub>3</sub> )	17.75
Iron Oxide (Fe <sub>2</sub> O <sub>3</sub> )	7.99
Sum of SiO <sub>2</sub> , Al <sub>2</sub> O <sub>3</sub> , & Fe <sub>2</sub> O <sub>3</sub>	78.23
Magnesium Oxide (MgO)	1.90
Sulfur Trioxide (SO <sub>3</sub> )	5.10
Moisture Content	5.14
Loss On Ignition	0.19
Calcium Oxide (CaO)	10.21

AUTHORIZED SIGNATURE:







## **APPENDIX B**



Table B-1. Liquid Limit, Plastic Limit, PI, and pH of A-7-6 Soil from Turkey Creek.

Lime, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index, %	pH
0	87.3	25.2	62.1	7.5
6	79	48.3	31.3	11.1
8	77	48.09	28.91	11.89
10	75.7	51.07	24.6	12.01
12	71.8	50.5	21.3	12.15

Table B-2. Liquid Limit, Plastic Limit, and PI of Soil from the Southwest Corner of George Bush Drive and Wellborn Road Intersection.

Lime, %	'C' Fly Ash %	'F' Fly Ash %	Liquid Limit %	Plastic Limit %	Plasticity Index %
0	0	0	44.55	16.4	28.15
4	0	0	42.5	31.06	11.44
2	0	5	39.8	24.39	15.41
0	10	0	40.8	23.03	17.77
0	15	0	41.95	26.52	15.43
0	20	0	39.7	27.39	12.31

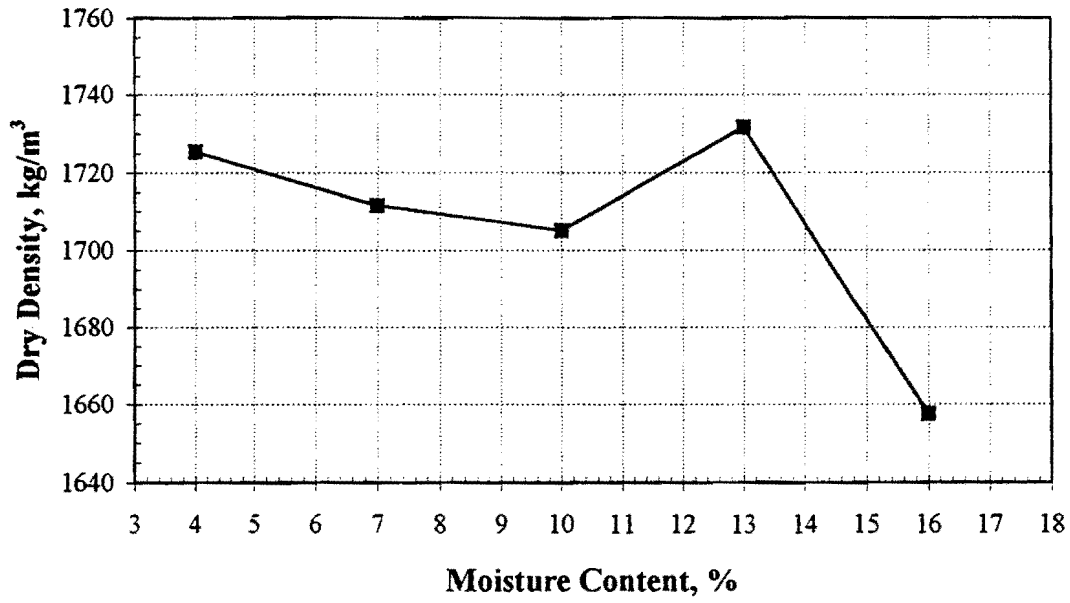


Figure B-1. Moisture-Density Relationship for A-3 Soil Without Stabilizers.

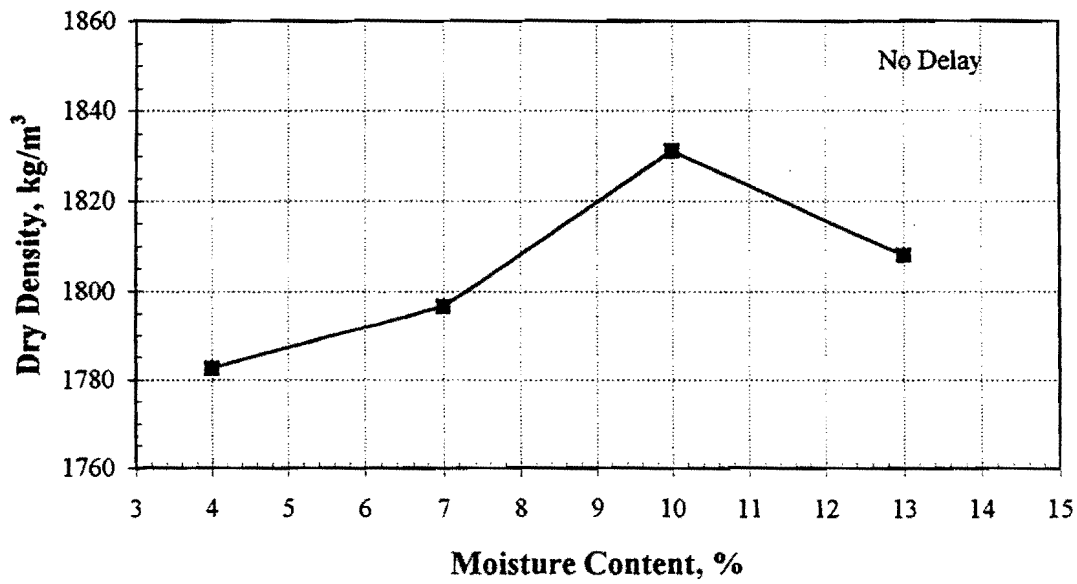


Figure B-2. Moisture-Density Relationship for A-3 Soil with 10% Class 'C' Fly Ash.

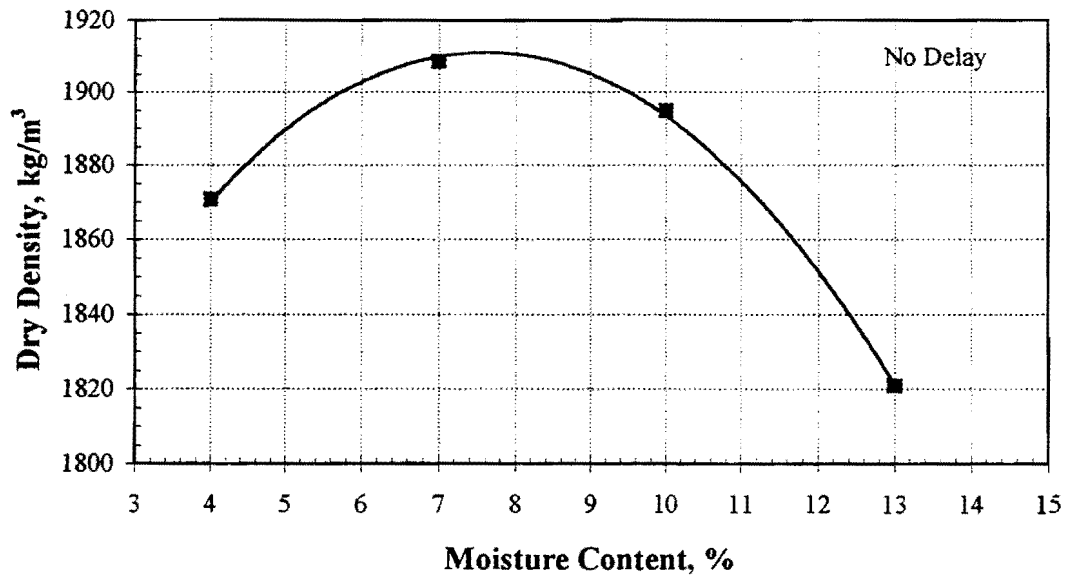


Figure B-3. Moisture-Density Relationship for A-3 Soil with 15% Class 'C' Fly Ash.

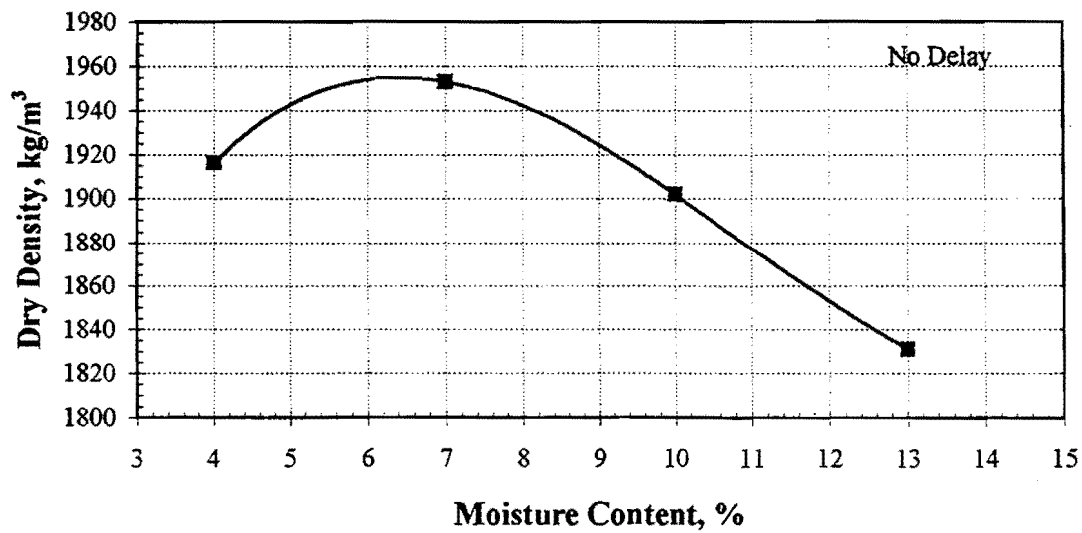


Figure B-4. Moisture-Density Relationship for A-3 Soil with 20% Class 'C' Fly Ash.

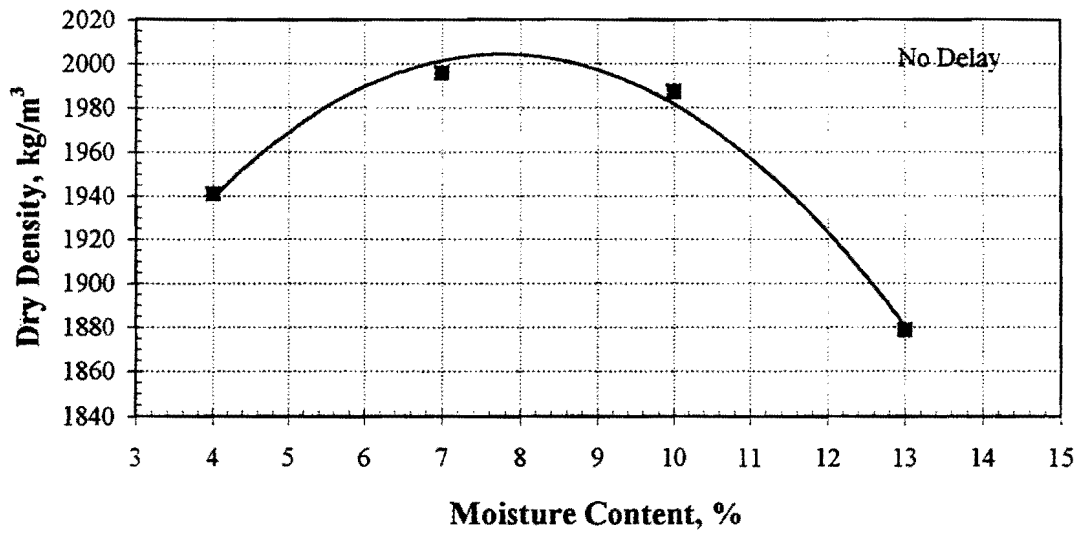


Figure B-5. Moisture-Density Relationship for A-3 Soil with 25% Class 'C' Fly Ash.

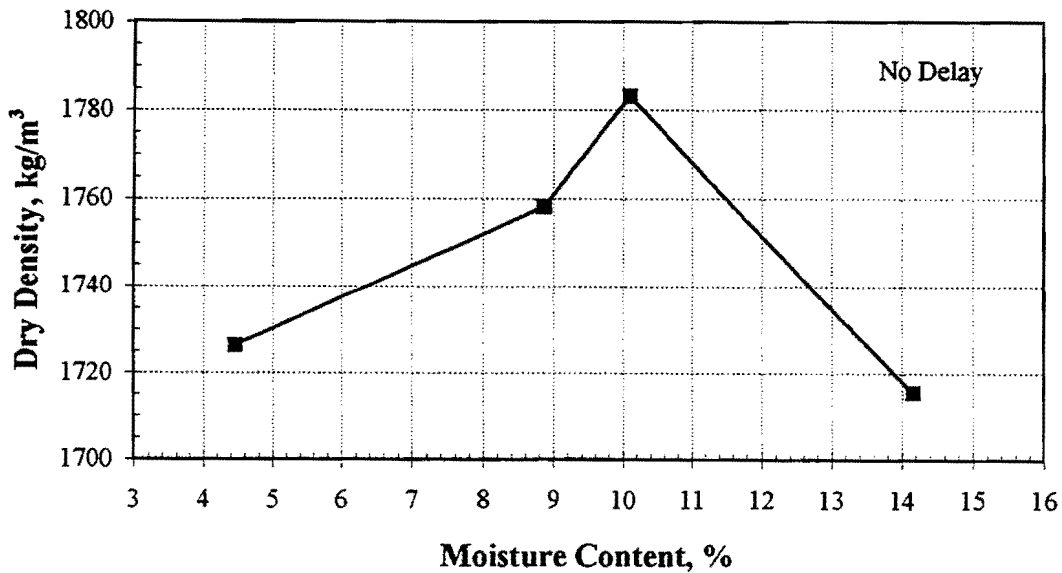


Figure B-6. Moisture-Density Relationship for A-3 Soil with 2.5% Quick Lime and 5% Class 'F' Fly Ash.

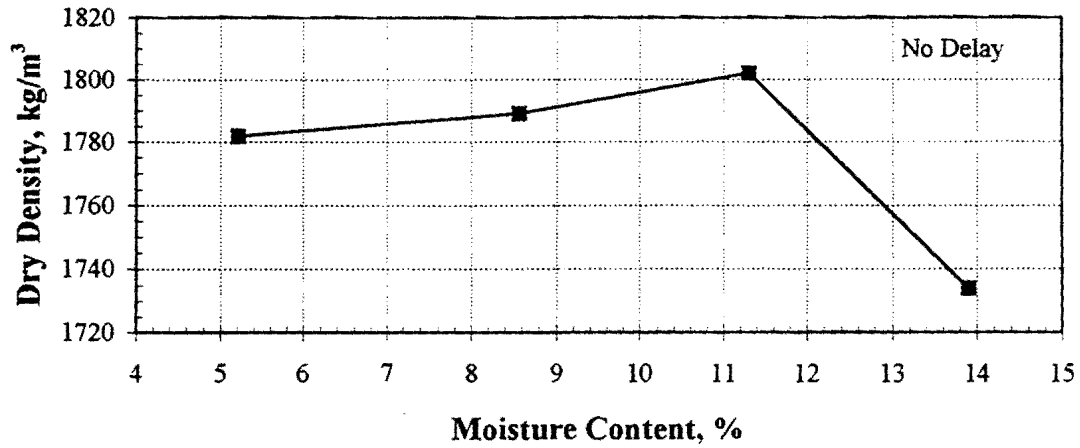


Figure B-7. Moisture-Density Relationship for A-3 Soil with 3% Quick Lime and 8% Class 'F' Fly Ash.

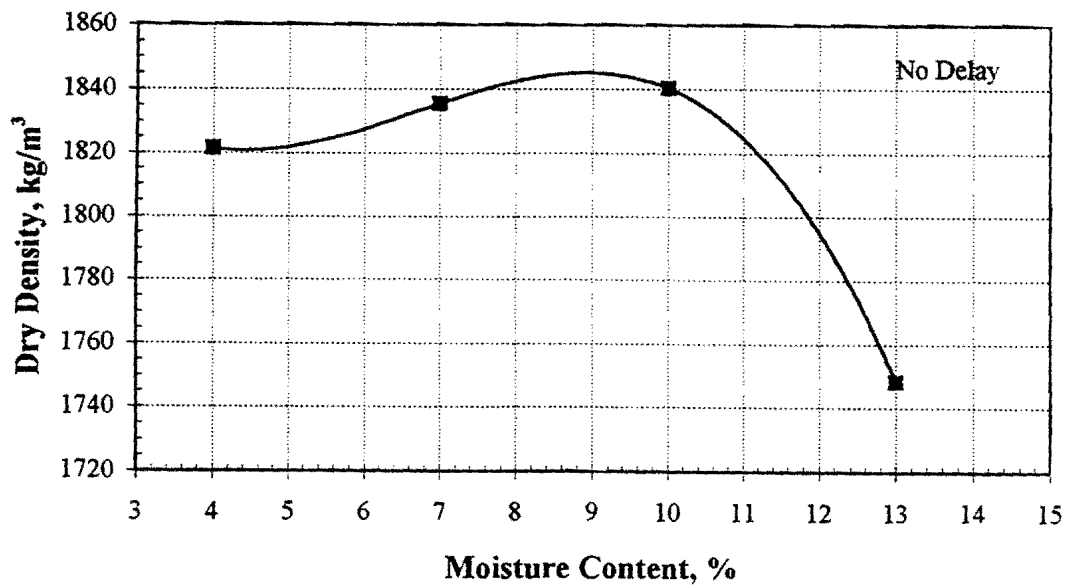


Figure B-8. Moisture-Density Relationship for A-3 Soil with 20% Class 'F' Fly Ash.

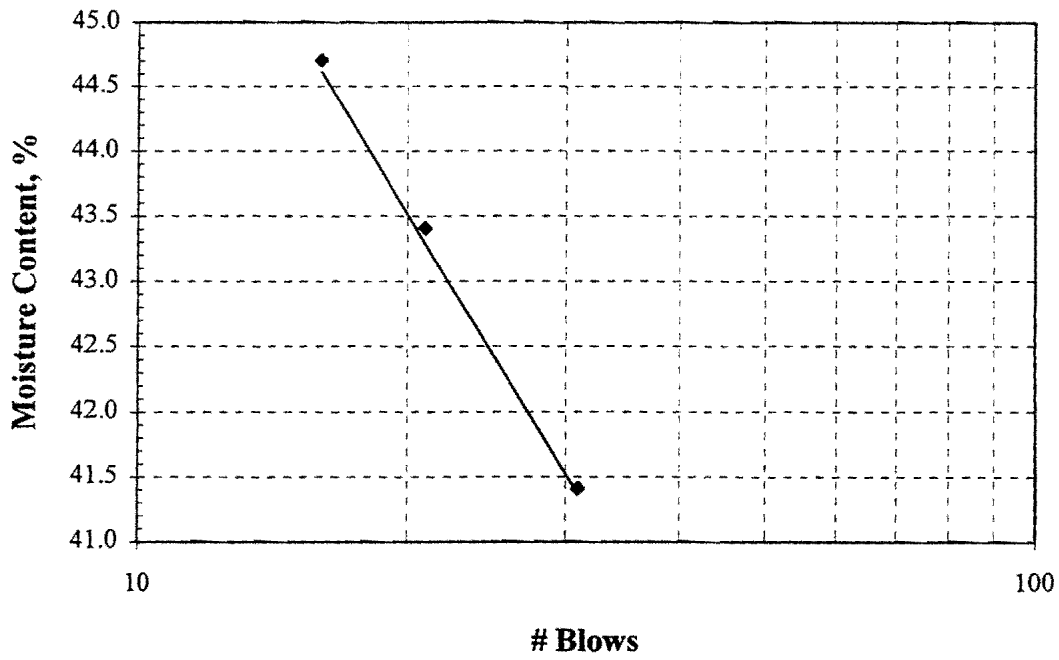


Figure B-9. Flow Curve for A-7-6 Soil with 4% Quick Lime.

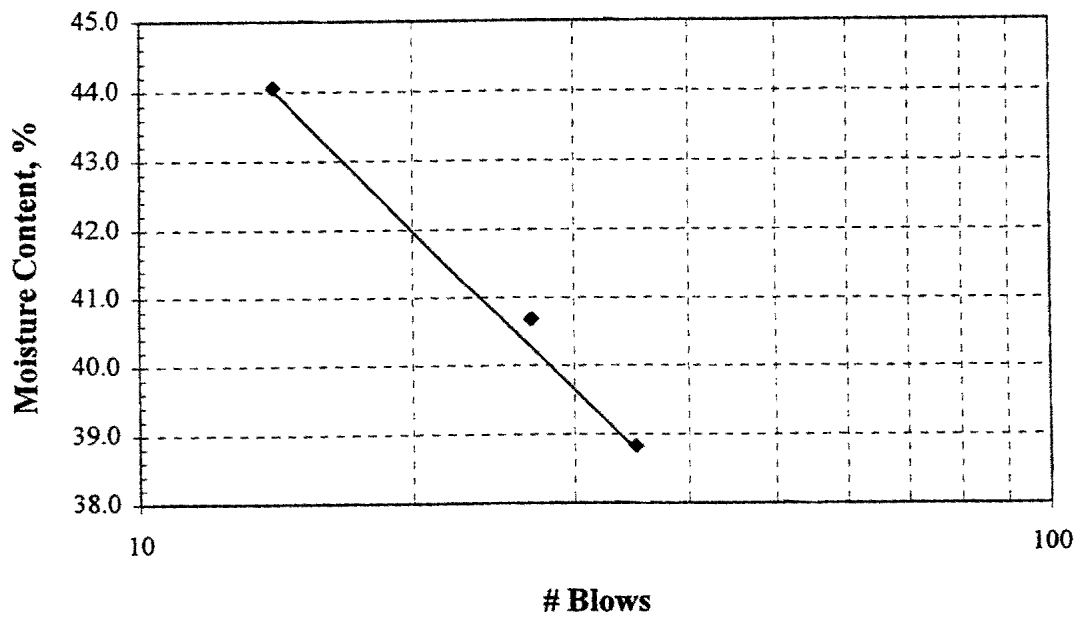


Figure B-10. Flow Curve for A-7-6 Soil with 10% Class 'C' Fly Ash.



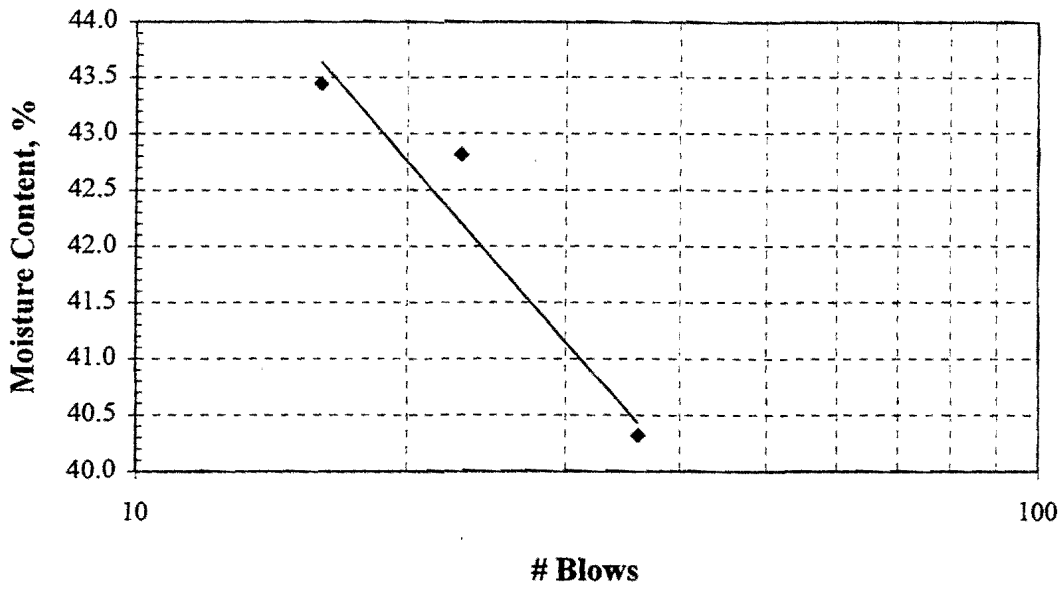


Figure B-11. Flow Curve for A-7-6 Soil with 15% Class 'C' Fly Ash.

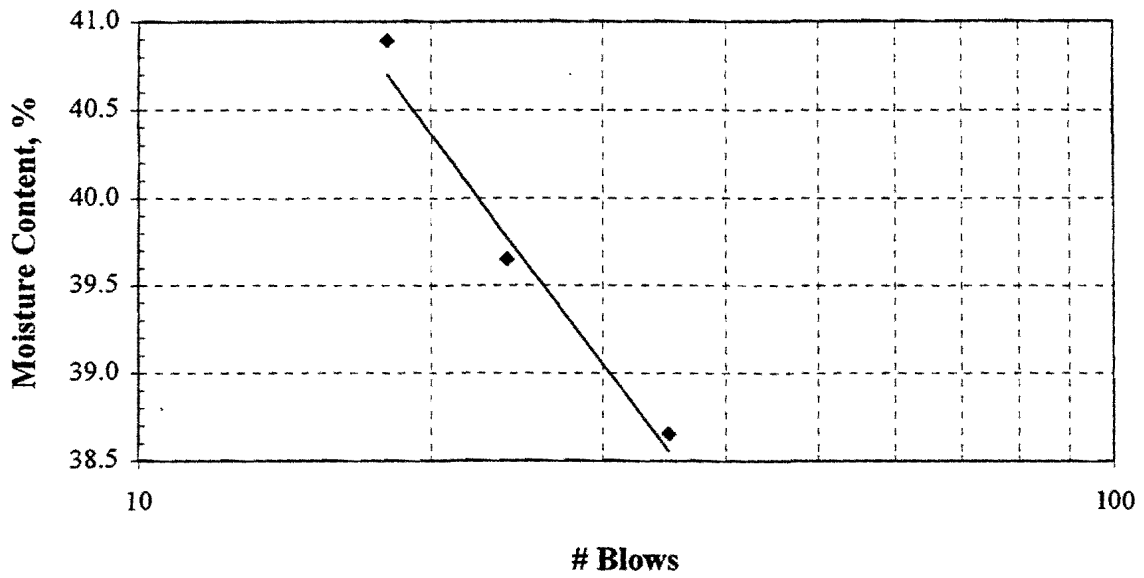


Figure B-12. Flow Curve for A-7-6 Soil with 20% Class 'C' Fly Ash.

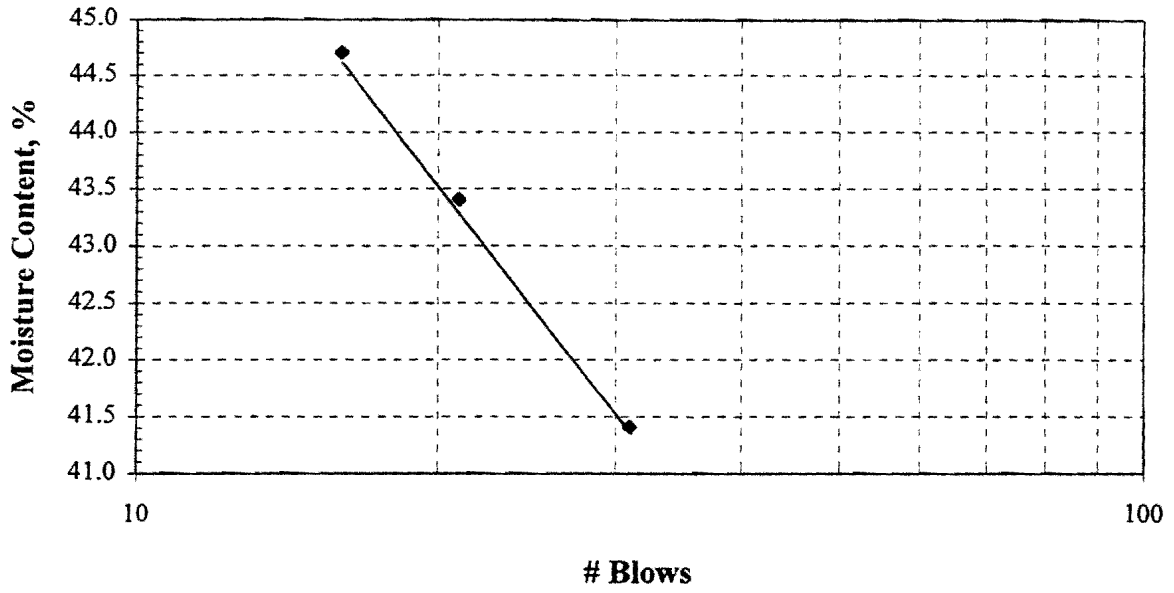


Figure B-13. Flow Curve for A-7-6 Soil with 2% Quick Lime and 5% Class 'F' Fly Ash.

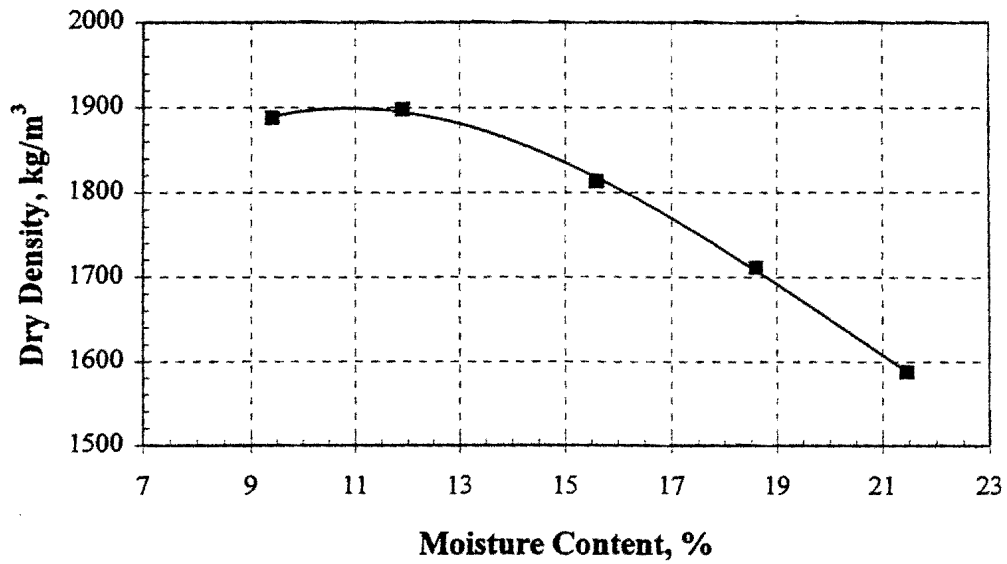


Figure B-14. Moisture-Density Relationship for A-7-6 Soil Without Stabilizers.

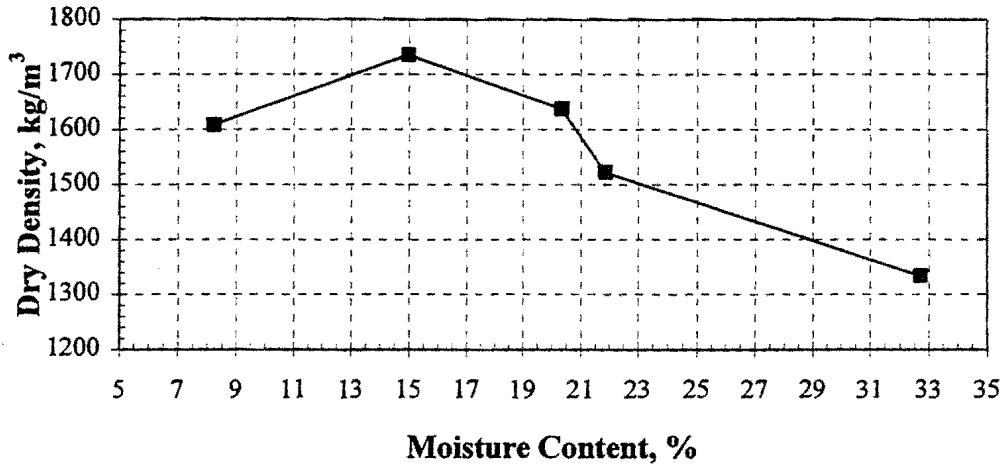


Figure B-15. Moisture-Density Relationship for A-7-6 Soil with 4% Quick Lime.

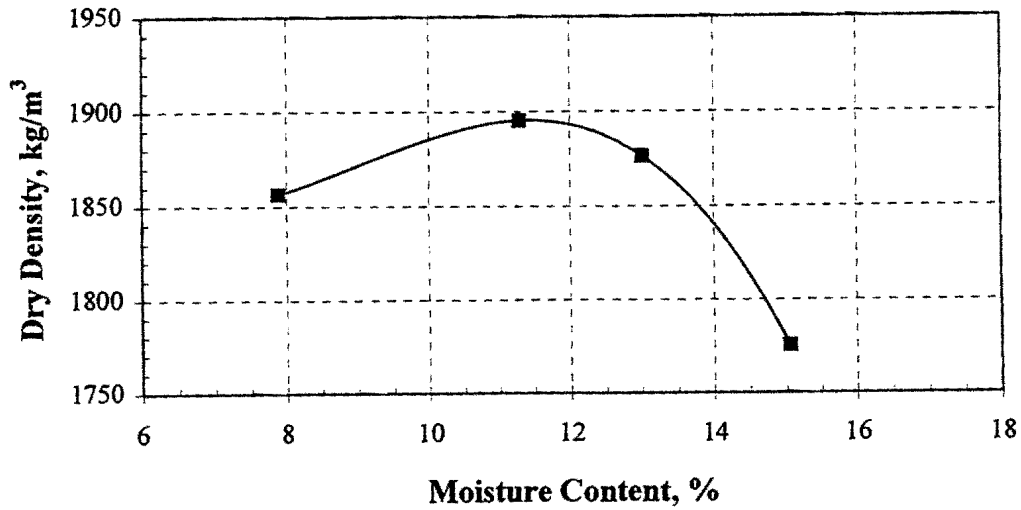


Figure B-16. Moisture-Density Relationship for A-7-6 Soil with 10% Class 'C' Fly Ash.

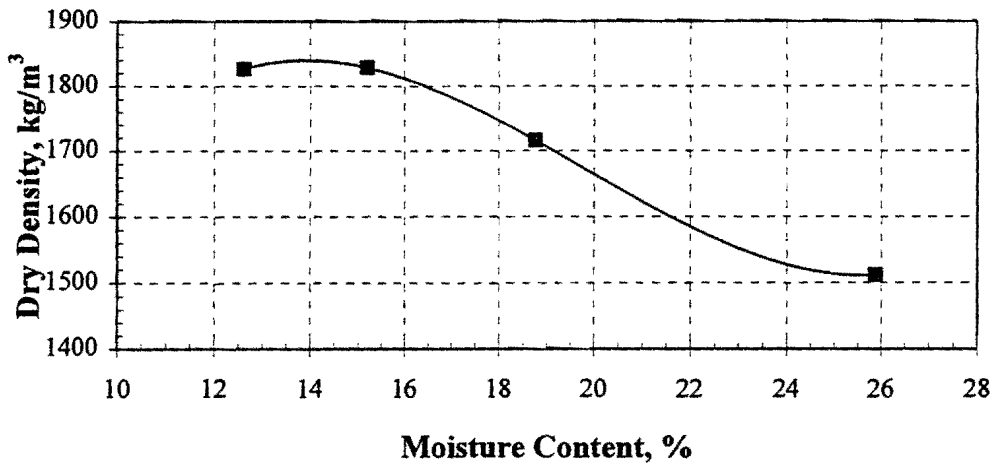


Figure B-17. Moisture-Density Relationship for A-7-6 Soil with 15% Class 'C' Fly Ash.

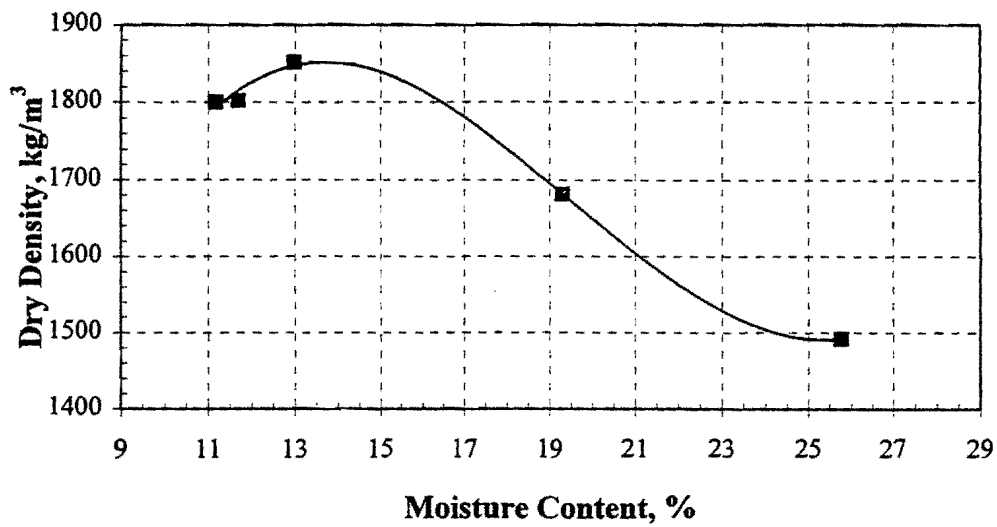


Figure B-18. Moisture-Density Relationship for A-7-6 Soil with 20% Class 'C' Fly Ash.

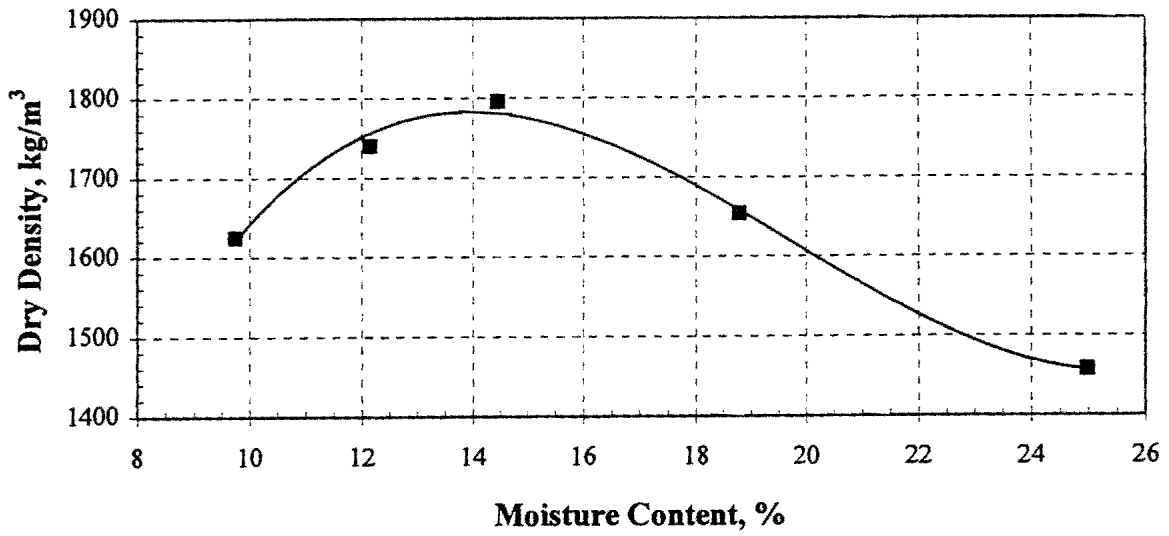


Figure B-19. Moisture-Density Relationship for A-7-6 Soil with 2% Quick Lime and 5% Class 'F' Fly Ash.

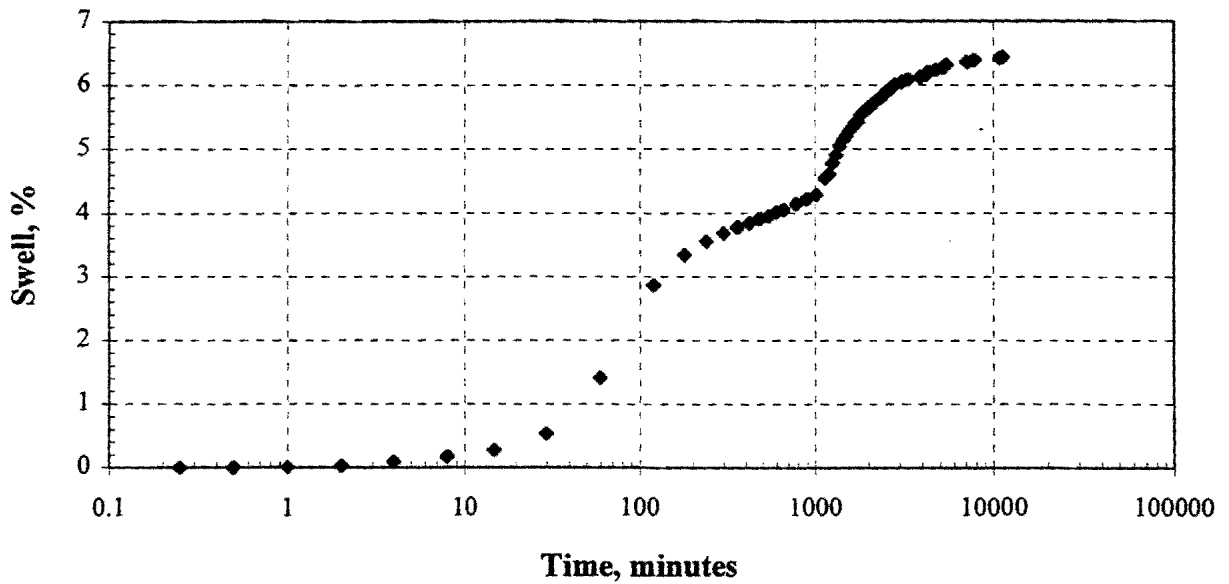


Figure B-20. Swell Curve for A-7-6 Soil Without Admixtures.

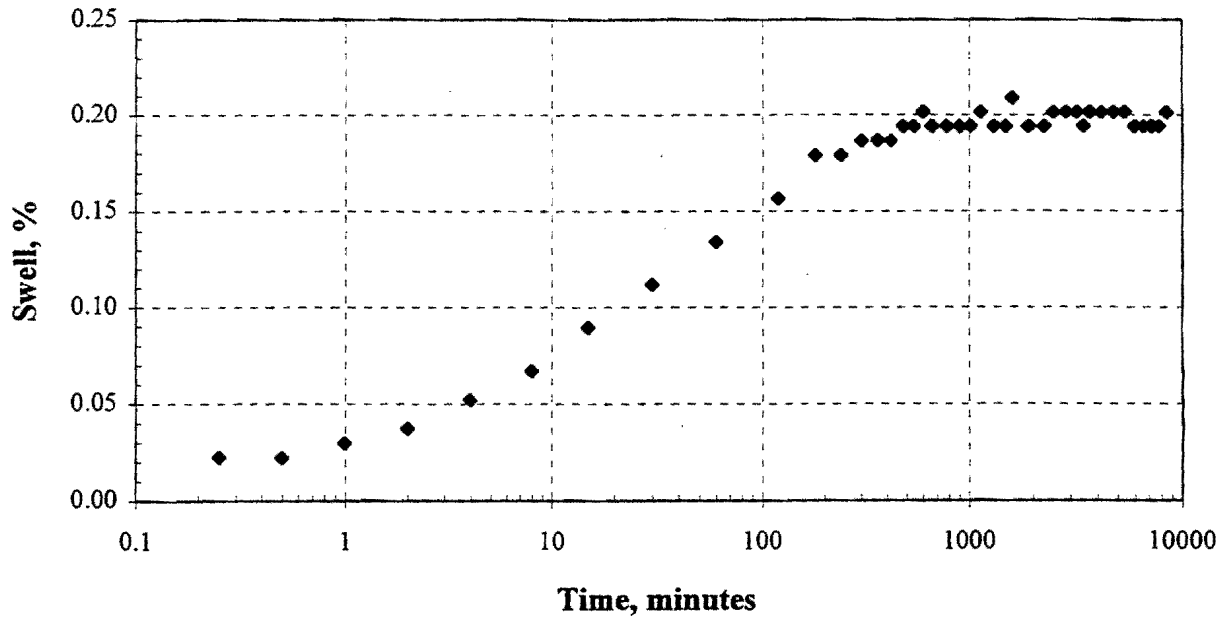


Figure B-21. Swell Curve for A-7-6 Soil with 4% Lime.

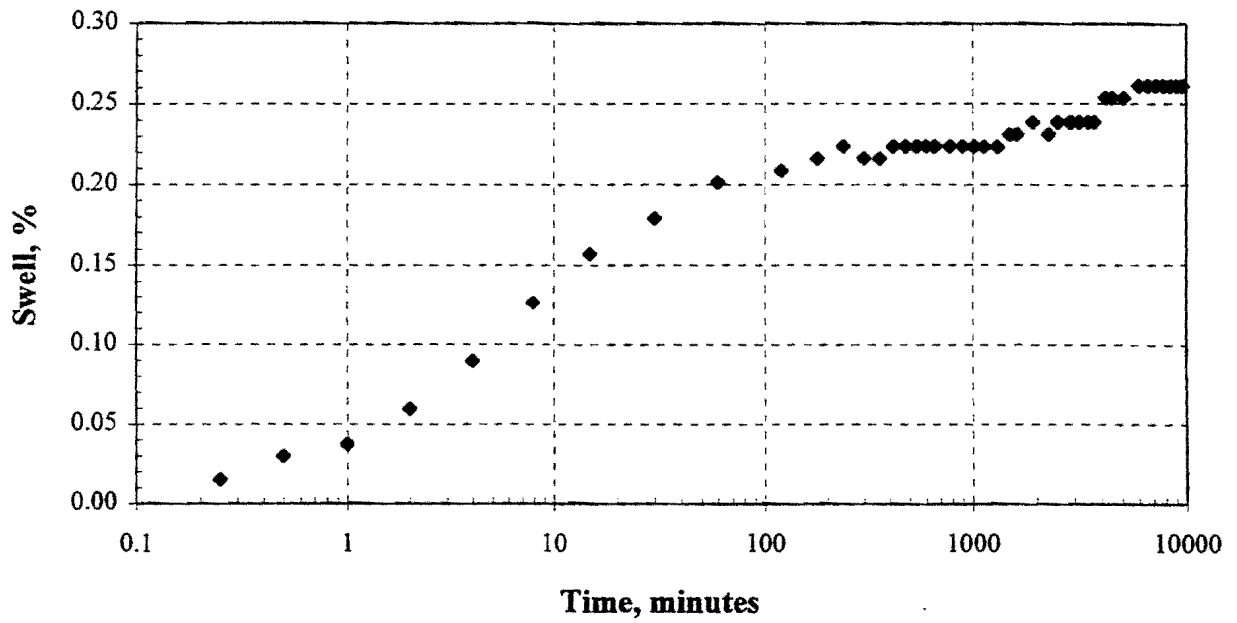


Figure B-22. Swell Curve for A-7-6 Soil with 10% Class 'C' Fly Ash.

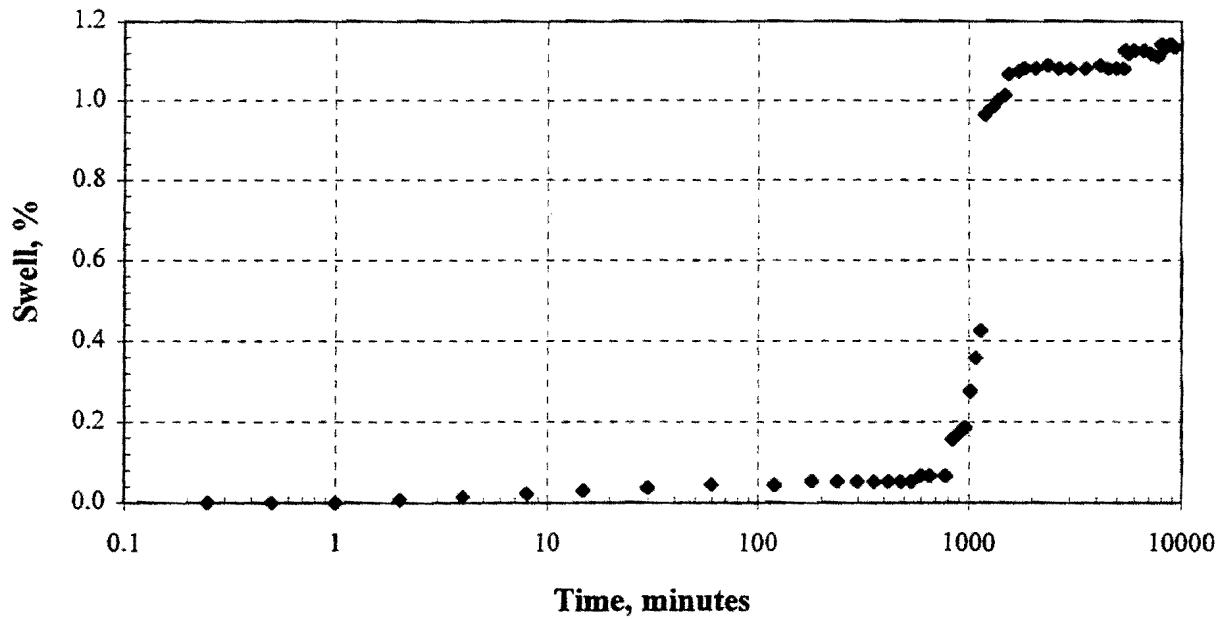


Figure B-23. Swell Curve for A-7-6 Soil with 15% Class 'C' Fly Ash.

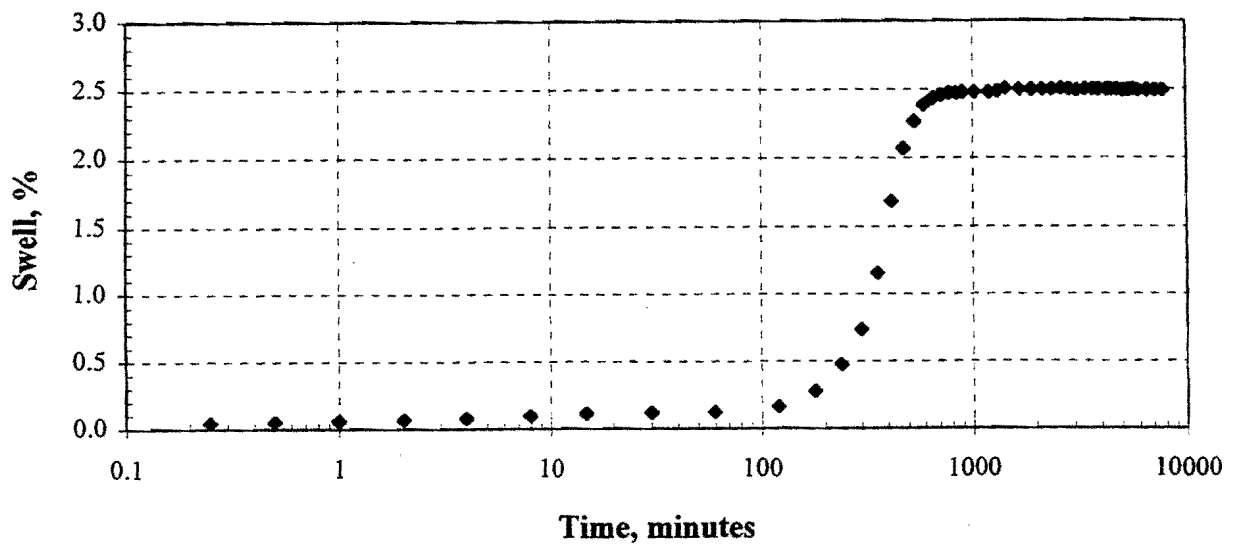


Figure B-24. Swell Curve for A-7-6 Soil with 20% Class 'C' Fly Ash.

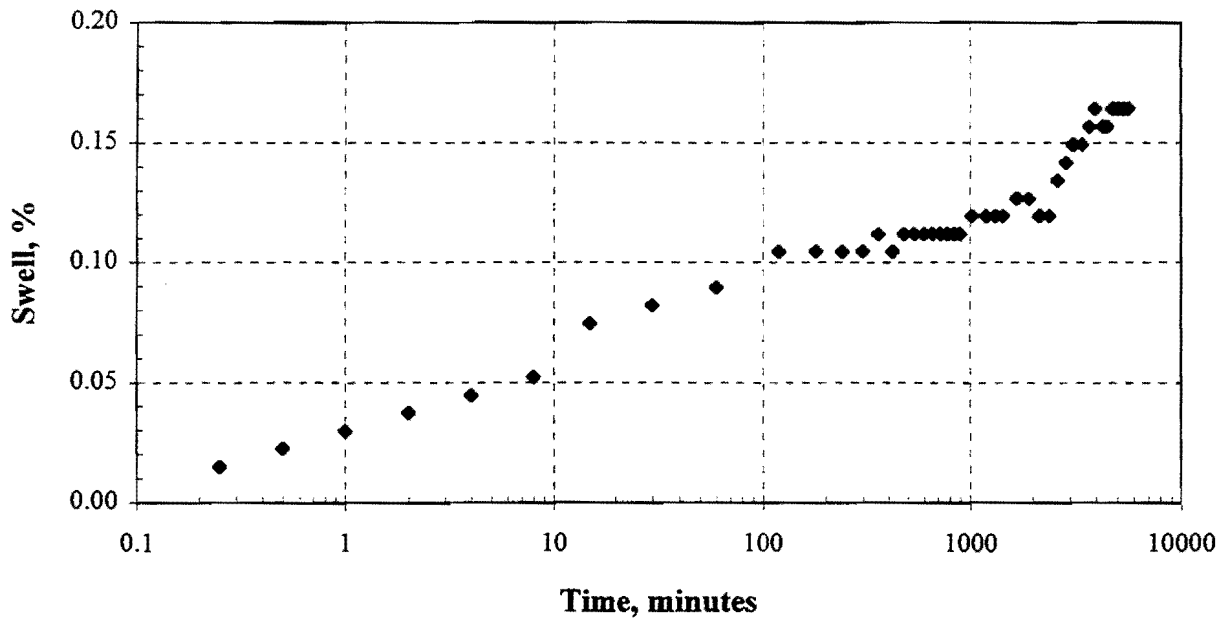


Figure B-25. Swell Curve for A-7-6 Soil with 2% Lime and 5% Class 'F' Fly Ash.



## **APPENDIX C**



**Table C-1. Fly Ash, TCLP Leachate Analyses, All Units.**

PARAMETER	EPA	TNRCC	ANALYSIS RESULTS				n
	MCL <sup>1</sup> mg/l	MCL <sup>2</sup> mg/l	Mean mg/l	Std Dev mg/l	Max mg/l	Coeff of Variance	
Antimony		1	0.159	0.168	0.500	1.056	11
Arsenic	5.0	1.8	0.132	0.110	0.500	0.834	21
Barium	100.0	100.0	1.105	2.015	10.000	1.823	21
Beryllium		0.08	0.023	0.032	0.100	1.401	7
Cadmium	1.0	0.5	0.024	0.023	0.100	0.969	21
Chromium	5.0	5.0	0.229	0.189	0.792	0.829	21
Lead	5.0	1.5	0.172	0.184	0.800	1.070	20
Mercury	0.2	0.2	0.045	0.067	0.150	1.499	21
Nickel		70	0.126	0.073	0.240	0.578	14
Selenium	1.0	1.0	0.299	0.427	1.800	1.426	27
Silver	5.0	5.0	0.160	0.261	0.750	1.627	21
Vanadium			0.386	0.415	1.100	1.076	4

- Notes:**
1. Maximum level for exclusion from Toxicity Characteristic: 40 CFR Part 261, Appendix II
  2. Maximum level for exclusion from TNRCC Waste Class 1: 30 TAC Chapter 335, Subchapter R, Appendix 1, Table 1.

**Utilities/Plants Included:**

TU Electric	Big Brown, Martin Lake, Monticello
Houston Lighting & Power	Limestone, Parrish
Lower Colorado River Authority	Fayette
City Public Service	Deely
Southwestern Electric Power Co.	Welsh, Pirkey
ALCOA	Sandow
Southwestern Public Service Co.	Tolk, Harrington

**Table C-2. Bottom Ash, TCLP Leachate Analyses, All Units.**

PARAMETER	EPA	TNRCC	ANALYSIS RESULTS				
	MCL <sup>1</sup> mg/l	MCL <sup>2</sup> mg/l	Mean mg/l	Std Dev mg/l	Max mg/l	Coeff of Variance	n
Antimony		1	0.159	0.169	0.500	1.059	11
Arsenic	5.0	1.8	0.111	0.137	0.500	1.235	23
Barium	100.0	100.0	1.902	2.638	10.000	1.387	23
Beryllium		0.08	0.028	0.039	0.100	1.370	9
Cadmium	1.0	0.5	0.025	0.028	0.100	1.147	23
Chromium	5.0	5.0	0.088	0.145	0.500	1.648	23
Lead	5.0	1.5	0.116	0.136	0.500	1.172	22
Mercury	0.2	0.2	0.022	0.050	0.150	2.310	23
Nickel		70	0.079	0.042	0.180	0.534	13
Selenium	1.0	1.0	0.076	0.066	0.200	0.872	23
Silver	5.0	5.0	0.068	0.137	0.500	2.011	23
Vanadium			0.112	0.118	0.300	1.054	5

- Notes:**
1. Maximum level for exclusion from Toxicity Characteristic; 40 CFR Part 261, Appendix II
  2. Maximum level for exclusion from TNRCC Waste Class 1; 30 TAC Chapter 335, Subchapter R, Appendix 1, Table 1.

**Utilities/Plants Included:**

TU Electric	Big Brown, Martin Lake, Monticello
Houston Lighting & Power	Limestone, Parrish
Lower Colorado River Authority	Fayette
City Public Service	Deely
Southwestern Electric Power Co.	Welsh, Pirkey
Southwestern Public Service Co.	Harrington

**Table C-3. FGD Material, TCLP Leachate Analyses, All Units.**

PARAMETER	EPA	TNRCC	ANALYSIS RESULTS				
	MCL <sup>1</sup> mg/l	MCL <sup>2</sup> mg/l	Mean mg/l	Std Dev mg/l	Max mg/l	Coeff of Variance	n
Antimony		1	0.236	0.221	0.500	0.936	5
Arsenic	5.0	1.8	0.081	0.132	0.500	1.627	12
Barium	100.0	100.0	1.202	2.693	10.000	2.241	12
Beryllium		0.08	0.008	0.002	0.010	0.306	5
Cadmium	1.0	0.5	0.018	0.025	0.100	1.444	12
Chromium	5.0	5.0	0.085	0.150	0.500	1.765	12
Lead	5.0	1.5	0.090	0.128	0.500	1.423	12
Mercury	0.2	0.2	0.003	0.005	0.020	1.938	12
Nickel		70	0.152	0.171	0.460	1.125	6
Selenium	1.0	1.0	0.392	0.464	1.500	1.185	12
Silver	5.0	5.0	0.138	0.290	0.990	2.103	12
Vanadium			0.137	0.089	0.200	0.645	3

- Notes:**
1. Maximum level for exclusion from Toxicity Characteristic; 40 CFR Part 261, Appendix II
  2. Maximum level for exclusion from TNRCC Waste Class 1; 30 TAC Chapter 335, Subchapter R, Appendix 1, Table 1.

**Utilities/Plants Included:**

TU Electric	Martin Lake, Monticello
Houston Lighting & Power	Limestone, Parrish
Lower Colorado River Authority	Fayette
Texas Municipal Power Authority	Gibbons Creek
Southwestern Electric Power Co.	Pirkey
City Public Service	Spruce

## **APPENDIX D**



**Table D-1. Fly Ash TNRCC Leachate Analyses - All Units.**

PARAMETER	TNRCC Leachate MCL* mg/l	SDWA PDWS MCL Grab mg/l	ANALYSIS RESULTS				
			Mean mg/l	Std Dev mg/l	Max mg/l	SD/Mean	n
Antimony			0.100	0.000	0.100	0.000	2
Arsenic	0.05	0.05	0.013	0.012	0.040	0.947	47
Barium	1	1	14.74	31.84	216.65	2.16	74
Beryllium			0.023	0.019	0.050	0.808	6
Cadmium	0.005	0.010	0.010	0.007	0.070	0.689	78
Chromium	0.1	0.05	0.068	0.121	0.480	1.773	87
Copper		1**	0.039	0.049	0.176	1.249	22
Lead	0.05	0.05	0.011	0.009	0.040	0.818	47
Manganese		0.05**	0.803	0.575	1.430	0.717	16
Mercury	0.002	0.002	0.001	0.001	0.005	1.031	47
Nickel			0.023	0.014	0.040	0.596	11
Selenium	0.05	0.01	0.056	0.108	0.865	1.931	87
Silver	0.05		0.010	0.008	0.050	0.772	87
Zinc			0.034	0.036	0.120	1.083	28
TDS	500	500**	1059	396	2078	0.374	55

\* Maximum Contaminant Level; 30 TAC 335, SubPart R, Appendix 1, Table 3, TNRCC Leachate  
 \*\* Secondary Drinking Water Standards

**Utilities/Plants included:**

Texas Utilities	Monticello
Texas Municipal Power Authority	Gibbons Creek
Lower Colorado River Authority	Fayette Power Project, Units 1 and 3
City Public Service	J.T. Dealy, J.K. Spruce

All analyses performed using the Texas Natural Resource Conservation Commission's (Demineralized Water) Leachate Procedure.



**Table D-2. Bottom Ash TNRCC Leachate Analyses - All Units.**

PARAMETER	TNRCC	SDWA	ANALYSIS RESULTS				
	Leachate MCL*	PDWS MCL Grab mg/l	Mean mg/l	Std Dev mg/l	Max mg/l	SD/Mean	n
Antimony			0.190	0.100	0.330	0.528	3
Arsenic	0.05	0.05	0.021	0.037	0.160	1.770	61
Barium	1	1	0.368	0.220	1.170	0.597	88
Beryllium			0.030	0.020	0.050	0.667	6
Cadmium	0.005	0.010	0.009	0.003	0.020	0.315	94
Chromium	0.1	0.05	0.102	0.093	0.380	0.911	102
Copper		1**	0.021	0.036	0.160	1.696	16
Lead	0.05	0.05	0.020	0.017	0.050	0.841	61
Manganese		0.05**	0.086	0.182	0.680	2.121	22
Mercury	0.002	0.002	0.001	0.001	0.002	0.602	61
Nickel			0.025	0.015	0.050	0.594	19
Selenium	0.05	0.01	0.011	0.013	0.087	1.139	102
Silver	0.05		0.011	0.009	0.050	0.823	101
Zinc			0.040	0.055	0.180	1.355	35
TDS	500	500**	997	776	4064	0.779	54

\* Maximum Contaminant Level; 30 TAC 335, SubPart R, Appendix 1, Table 3, TNRCC Leachate  
 \*\* Secondary Drinking Water Standards

**Utilities/Plants included:**

Texas Utilities	Big Brown, Monticello, Martin Lake
Texas Municipal Power Authority	Gibbons Creek
Lower Colorado River Authority	Fayette Power Project, Units 1 and 3
City Public Service	J.T. Dealy, J.K. Spruce
Southwestern Public Service	Harrington Units 1.2. & 3

All analyses performed using the Texas Natural Resource Conservation Commission's (Demineralized Water) Leachate Procedure.

**Table D-3. FGD Material TNRCC Leachate Analyses - All Units.**

PARAMETER	TNRCC Leachate MCL*	SDWA PDWS MCL Grab mg/l	ANALYSIS RESULTS				n
			Mean	Std Dev	Max	SD/Mean	
			mg/l	mg/l	mg/l		
Antimony			0.260	0.000	0.260	0.000	1
Arsenic	0.05	0.05	0.015	0.014	0.040	0.915	21
Barium	1	1	0.106	0.301	1.820	2.827	34
Beryllium			0.018	0.016	0.050	0.889	5
Cadmium	0.005	0.010	0.043	0.163	1.010	3.748	38
Chromium	0.1	0.05	0.032	0.068	0.430	2.125	41
Copper		1**	0.043	0.046	0.130	1.071	9
Lead	0.05	0.05	0.013	0.010	0.030	0.717	21
Manganese		0.05**	0.138	0.181	0.450	1.315	4
Mercury	0.002	0.002	0.001	0.001	0.004	0.820	21
Nickel			0.162	0.253	0.800	1.557	9
Selenium	0.05	0.01	0.149	0.339	1.400	2.273	41
Silver	0.05		0.012	0.006	0.040	0.552	40
Zinc			0.028	0.039	0.140	1.366	12
TDS	500	500**	4460	4061	21090	0.911	23

\* Maximum Contaminant Level: 30 TAC 335, SubPart R, Appendix 1, Table 3, TNRCC Leachate  
 \*\* Secondary Drinking Water Standards

**Utilities/Plants Included:**

Texas Utilities	Monticello
Lower Colorado River Authority	Fayette Power Project, Unit 3

All analyses performed using the Texas Natural Resource Conservation Commission's (Demineralized Water) Leachate Procedure.