

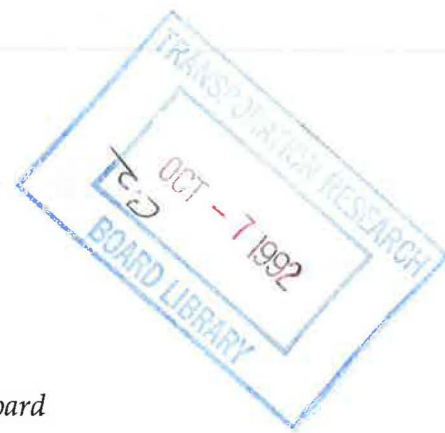
# TRANSPORTATION RESEARCH RECORD

No. 1345

*Soils, Geology, and Foundations*

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## Geoenvironmental and Engineering Properties of Rock, Soil, and Aggregate



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# Foreword

One major concern of the modern industrialized world is the production of enormous quantities and varieties of waste materials, either as by-products or as residues. Therefore, it is not surprising that the list of materials considered to be waste is a long one.

In recent years, a shortage of suitable conventional construction materials in some regions of the world and increasing accumulations of waste materials worldwide have prompted governments at all levels to enact requirements on the use of waste and recycled materials, the unconventional materials. This has triggered a large number of investigations by researchers from government agencies, academia, and the private sector into various aspects of unconventional materials. The 15 papers included in this Record primarily provide information on current research activities in the United States and only a glimpse of similar activities in the international arena.

The materials investigated for feasibility include various types of ash, rubber tires, glass, plastics, industrial sludge, crushed concrete, and so forth. The applications described are for construction of asphalt pavements, embankments, stabilization of pavement base materials, and other transportation structures. The papers focus on physical and chemical properties, performance, cost-effectiveness, and assessment of the environmental effects of the materials investigated by the researchers.

# Use of Waste Materials in Highway Construction: State of the Practice and Evaluation of the Selected Waste Products

IMTIAZ AHMED AND C. W. LOVELL

The use of waste materials in highway construction in the United States and applications of selected waste materials, including waste tires, waste glass, reclaimed paving materials, slags and ashes, building rubble, and sewage sludge are discussed. An evaluation based on technical, environmental, and economic factors has indicated that reclaimed paving materials, coal fly ash, blast furnace slag, bottom ash, boiler slag, steel slag, and rubber tires have significant potential to replace conventional materials for various applications in highway construction and should be projected for future construction. Specific applications of the waste products and the potential problems associated with their usage in highway operations, which must be addressed before their extensive use, are included.

Enormous quantities of domestic, industrial, and mining waste are generated annually in the United States. There are three techniques for disposal of these waste materials: (a) recycling, (b) incineration with or without generation of energy, and (c) burial. The published data on current practice indicate that the bulk of domestic refuse is either incinerated or landfilled. Of the total municipal solid waste (MSW) generated in 1988 (i.e., 180 million tons), 13.1 percent was recovered, 14.2 percent was incinerated, and 72.7 percent was landfilled (1). Public concern is constantly expressed about the vast quantities of useful materials being discarded or destroyed. Legislation intended to stimulate recycling efforts is in force in a number of states and is being debated in others.

The state of the practice in the use of waste materials in highway construction in the United States and the performance of the selected waste products are discussed. Technical feasibility, environmental consequences, and economic benefits are considered.

## STATE OF THE PRACTICE

### Questionnaire Survey

To obtain information on current practices in the United States on the use of waste materials in highway construction, a questionnaire was developed and distributed to each state highway agency. The questionnaire requested information on the type

of waste materials currently used in highway construction; their applications, annual quantities, and field performance; the materials and applications that appeared favorable and would be projected for future construction; and the materials and uses mandated by state laws. Of the 52 questionnaires distributed, 44 were returned, representing a return ratio of 85 percent (2).

### Overview of Current Practice

A total of 27 waste products were reported by the 44 state highway agencies responding to the questionnaire. The waste products are currently in use (or being studied experimentally) in a variety of highway applications (see Table 1). Of the 27 waste products, only 11 (reclaimed paving materials, coal fly ash, rubber tires, blast furnace slag, steel slag, coal bottom ash, boiler slag, used motor oil, waste paper, mine tailings, and sewage sludge) are presently used by more than about 7 percent of the respondents. The six waste products currently used (or being studied experimentally) by two of the respondents are building rubble, waste glass, sawdust, ceramic waste, incinerator residue, and highway hardware. The remaining 10 waste products were reported by one of the respondent state highway agencies and are generally available in lesser quantities or their production is restricted to limited geographical locations.

Current practice indicates that reclaimed paving materials, fly ash, and rubber tires are used by a large number of respondents (98, 75, and 68 percent, respectively). The use of blast furnace slag has also been reported by a significant number of respondents (39 percent). The use of steel slag, bottom ash, and boiler slag also seems fairly attractive for highway applications (used by 16 to 20 percent of the respondents). The remaining products are less frequently used by the respondents.

The respondent state highway agencies have generally reported approximate annual quantities of waste materials currently used. The reported quantities indicate that reclaimed paving materials, slags, and ashes are generally used in large quantities. Rubber tires, although used (or studied experimentally) by a large number of states, are generally used in small quantities, with a few exceptions (Arizona, Oregon, and Vermont). This indicates that the use of tires in most of the states is generally in an experimental stage.

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TABLE 1 CURRENT USES OF WASTE MATERIALS IN THE UNITED STATES HIGHWAY INDUSTRY

Waste Material	States Using the Material <sup>a</sup>	Material is Used as Additive to <sup>b</sup> :/ Material is Used as <sup>b</sup> :				Other Uses <sup>c</sup>
		Wearing Course	Base	Subbase	Subgrade/ Embankment	
Reclaimed Paving Materials	43	23/8	26/16	14/8	6/5	3 (sh)
Coal Fly Ash	33	22/1	6/2	7/0	5/2	9 (cc)
Scrap Tires	30	22/0	6/1	1/0	3/3	11 (cs)
Blast Furnace Slag	17	5/4	3/5	0/3	1/2	4 (cc)
Steel Slag	9	4/1	2/2	1/0	0/2	1 (ic)
Coal Bottom Ash	7	2/0	2/1	1/1	1/1	3 (ic)
Boiler Slag	7	4/0	1/1	0/1	0/1	3 (ic)
Used Motor Oil	7	-	-	-	1/0	3 (recycle)
Waste Paper	7	-	-	-	-	3 (apf) 5 (ls) 2 (recycle)
Mine Tailings	5	-	-	-	0/5	-
Sewage Sludge	3	1/0	-	-	-	3 (ls)
Building Rubble	2	-	1/0	1/0	0/1	-
Waste Glass	2	1/0	2/0	-	-	-
Sawdust	2	-	-	-	0/2	-
Ceramic Waste	2	1/0	1/0	1/1	0/1	1 (pb)
Incinerator Residue	2	2/0	1/0	-	-	-
Highway Hardware	2	-	-	-	-	2 (recycle)

<sup>a</sup>Of the 44 states who responded to the questionnaire.

<sup>b</sup>The figures under each column indicate the total number of states that currently use the material.

<sup>c</sup>Abbreviations used: sh-shoulders, cc-cement concrete, cs-crack sealant, ic-ice control, apf-asphalt plant fuel, ls-landscaping, and pb-pipe bedding.

<sup>d</sup>A (-) indicate data not applicable.

The evaluation of waste materials with respect to economic, technical, and environmental factors is generally reported as at least competitive with conventional materials, satisfactory, and acceptable, respectively, with some exceptions. The most varied experience is reported in the case of rubber tires. Of the 30 state highway agencies that reported the use of waste tires in highway construction, 64 percent described their experience in the use of this product in asphalt pavements. In summary, 50 percent of those who reported their experience consider its use as uneconomical, 30 percent experienced poor performance, and 9 percent are doubtful about its environmental acceptability. The use of glass is reported as uneconomical by the only state highway agency that offered comments. One state highway agency identified potential problems with steel slag related to its expansive nature when used as an aggregate in portland cement concrete (PCC) and also expressed doubts about its environmental acceptability. Some of the agencies also expressed doubts about the environmental acceptability of reclaimed paving materials, fly ash, blast furnace slag, and sewage sludge. The only state highway agency that reported experience in the use of incinerator residue considers it environmentally unacceptable.

The survey indicates that the use of waste materials, in the majority of the respondent states, is not required by state laws. However, a number of state legislatures are considering required use of some waste products in highways to reduce waste disposal problems. This has stimulated research and investigations to determine the suitability of a number of waste products. Ten waste products were selected and have been evaluated in some detail in later sections.

## RUBBER TIRES

### Background

An estimated 240 million waste tires are discarded annually in the United States. Generation was higher in the 1970s and early 1980s, but the trend to smaller and longer-wearing tires has reduced the quantities (1). Projected estimates indicate a modest growth in tonnage and nearly a flat percentage of total generation (2.0 million to 2.2 million tons from 1995 to 2010). Small amounts of rubber are recovered for recycling (5.6 percent was recovered in 1988).

Tires occupy a large landfill space. Disposal of large quantities of tires accordingly has many economic and environmental implications. Scrap tire piles, which are growing each year, pose two significant threats to the public: fire hazard (once set ablaze, they are almost impossible to extinguish) and health hazard (the water held by the tires provides an ideal breeding ground for mosquitoes) (3).

### Use of Scrap Tires in Asphalt Pavements

Crumb rubber additive (CRA) is the generic term for the product from scrap tires used in asphalt products. Addition of CRA to asphalt paving products can be divided into two basic processes. The wet process blends CRA with hot asphalt cement and allows the rubber and asphalt to fully react in mixing tanks to produce an asphalt-rubber binder. The dry process mixes CRA with the hot aggregate at the hot mix

asphalt (HMA) facility before adding the asphalt cement to produce a rubber-modified HMA mixture. The four general categories of asphalt paving products that use CRA include crack/joint sealants, surface/interlayer treatments, HMA mixtures with asphalt-rubber binder, and rubber-modified HMA mixtures.

#### *Crack/Joint Sealant*

Crack/joint sealant is an asphalt-rubber product blending 15 to 30 percent CRA with the asphalt cement. It is covered in the American Society for Testing and Materials' specifications (ASTM D3406). The results of the survey reported here (Table 1) indicate that 11 state highway agencies currently use asphalt-rubber as a crack/joint sealant. The performance of asphalt-rubber as a crack/joint sealant is reported to be satisfactory. Stephens (4), based on a 9-year evaluation of field performance of asphalt-rubber as joint sealant, reported that site-mixed materials performed better than premixed materials.

#### *Surface/Interlayer Treatments*

Surface/interlayer treatments may use an asphalt-rubber binder with 15 to 30 percent CRA. This application of CRA began in the late 1960s and was patented under the trade name SAM (stress absorbing membrane) and SAMI (stress absorbing membrane interlayer).

SAM is a chip seal with an asphalt-rubber sealant. The purpose of this layer is to seal the underlying cracks, thereby preventing the entry of surface water into the pavement structure. It is also intended to absorb the stresses that would lead the underlying cracks reflecting up to the surface. It is formed by applying asphalt-rubber on the road, covering it with aggregate, and seating the aggregate with a roller. The thickness of the application usually varies from  $\frac{3}{8}$  to  $\frac{5}{8}$  in. (5), and 0.5 to 0.65 gal/yd<sup>2</sup> of binder is applied to the surface. Another approach to the construction of a SAM is to proportion and mix the asphalt-rubber material and chips in a conventional asphalt hot mix plant and to place the resulting mixture on a grade with a conventional asphaltic concrete spreading machine. However, the cast-in-place SAMs have performed better (6).

SAMI is a layer, with an asphalt-rubber binder, sandwiched between the road base and an overlay. The only difference between SAM and SAMI is that SAM does not have an overlay, whereas SAMI does. The purpose of SAMI is to reduce reflection cracking by cushioning or dissipating the stresses from the underlying pavement before they are transferred to the overlay. The procedure in placing the SAMI is similar to that used in placing the SAM, with a few differences in design aspects.

#### *Asphalt-Rubber Mixtures*

Since the late 1960s, the use of asphalt-rubber binder in HMA mixtures has been researched. Two such processes have been reported.

In the McDonald process, initiated in 1968, hot asphalt cement is mixed with 25 percent ground tire rubber to establish a reaction and then is diluted with kerosene for easy application (7). The Arm-R-Shield or Arizona refinery process, initiated in 1975, was patented by the Union Oil Company. It is currently marketed by Arizona Refinery Company (ARCO). The ARCO product incorporates extender oils and 18 to 20 percent recycled rubber from scrap tires directly in the hot liquid asphalt (7). The reported benefits of using Arm-R-Shield modified hot mix surfacing include flexibility down to  $-26^{\circ}\text{C}$  ( $-15^{\circ}\text{F}$ ), higher viscosity than conventional asphalt at  $60^{\circ}\text{C}$  ( $140^{\circ}\text{F}$ ), a tougher and more elastic surface, greater resistance to aging, and recycling of used tires (8,9).

#### *Rubber Modified Asphalt Mixtures*

The concept of introducing coarse rubber particles into asphaltic pavements (using the dry process) was developed in the late 1960s in Sweden. It was originally marketed by Swedish companies under the patented name Rubit. This technology was introduced in the United States in the 1970s as the patented product PlusRide and is marketed by All Seasons Surfacing Corporation of Bellevue, Washington (10,11). The PlusRide process typically uses 3 percent by weight granulated coarse and fine rubber particles to replace some of the mix aggregates. The reported advantages of PlusRide in HMA applications are as follows: reflective and thermal pavement cracking are greatly reduced, resistance to studded tire wear is increased, skid resistance is increased, ice removal is easier, pavement tire noise is suppressed, and tires are recycled (9,12).

#### *Discussion of Scrap Tires in Asphalt Pavements*

Various laboratory and analytical studies (6,13–16) and industry publications (8,12) indicate that adding CRA to asphalt paving products (as a binder or as an aggregate) improves the engineering characteristics of the pavements, including service life. However, a careful analysis of information obtained as a result of the questionnaire survey and scrutiny of the published literature indicates that these claims are not always substantiated by the field performance of asphalt paving products containing CRA. Experience in the use of CRA in asphalt paving products showed both successes and failures.

The experience of a number of states in the use of CRA in asphalt paving products was studied to establish the causes of observed failures (2). However, it appeared that with a few exceptions, the failures and successes had been random, and no definite reasons could be offered with confidence for this unusual behavior (same percentage of CRA used in similar products under similar climatic environments demonstrated different behavior—one failed within a short period of construction, whereas the other performed much better than the control sections). Various reasons have been offered for the inadequate performance of the products (17). The writers believe that more research (analytical, laboratory, and field studies) is required to completely understand this technology.



Our study (2) indicated that asphalt paving products with CRA have also demonstrated consistently better performance in some states [e.g., Alaska (rubber-modified asphalt) and Arizona (asphalt-rubber)]. Similarly, some of the asphalt paving products, including two products that use asphalt-rubber binder (i.e., joint/crack sealant and SAMs) have displayed better performance in most of the cases and suffered fewer failures.

Various studies on the economics of using CRA in asphalt paving products (14,17,18) indicate that the products are not cost-effective, since the performance of the products is generally not commensurate with the enormous increase in cost (the cost is generally 50 to more than 100 percent above conventional materials). However, the additional cost of asphalt-rubber binder as a joint/crack sealant is justified in view of better performance. Similarly, additional costs of materials used in SAMs has also been acceptable on the basis of life cycle cost in most of the cases, due to its somewhat better performance and generally longer service life.

The asphalt paving products containing CRA are generally acceptable from an environmental viewpoint. However, some concerns have been expressed over increased air pollution as a result of adding rubber to the mix and the requirement of elevated temperatures during mixing.

The recycling of conventional asphalt pavements has gained wide popularity because of obvious economic and environmental benefits. Research studies have generally not addressed this issue (limited studies have been performed, but conclusions cannot be generalized (19)) in the cases of asphalt-rubber or rubber modified asphalt. If these pavements cannot be recycled on completion of their service lives, the disposal of these pavements will create another major waste disposal problem.

### Use of Tires in Subgrade/Embankment

Two techniques to incorporate waste tires in subgrade/embankment are to use shredded tires as a lightweight fill material and to use whole tires or their sidewalls for soil reinforcement in embankment construction. Both techniques are practical and have been researched by some of the state highway agencies. The concept of using tires in embankment is also extended to enhance the stability of steep slopes along the highways (20), for temporary protection of slopes (21), for retaining of forest roads (22), and for protection of coastal roads from erosion (23).

#### *Use of Tires as Lightweight Aggregates*

Construction of roads across soft soil presents stability problems. To reduce the weight of the highway structure at such locations, wood chips or sawdust have traditionally been used as a replacement for conventional materials. Wood is biodegradable and thus lacks durability. Rubber tires are non-biodegradable and thus more durable.

The Oregon DOT used 400,000 shredded tires as a lightweight fill on Highway 42 (Coos Bay–Roseburg) 3 mi east of Camas Valley in a slide area and described the experience as a success (24). The Mn/DOT has experimented with the use

of scrap tires in roadway fill across a swamp that was underlain with peat and muck. About 52,000 shredded tires were used as lightweight fill material in a 250-ft section of roadway. The section is reportedly performing satisfactorily (25). The University of Wisconsin-Madison has constructed a test embankment consisting of 10 sections using locally available soil and shredded tires in a number of different ways, including pure tire chips, tire chips mixed with soil, and tire chips layered with soil (26). Their preliminary monitoring and evaluation of the test embankment indicates satisfactory performance.

The Minnesota Pollution Control Agency sponsored a study on the feasibility of using waste tires in subgrade roadbeds (27). Twin City Testing Corporation of St. Paul, Minnesota, performed the laboratory study to evaluate the compounds produced by the exposure of tires to different leachate environments. They found that the recommended allowable limits set by the Minnesota Department of Health for drinking water were exceeded under "worst case" conditions for certain parameters.

The use of shredded tires in embankments as lightweight fill offers some economic and technical advantages under certain conditions. However, further research is required to address the various issues affecting long-term performance, including environmental concerns.

#### *Use of Tires for Soil Reinforcement*

Various agencies have practiced and evaluated the use of tires for soil reinforcement. Forsyth and Egan (28) described a method for use of waste tires in embankments and considered it promising. The method involves the use of tire sidewalls as mats or strips in an embankment to increase its stability. Their study indicated that the systematic inclusion of tire sidewalls benefits a fill and thus permits steeper side slopes and increases resistance to earthquake loading. Encouraged by the results of their study, Caltrans designed a tire-anchored wall system, in which tire sidewalls are used to anchor timber-retaining structures (29). The use of tires in retaining structures has also been practiced primarily for maintenance and rehabilitation of road embankments (22). Whole tires anchored in the backfill are used in various configurations for wall heights up to 10 ft. This application is economical, results in moderate face settlement, and may have aesthetic and environmental implications.

### WASTE GLASS

#### **Background**

The generation of waste glass constituted 6.7 million tons in 1960. It continued to grow over the next two decades, but then glass containers were widely replaced by other materials, principally aluminum and plastics. Thus, the fraction of glass in the MSW declined in the 1980s, from 15 million tons in 1980 to 12.5 million tons in 1988. The projected estimates demonstrate a continuous declining trend in the generation of waste glass. An increase in the recovery of waste glass for recycling is predicted, from 1.5 million tons in 1988 to 2.1 to 3.1 million tons in 1995 (1).



The most obvious use for waste glass (commonly called cullet) is to recycle it to make new glass. However, the entire waste glass generated cannot be reused by glass manufacturers since only color-sorted and contamination-free cullet is considered feasible for reuse in the glass industry. Therefore, significant quantities of glass may be available for secondary applications. The feasibility of using waste glass in highway construction has been examined in the past. Various studies evaluated the use of waste glass as aggregate replacement in PCC structures and pavements (30–32). Asphalt pavements have also been studied (33,34). Glass has also been used as unbound aggregate in base layers and as fill material in embankments (35).

#### Use of Glass in Asphalt Pavements

Two states, Connecticut and Virginia, have recently conducted feasibility studies on the use of glass in asphalt pavements (36,37). The ConnDOT study, based on a review of literature, reports that glassphalt (glass-asphalt mixes in which glass replaces the conventional aggregates) was successfully mixed and placed in at least 45 locations in the United States and Canada between 1969 and 1988, mostly on city streets, driveways, and parking lots. It identifies potential problems with glassphalt, including loss of adhesion between asphalt and glass, maintenance of an adequate level of skid resistance, and breakage of glass and subsequent raveling under studded tires. The report recommends that glassphalt be used only as a base course (if laboratory mixes prove acceptable) to minimize potential skid resistance and surface raveling problems. Their economic analysis indicates that the use of glassphalt would be uneconomical (estimated at 15 percent more than conventional HMA in Connecticut under ideal conditions).

The limited laboratory study conducted by the Virginia DOT (37) indicates that the use of glass in asphalt mixes is technically feasible (with some reservations about the ability of glass to resist moisture damage) if several restrictions are observed. The restrictions include limitation of glass content to 15 percent or less; determination of the optimum asphalt content with the target percent of glass to be used; gradation controls of 100 percent passing the  $\frac{3}{8}$ -in. sieve and a maximum of 6 percent passing the No. 200 sieve; and a tensile strength ratio (TSR) of the mix of 0.9 or higher. On economic feasibility, Hughes (37) concludes that “there is little monetary incentive to use recycled glass at the present time” in glassphalt in Virginia.

#### Use of Waste Glass in Portland Cement Concrete

The feasibility study conducted by ConnDOT (36) concluded that glass is not suitable for placement in PCC pavement or structures in ConnDOT facilities. The conclusion is based mainly on the study reported by Johnston (31), which indicated that glass is highly susceptible to alkali-aggregate reaction. The reaction between glass and cement causes expansion of glass and reduction in concrete strength. The elongated particles typical of glass cullet also present a problem with the workability of the concrete mix.

#### Use of Glass in Base Layers/Embankment Construction

The use of glass in unbound aggregate base layers is technically feasible (37). However, the use of glass as an aggregate will require it to be crushed to the appropriate gradation and pretreated if the level of contamination is not within acceptable limits. The economics of using glass in embankments depend on the local conditions.

#### RECLAIMED PAVING MATERIALS

The results of our questionnaire survey indicate that reclaimed paving materials are the most widely used waste products by the United States highway industry. Of the 44 state highway agencies responding to the questionnaire, 43 are engaged in testing, evaluation, and use of these materials in a variety of applications (see Table 1). The experiences of state highway agencies indicate that the use of these materials is economically feasible (cost competitive with the virgin materials), technically feasible (performance very good to satisfactory), and generally acceptable from an environmental viewpoint (good to satisfactory). A few state highway agencies have expressed their concerns over air pollution from effluents during heating of reclaimed asphalt pavements (RAP).

#### Recycling of Asphalt Pavements

Recycling of asphalt pavements is not a new concept. The first mention of recycling is in Warren Brothers portable asphalt plant sales brochure of 1915 (38). However, it was not until the oil crisis of the early 1970s, which rapidly increased asphalt prices and energy costs, that recycling became a feasible method of lowering highway construction costs. There have been numerous laboratory, field, and synthesis studies on the various aspects of hot mix and cold mix recycling (39–41). The experience of a number of states in recycling asphalt pavements has also been documented in NCHRP (39).

Recycling of asphalt pavements is a proven fact and many viable processes exist. It is generally cost-effective, and recycling of pavements has a positive impact on the environment. The potential problem of air pollution from asphalt plant operation can be reduced by installing emission control devices to make it environmentally safe. However, there is a need to standardize the design, construction, testing, and evaluation procedures.

#### Recycling of Concrete Pavements

The recycling of PCC pavements has been researched and practiced for a number of years in the United States (42,43). Experiences of a few state highway agencies and research findings were analyzed to determine the feasibility of recycling PCC pavements (2). Our analysis concluded that recycling of PCC pavements is technically and economically feasible. In addition, recycling of pavements will reduce waste disposal problems. However, further research is needed to address the potential problems (e.g., cracking of recycled concrete pave-

ment) and to refine the mix design procedures and construction techniques. The economics of recycling PCC pavements vary with local conditions.

## SLAGS AND ASHES

Slags and ashes, derived from the iron, steel, and electrical power industries, are perhaps the waste materials of greatest interest to the highway industry, given their wide availability and scope of uses. The by-products of the iron and steel industry, which have been historically used in the highway industry, are iron blast furnace slag and steel slag. The by-products of coal-burning plants, which have been widely tested in service and are useful for a wide range of engineering applications, are coal dry bottom ash, wet bottom ash, and fly ash.

### Iron Blast Furnace Slag

Iron ore, coke, and limestone are heated in the blast furnace to produce pig iron. Produced simultaneously in the blast furnace is a material known as blast furnace slag. It is defined as "the non-metallic by-product consisting essentially of silicates and aluminosilicate of lime and other bases," and it leaves the blast furnace resembling molten lava (35).

Selective cooling of the liquid slag results in four distinct types of blast furnace slag: (a) air-cooled (solidification under ambient conditions), which finds extensive use in conventional aggregate applications; (b) expanded or foamed (solidified with controlled quantities of water, sometimes with air or steam), which is mainly used as lightweight aggregate; (c) granulated (solidified by quick water quenching to a vitrified state), which is mainly used in slag cement manufacture; and (d) pelletized (solidified by water and air-quenching in conjunction with a spinning drum), which is used both as a lightweight aggregate and in slag cement manufacture. The bulk of iron blast furnace slag produced in the United States is of the air-cooled variety (44).

Miller and Collins (35) rank iron blast furnace slag as having the highest potential among the waste materials for use in highways. Emery (44) identified the features of air-cooled blast furnace slag that make it attractive for highway applications: low compacted bulk density (typically 1200 to 1450 kg/m<sup>3</sup>), high stability (California bearing ratio > 100) and friction angle (approximately 45 degrees), high durability and resistance to weathering and erosion; free draining and not frost susceptible; and noncorrosive to steel and concrete.

The engineering properties of air-cooled iron blast furnace slag and the current practice indicate that its use in various highway applications is economical and technically feasible. However, some doubts are expressed about its environmental acceptability that need to be further investigated.

### Steel Slag

Steel slag is a by-product of the steel industry. It is formed as the lime flux reacts with molten iron ore, scrap metal, or other ingredients charged into the steel furnace at melting

temperatures around 2800°F. During this process, part of the liquid metal becomes entrapped in the slag. The molten slag flows from the furnace into the pit area where it solidifies, after which it is transferred to cooling ponds. Metallics are removed by magnetic separation (35).

Steel slags are highly variable, even for the same plant and furnace. Steel slags have high bulk density and a potential expansive nature (volume change of up to 10 percent attributed to the hydration of calcium and magnesium oxides). In view of their expansive nature, steel slags are not feasible for use in PCC.

Steel slags have been used in the highway industry in asphalt mixes, pavement bases and shoulders, fills, and for ice control grit. Their most promising application is in asphalt mixes, since asphalt coating eliminates the expansion-related problems (44). However, the leachates from this material may be undesirable from an environmental viewpoint.

### Coal Bottom Ash

The materials collected from the burning of coal at electric utility plants are referred to as power plant ash. They are produced in two forms: bottom ash and fly ash. Bottom ash is the slag that builds up on the heat-absorbing surfaces of the furnace and subsequently falls through the furnace bottom to the ash hopper below. Depending on the boiler type, the ash under the furnace bottom is categorized as dry bottom ash (the ash in a solid state at the furnace bottom) or wet bottom ash (the ash in a molten state when it falls in water). It is more often called boiler slag. Of the 17.5 million tons of bottom ash produced in 1986 in the United States, 13.4 million tons was dry bottom ash (45).

Recently, comprehensive laboratory studies have been conducted on the feasibility of using bottom ash in highway construction at Purdue University (45-47). The results of the studies (45,46) suggested that bottom ashes have a non-hazardous nature, minimal effects on groundwater quality, low radioactivity, and low erosion potential, but that they may be potentially corrosive.

### Coal Fly Ash

Fly ash is the finely divided residue that results from the combustion of ground or powdered coal and is transported from the combustion chamber by exhaust gases. It is a siliceous material that, in the presence of water, combines with lime to produce a cementitious material with excellent structural properties. However, the properties depend on the type of coal-burning boiler. There are three types: (a) stoker-fired furnaces, usually not good for highway purposes; (b) cyclone furnaces, generally not good for use in PCC and not widely available; and (c) pulverized coal furnaces, usually the best quality and produced in large quantities (48).

Fly ash represents nearly 75 percent of all ash wastes generated in the United States (35). Our questionnaire survey shows that fly ash is the second most widely used waste product in practice. However, there is still much opportunity to expand the use of this product, since the reported data indi-

dicating that 80 percent of the fly ash produced in 1984 in the United States was wasted in disposal areas (48).

#### *Use of Fly Ash in Cement Concrete Mixes*

The technology for use of fly ash in PCC and stabilized road bases is fairly well developed and has been practiced for many years. There is an abundance of published literature including synthesis study and technology transfer guidelines (48,49) on the various aspects of the use of fly ash in PCC. It was known that PCC can benefit from the addition of fly ash as early as 1914. Subsequent research has identified many benefits of the addition of fly ash in concrete mixes, including improved workability, reduced heat of hydration, increased ultimate strength, increased resistance to alkali aggregates, resistance to sulfate attack, reduced permeability, and economy (48). The benefits realized will depend on the type of cement, fly ash, mix design, and construction procedures. A study evaluating fly ash as an admixture in PCC (50) did not recommend its use for bridge decks, heavily loaded PCC pavements, or prestressed concrete structures.

#### *Use of Fly Ash in Embankments*

Faber and DiGioia (51) described case histories of embankment projects in which fly ash was used as a fill material, and the experience is reported as successful and economical. Lewis (52) has described the construction of a fly ash highway test embankment in Illinois. The performance of this embankment has been reported as satisfactory. However, this application is presently not much practiced. Although large quantities of fly ash can be consumed in embankments, the implications of this application are less known, especially its impact on groundwater quality.

### **BUILDING RUBBLE**

Building rubble discussed in this subsection includes any suitable construction material resulting from the destruction/demolition and removal from any existing structures and buildings. Building rubble is generally a heterogeneous mixture of concrete, plaster, steel, wood, brick, piping, asphalt cement, glass, and so forth. Paulsen et al. (53) estimated that roofing waste contains about 36 percent asphalt cement, 22 percent hard rock granules, 8 percent filler, and smaller amounts of coarse aggregate and miscellaneous materials. Substantial variability in the composition of building rubbles is also expected. However, it is important to consider the feasibility of its use in highway construction, since large quantities of this material may be generated as a result of some catastrophic activity, like earthquakes. Five million tons of concrete debris was generated in the 1971 San Fernando earthquake (54).

The research and experience (53,55) in the use of building rubble indicate that it has a potential for use as subbase and subgrade/embankment material. However, its technical and environmental suitability must be determined before use. The

economics of using building rubble depends on many factors, which vary with local conditions.

### **SEWAGE SLUDGE**

Sewage sludge is created as solids are removed from wastewater during treatment. Most of the sludge is harmless organics. Nutrients like nitrogen and phosphorus are also present and can make for an effective fertilizer. But sludge also contains contaminants taken from wastewater, such as heavy metals, organic carcinogens, and pathogens. Currently, more than 7 million dry tons of sludge is produced each year in the United States. Disposal is as follows: municipal landfills, 41.0 percent; incineration, 21.4 percent; land application, 21.4 percent; distributing and marketing, 9.1 percent; and others, 12.9 percent (56).

The review of available information on the use of sewage sludge indicates that its by-products (i.e., compost and incinerated ash) have potential for use as a fertilizer and as an aggregate in landscaping and highway construction, respectively (57-59). The use of compost is beneficial but has potential safety and environmental risks, whereas use of sewage ash as an aggregate has technical, economic, and environmental implications. The risks must be investigated before their use in highways.

### **CONCLUSIONS AND RECOMMENDATIONS**

An evaluation based on technical, environmental, and economic factors indicated that reclaimed paving materials, coal fly ash, blast furnace slag, bottom ash, boiler slag, steel slag, and rubber tires have significant potential to replace conventional materials for various applications in highway construction and should be projected for future construction. Technical, economic, and environmental problems associated with various applications of waste materials, identified under each waste material and briefly discussed, must be addressed before extensive use of these waste products in highway construction.

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# Physical and Environmental Properties of Asphalt-Amended Bottom Ash

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AND TAYLOR EIGHMY

A 2-year study is under way to evaluate the physical and chemical properties of the bottom ash process stream from the 500 tons/day waste-to-energy facility in Concord, New Hampshire. The use of bottom ash as an aggregate substitute product in asphaltic base course is envisioned. Research is under way to characterize the time-dependent properties of the bottom ash for product acceptance, to develop asphalt concrete mixes with varied percentages of bottom ash, and to evaluate the leachate release rate characteristics from various asphalt blends using a variety of batch and lysimeter leach tests. Results to date suggest that the bottom ash product stream is relatively constant, hot mix formulations meet New Hampshire Department of Transportation specifications, and bitumen is effective in encapsulating bottom ash and reducing salt leachability.

In the United States consideration is being given to the use of bottom ash from municipal solid waste combustion as an aggregate substitute in construction materials (1). The anticipated hierarchy for use in the United States reflects regulatory concerns that certain waste products be encapsulated or stabilized before use. Consequently, the use of bottom ash is likely to be in bituminous base course, bituminous wearing course, bituminous stabilized aggregate base, and concrete construction materials before it is used in granular subbase, structural fill, or embankment applications. This hierarchy differs somewhat from typical uses of bottom ash in Europe as a granular, soil-like material (2-4).

Earlier work in the United States by Walter (5,6) presented hot mix formulations using 50 percent bottom ash with asphalt cement contents of 5.5 to 6.5 percent by weight. Other researchers developed similar formulations, which led to a number of demonstrations in the 1970s and early 1980s using bottom ash in base course and wearing courses (7-11). General observations from these studies (1) suggest that conventional asphalt mixing and paving equipment can be used, the ash loss-on-ignition should be less than 10 percent, fly ash should not be incorporated into the blends, vibrators on feed bins are necessary, and plant temperature control is important with regard to the high moisture content of the bottom ash. These studies suggest that optimum mixes for hot mix work can contain 50 to 75 percent bottom ash substituted for conventional aggregate.

Recent work by Chesner et al. (12) showed bottom ash from the Southwest Brooklyn, New York, combustor is a

viable aggregate substitute. Performance was as good as the control at the 30 percent substitution level.

Chesner (13) examined economic, regulatory, and environmental concerns surrounding the use of bottom ash and has suggested that institutional issues may be the largest impediment to active utilization in the United States despite the fact that its use is technically and economically feasible.

## OBJECTIVES OF RESEARCH

The objectives of the project are threefold: to characterize the physical properties of the bottom ash over time, to obtain an optimum structural blend of bottom ash/asphalt for hot mix formulations, and to examine the environmental properties of bottom ash and bottom ash/asphalt blends under laboratory and field conditions.

Bottom ash collected during the first 7 months of a 2-year study has been evaluated for its physical and chemical properties. Hot mix designs were developed for a control mix and 25, 50, 75, and 100 percent bottom ash blends. Leaching properties of the bottom ash and bottom ash/asphalt blends were evaluated using batch and lysimeter leaching tests.

## MATERIALS AND METHODS

The bottom ash evaluated is produced in a 500 tons/day mass burn combustor located in Concord, New Hampshire. The facility is owned by Wheelabrator Concord L.P. and operated for the Concord Regional Solid Waste/Resource Recovery Cooperative. The facility has two process trains consisting of von Roll reciprocating stoker grates, Babcock and Wilcox boilers, and Wheelabrator Technology dry lime scrubber/fabric filters. The bottom ash from each train is quenched in its own quench tank.

A daily composite was collected on each of the 10 sampling days during the 7-month period. Economizer and fly ash streams were diverted from the bottom ash drag chain conveyor during sampling. Bottom ash grab samples were obtained randomly every 10 min by sampling from the drag chain to create 250-lb hourly composites. Four hourly composites were collected each test day. Combustor performance was also monitored to relate bottom ash quality to combustor operation.

The tests indicated in Table 1 were conducted at varying frequencies for the evaluation of the time-dependent physical and chemical properties of the bottom ash.

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TABLE 1 TESTS USED FOR BOTTOM ASH USE IN GRANULAR AND ASPHALTIC MATERIALS

Ash Tests	
Chemical	Physical
- Elemental Composition	- Moisture Content (ASTM D2216)
- Acid Neutralizing Capacity (ANC)	- Percent Rejected ( $> 3/4"$ )
- Total Availability Leach Test	- Organic Content/LOI (24)
- pH Dependent Leach Test	- Ferrous Content
- Monolith Leach Test	- Size Distribution (ASTM)
- Toxicity Characteristics	- Absorption and Specific Gravity (ASTM C127 and C128)
- Leaching Procedure	- Unit Weight and Voids (ASTM C29)
- Lysimeter Leach Test	- Moisture Density Test (ASTM D1557)
	- CBR (ASTM D1863)
	- Sodium Sulfate Soundness of Aggregates (ASTM C-88)
	- Los Angeles Abrasion (ASTM C131)
	- Unconfined Compressive Strength (ASTM D2166)
	- Marshall Stability (ASTM D1559)

## RESULTS AND DISCUSSION

### Time Dependence

The physical and chemical properties of bottom ash are of great importance in evaluating its potential use as a substitute for conventional aggregate in bituminous mixtures. The sampling program was designed to examine the hourly, daily, weekly, and monthly variability in physical and chemical characteristics of the bottom ash. Table 2 presents the statistical data of the ash physical testing accomplished over the first 7 months of the project. All average and standard deviation values were calculated on the basis of 40 hourly composite

samples obtained during the first 10 sampling events except where noted.

The amount of material passing  $3/4$  in. that could potentially be used as an aggregate is 67.1 percent on the average, with a standard deviation of 6.6 percent. Although it was not collected, a significant amount of aggregatelike material ranging from approximately 3 to  $3/4$  in. in size is also potentially available for use as aggregate.

The ash in general consists of equal amounts of glass and ferrous and nonferrous metals. The glass and nonferrous metals commonly melt and create conglomerate particles. The particles are therefore variable in texture and shape and, in the case of the glass, tend to be brittle. A minor amount of degradation of the brittle particles is to be expected during compaction.

The moisture content of the bottom ash is 36.7 percent on the average, with a standard deviation of 6.4 percent. Moisture contents of bottom ash from other facilities have been reported to range from 20 to 57 percent (14).

Figure 1 shows the bulk specific gravity as a function of test day. The data for each test day is the average of four hourly samples.

LOI is 7.1 percent on the average, with a standard deviation of 2.2 percent. The test consists of heating ash passing the No. 4 sieve to 600°C. The LOI increases as ash particle size decreases as shown in Figure 2.

Ferrous content, as defined by passing a magnet across a dispersed sample of dried,  $< 3/4$ -in. material, is 26.7 percent with a standard deviation of 5.6 percent.

The degradation of bottom ash, as measured by the Los Angeles abrasion test, meets the ASTM D692 maximum 50 percent requirement for use in bituminous paving mixtures. The degradation is high due to the severity of the test and the brittleness of the glass particles.

The sodium sulfate soundness test for coarse fraction of ash ( $\geq$  No. 4) meets the ASTM D692 specification ( $\leq$  12 percent). The fine fraction of ash ( $<$  No. 4) is less dense and not as strong. However, since the fine fraction of the ash comprises less than 50 percent of the bottom ash, the natural combination of the coarse and fine fractions passes the ASTM requirement.

The maximum modified proctor dry density is 109.4 lb/ft<sup>3</sup> on the average with a standard deviation of 1.29. Figure 3 shows the density and zero air void relationship for a typical bottom ash.

The gradation of any aggregate to be used in asphaltic concrete is important because of its direct effect on performance. Figure 4 shows the average gradation of bottom ash along with the upper and lower limits as required for a Type B NHDOT binder mix. All data are calculated on the basis of the 40 hourly samples. The bottom ash is well graded from the coarse to the fine sizes and meets the New Hampshire state gradation specifications for Type B base course.

### Aggregate Blends

The conventional aggregates used in this study were obtained from Pike Industries Inc., a materials supplier, asphaltic concrete producer, and contractor, as well as Concord Sand and

TABLE 2 ASH CHARACTERISTICS AND STATISTICAL VARIATION

	Average	Standard Deviation
Mass $< 3/4"$ (%)	67.1	6.6
Moisture Content (%)	36.7	6.4
Specific Gravities:		
Bulk fine	1.90	0.15
Bulk Coarse	2.25	0.11
Bulk SSD fine	2.15	0.11
Bulk SSD coarse	2.35	0.09
Apparent fine	2.51	0.16
Apparent coarse	2.51	0.08
Absorption fine (%)	12.6	2.9
Absorption coarse (%)	4.7	1.4
Loss on Ignition (%)	7.1	2.2
Ferrous Content (%)	26.7	5.6
Passing #4 sieve (%)	48.9	5.3
Passing #200 sieve (%)	4.1	1.1
LA Abrasion Grading B (%) <sup>a</sup>	47.3	1.3
LA Abrasion Grading C (%) <sup>a</sup>	43.4	1.1
Soundness $<$ no. 4 (%) <sup>a</sup>	13.5	1.2
Soundness $>$ no. 4 (%) <sup>a</sup>	2.6	0.2
Modified Proctor Density (pcf)		
Maximum	109.4	1.3
Density <sup>b</sup>	107.8	2.7

<sup>a</sup> average of two composites.

<sup>b</sup> average of seven composites compacted at 16% moisture.



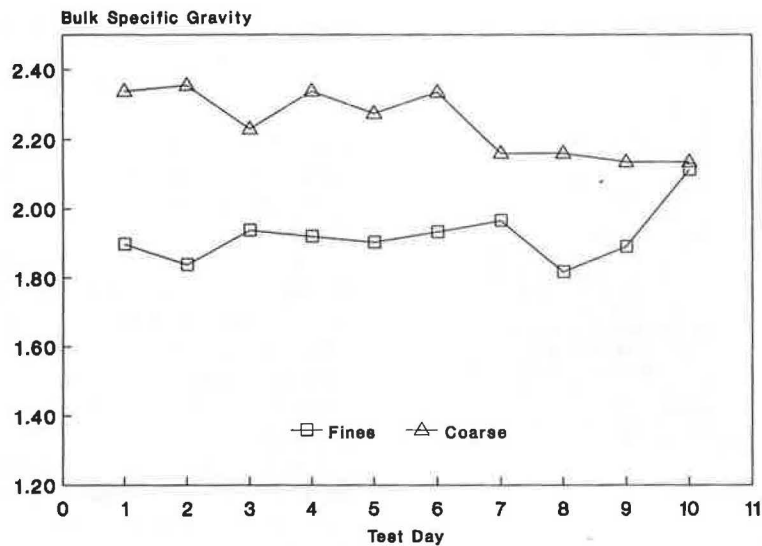


FIGURE 1 Bulk specific gravity versus test day.

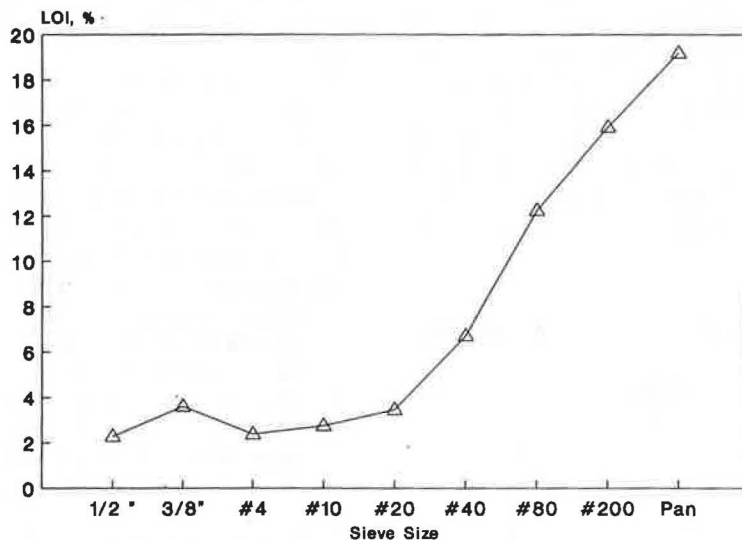


FIGURE 2 Loss on ignition versus sieve size.

Gravel, a materials supplier. The Pike aggregates used in this study ( $\frac{3}{4}$  in.,  $\frac{1}{2}$  in.,  $\frac{3}{8}$  in., manufactured dust, and manufactured washed dust) are manufactured crushed stone. The sand supplied by Concord Sand and Gravel is glacial in origin.

Five blends of the Pike aggregates and bottom ash were selected for asphaltic concrete testing. The control consisted of a unique combination of Pike aggregates that met the mid-point gradation requirements of a NH Type B mix. Bottom ash was substituted on a weight basis for the Pike aggregates. The blends evaluated were 0, 25, 50, 75, and 100 percent bottom ash.

Figure 5 shows the unit weight and void relationships of the blends as a function of ash content. The unit weight of the aggregate blends decreases from 119.8 lb/ft<sup>3</sup> for the control to 76.6 lb/ft<sup>3</sup> for 100 percent bottom ash, and the voids, by

absolute volume, increase from 26 to 39 percent as the bottom ash increases 0 to 100 percent.

Figure 6 shows the specific gravity of the aggregate blends and the effective porosity as a function of ash content. The specific gravities decrease with increased ash as would be expected. The fact that the apparent specific gravity changes very little with increasing ash content relative to the bulk specific gravity gives an indication that most of the voids in the ash are interconnected and continuous. The effective porosity indicates that the quantity of continuous pores is relatively high compared with conventional aggregate. The most significant effect of the unique pore system is to create an absorptive aggregate.

In that the ash has a much smaller specific gravity, and the blends were created by substituting the ash on a weight basis,

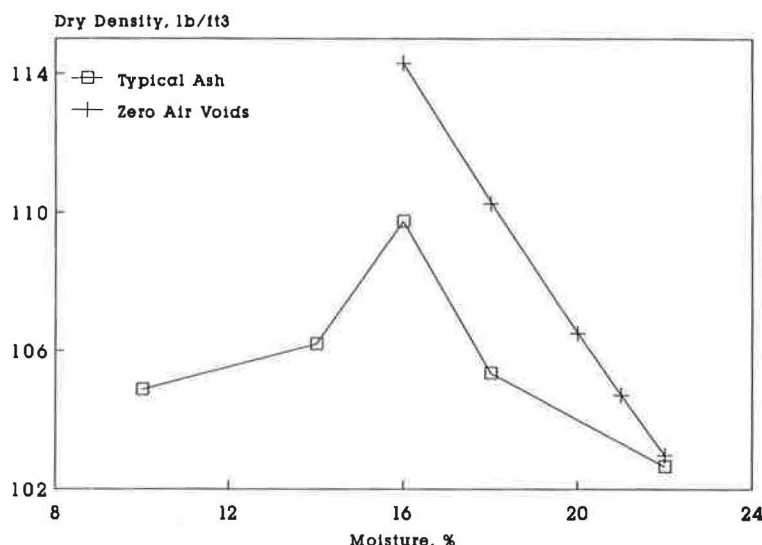


FIGURE 3 Density versus moisture content.

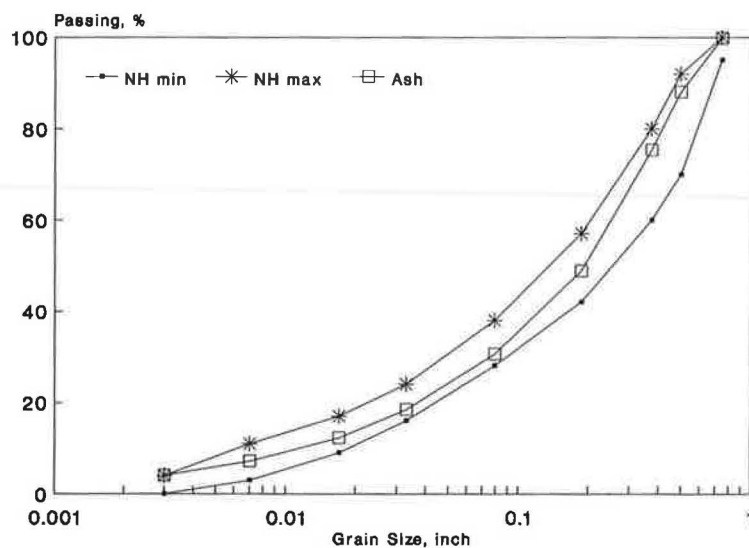


FIGURE 4 Average grain size distribution.

the mix volumes increase on a relative basis with increasing ash. All blends meet the requirements of NHDOT specifications, as shown in Figure 7.

#### Asphalt Concrete Mixes

The Marshall mix design method using a 50-blow compactive effort was used to develop the asphalt concrete mixes. AC-20 asphalt cement was employed. The Marshall mix design results are presented in Figures 8 through 13.

The effect of the low specific gravity of the ash is indicated by the lower unit weights, which range from 124.6 to 147.9 lb/ft³, as shown in Figure 8. This reduced weight to volume relationship for the ash mixes has also been noted by Collins et al. (9).

An average of three test specimens was used in the Marshall mix design analysis. Stability as shown in Figure 9 ranges from 1,840 to 2,903 lb and exceeds the New Hampshire specification. Depending on the asphalt content, the presence of the ash is capable of increasing the stability compared with the control. This could be due to the particle shape of the ash, which is a combination of angular and rounded particles.

Figure 10 shows flow as a function of asphalt content. The Asphalt Institute requires the flow to be in the 8 to 18 range for medium traffic. The ash mixes exhibit higher flow values than the control mix due to the increased asphalt contents. Flow varies in an inconsistent manner as asphalt content is varied for 75 and 100 percent ash mixes. Collins et al. (15) have reported that Marshall flow values for mixtures using ash residue are sometimes erratic due to variation of the ash properties.

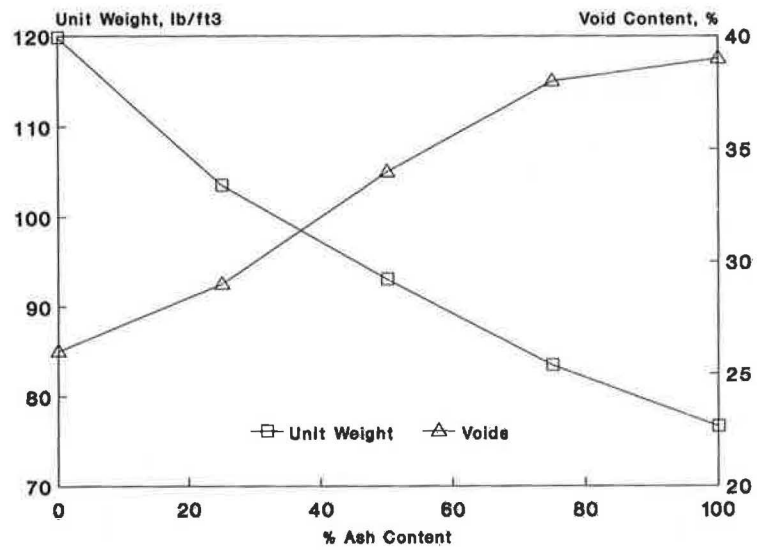


FIGURE 5 Unit weight and void content versus ash content.

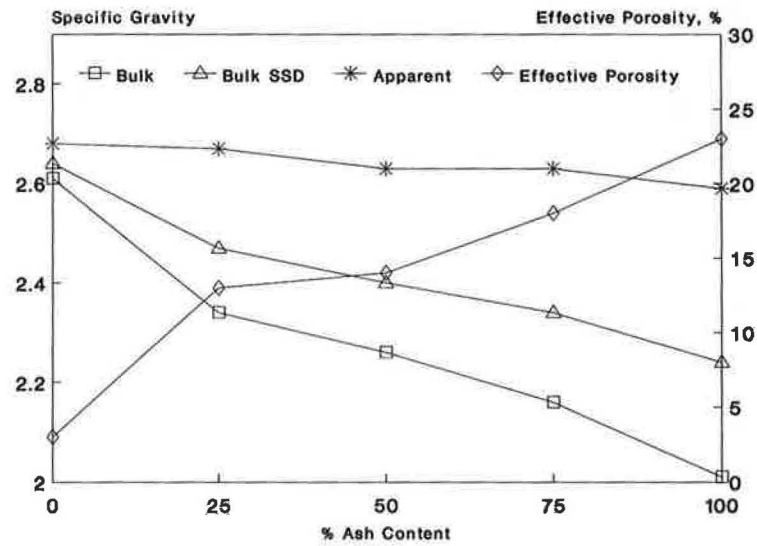


FIGURE 6 Specific gravity and effective porosity versus ash content.

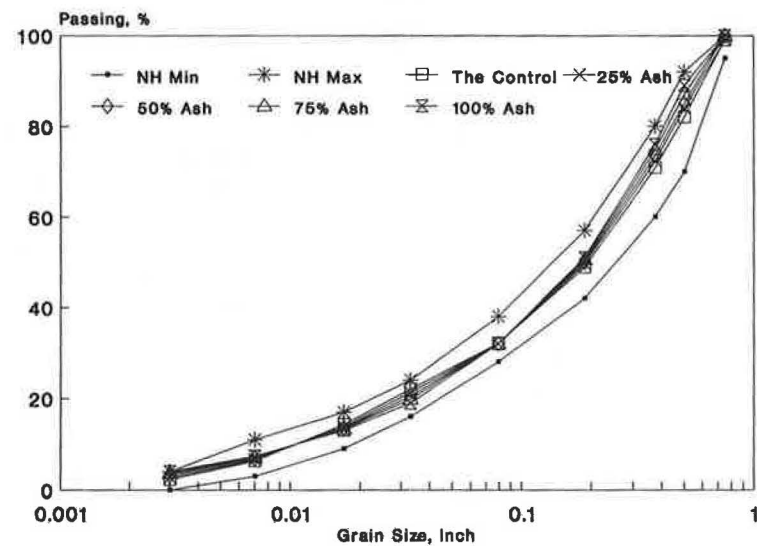


FIGURE 7 Master gradation curves of test blends.

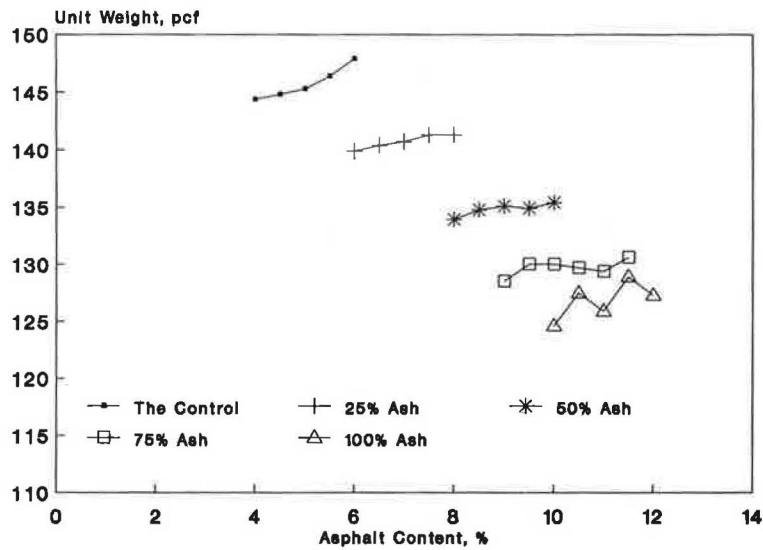


FIGURE 8 Unit weight versus asphalt content.

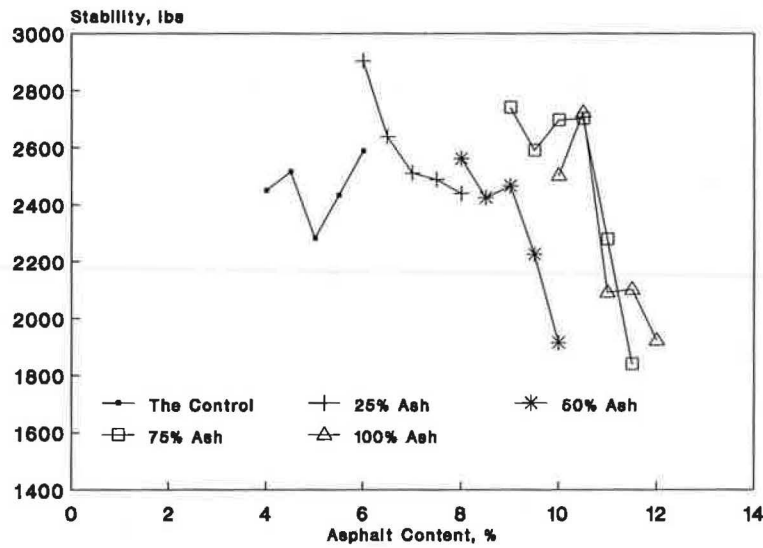


FIGURE 9 Stability versus asphalt content.

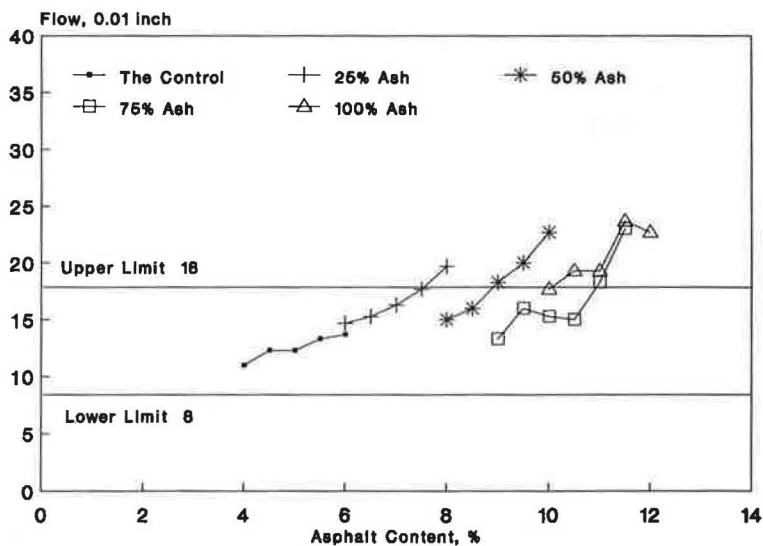


FIGURE 10 Flow versus asphalt content.

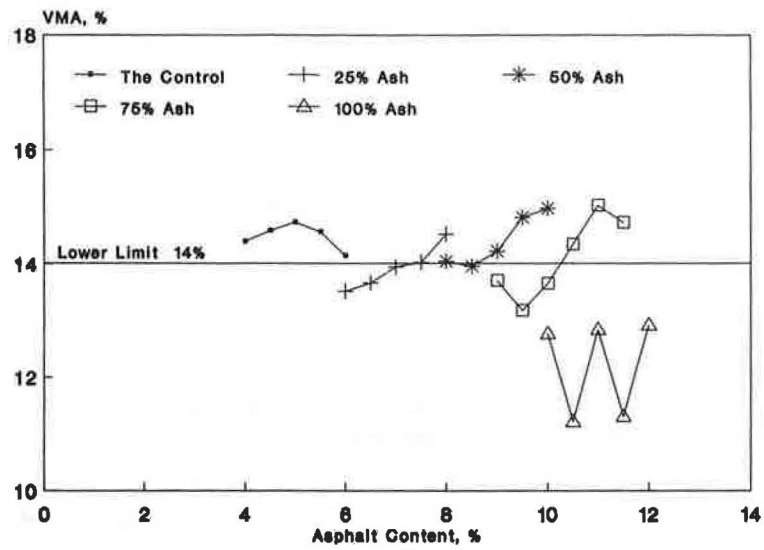


FIGURE 11 VMA versus asphalt content.

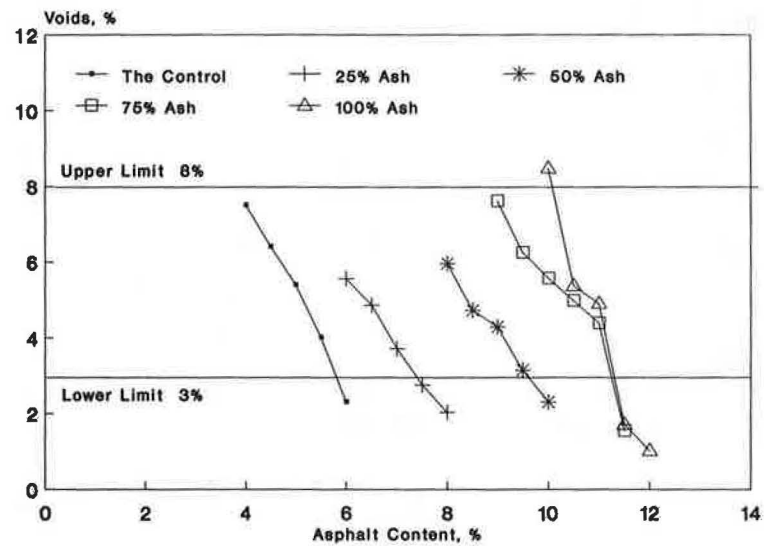


FIGURE 12 Total voids versus asphalt content.

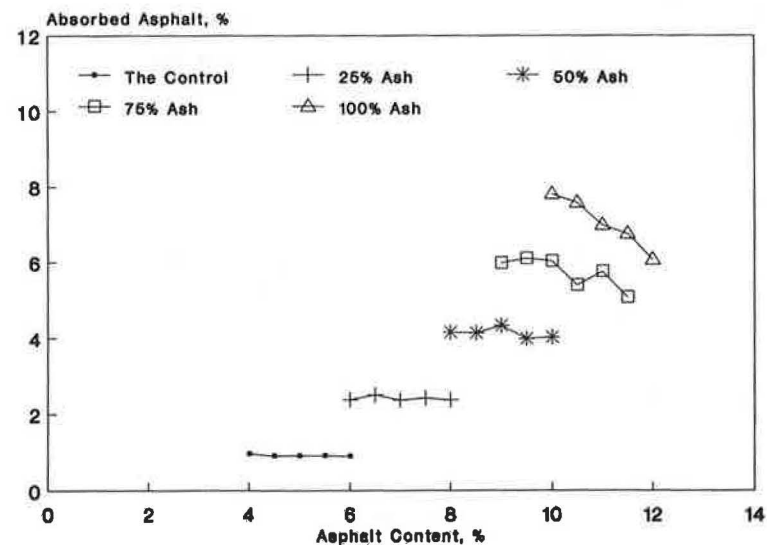


FIGURE 13 Absorbed asphalt versus asphalt content.

The voids in the mineral aggregate (VMA) relationship of the ash mixes is shown in Figure 11. The VMA significantly affects the performance of a mix because if it is too low, the mix may suffer durability problems, and if it is too high the mix may show stability and bleeding problems. The Asphalt Institute requires a minimum VMA of 14 percent for a nominal maximum particle size of  $\frac{3}{4}$  in. All mixes containing up to 75 percent ash pass the VMA requirement of a minimum of 14 percent.

Figure 12 shows the void-asphalt content relationship of the Marshall mixes. The void content is based on the maximum theoretical specific gravity of the paving mix as determined by the Rice method. The void content criteria suggested by the Asphalt Institute for a base subject to medium traffic is between 3 and 8 percent. The asphalt content needed to meet the specification increases as the ash content increases. The reason that the ash mixes demand more asphalt is that the absorption of the ash is higher than the conventional aggregate. Some asphalt is absorbed into the ash particles, particularly the finer particles, increasing the amount of asphalt needed to attain complete coating of the aggregate particles in the mix.

Satisfying the Asphalt Institute design criteria as shown in the figures requires 5, 7.4, 8.6 and 10.5 percent asphalt cement for the 0, 25, 50 and 75 percent ash mixes. The 100 percent ash mixes do not satisfy the design criteria at any asphalt content.

The asphalt absorbed into the aggregate pores is shown in Figure 13. In concept, the amount of asphalt absorbed into the aggregate is not a function of asphalt content, as shown by the blends up to 50 percent ash. For reasons unknown at this time, the higher ash blends show a decrease in absorbed asphalt with increasing asphalt cement content. The effective asphalt, defined as the total asphalt content minus the absorbed portion, increases with increased asphalt content for both the ash mixes and the aggregate control. An extensive field test is planned to evaluate the long-term performance of the bottom ash mixes.

## ENVIRONMENTAL TESTING

A number of methods were employed to evaluate the environmental properties of the bituminous blends. Lysimeters were used to evaluate leachate characteristics of a bottom ash pavement mix that was excavated and landfilled. A control lysimeter with bottom ash was used for comparative purposes. Batch laboratory leach tests were used to evaluate the effects of percent asphalt cement and percent bottom ash substitution on the encapsulating properties of bitumen, and a standard Environmental Protection Agency Toxicity Characteristic Leaching Procedure (EPA TCLP) test was run on the bottom ash.

### Lysimeter Test Procedures

Approximately 30 yd<sup>3</sup> of bottom ash was collected at the waste-to-energy facility and processed to pass the  $\frac{3}{4}$ -in. sieve. Approximately 4 tons of this material was placed directly in a 20-yd<sup>3</sup> double-lined roll-off container. The remaining processed bottom ash was batched at a local hot mix plant (Pike

Industries) to produce 12 tons of  $\frac{3}{4}$ -in. binder with 25 percent bottom ash (75 percent natural aggregate) at an asphalt cement content of 9 percent. The mix was paved, compacted, and broken up after a 7-day period with a backhoe into large pieces typical of what might be expected to be dumped into a landfill as construction debris. The broken up pavement, ranging in size from small palm size to large 2- by 3-ft plates, was put in a second double-lined roll-off container. These lysimeters generate time-dependent data on the leachate properties of the control bottom ash and the 25 percent ash asphalt pavement mix. The leachate, originating from natural precipitation, was collected and analyzed for analytes.

### Regulatory Test Procedure

TCLP regulatory leach test was performed on composite bottom ash samples from material used in the control lysimeter. A number of additional inorganic analytes besides the specified regulatory analytes were analyzed to help develop a bottom ash leaching data base.

### Results

The results of the lysimeter data for the first two collection events are given in Table 3. The cumulative liquid/solid ratio for the first event was 0.069 and 0.073 and for the second event was 0.174 and 0.184 for the ash and ash pavement lysimeters, respectively. Bottom ash releases significantly lower concentrations of contaminants compared with combined ashes because it does not contain the highly soluble Ca(OH)<sub>2</sub> added

TABLE 3 LYSIMETER LEACHATE DATA

	Bottom Ash		Bottom Ash Asphalt	
	First Event	Second Event	First Event	Second Event
L/S <sup>a</sup>	0.069	0.174	0.073	0.184
pH	6.41	6.75	7.15	7.32
COD <sup>b</sup>	310	400	22	15
NO <sub>3</sub> <sup>-</sup> -N	<0.50 <sup>c</sup>	<0.25	<0.050	<0.050
NO <sub>2</sub> <sup>-</sup> -N	<0.50	<0.50	<0.050	<0.25
NH <sub>4</sub> <sup>+</sup> -N	5.0	16	0.20	0.44
SO <sub>4</sub> <sup>2-</sup>	1,700	1,600	31	68
Br <sup>-</sup>	30	28	<1.0	2.1
PO <sub>4</sub> <sup>3-</sup>	<0.01	0.041	<0.01	<0.010
Cl <sup>-</sup>	1,700	1,800	24	30
Al	0.20	0.16	<0.10	<0.10
Ba	<0.10	<0.10	<0.10	<0.10
Be	<0.0050	<0.50	<0.0050	<0.50
Cd	<0.0050	0.0007	<0.0050	0.0014
Ca	590	870	22	45
Cr	<0.010	<0.010	<0.010	<0.010
Cu	0.59	0.15	<0.020	0.024
Fe	0.050	7.3	<0.030	0.038
Pb	0.0053	0.028	<0.0050	<0.005
Mg	6.5	6.9	1.3	2.8
Mn	0.32	2.3	0.020	0.063
Hg	0.0006	0.0005	<0.0003	<0.0003
Mo	0.34	0.27	<0.10	<0.10
K	220	190	3.9	4.5
Si	2.2	1.4	1.2	2.7
Sr	4.4	4.3	0.15	0.21
Zn	0.068	0.031	<0.020	0.031

<sup>a</sup>L/S liquid to solid ratio weight of leachate to wet weight of ash

<sup>b</sup>Concentrations are expressed in mg/L

<sup>c</sup>Less than sign shows that the concentration was below the indicated detection limit.

TABLE 4 BOTTOM ASH TCLP DATA

Composite	1	2	3	Average	Regulated Limit
	Concentration, mg/L				
Al	3.9	1.7	1.1	2.2	5
As	<0.2	<0.2	<0.2	<0.2	5.0
Ba	0.38	0.38	0.35	0.37	100.0
B	0.53	0.52	0.55	0.53	-
Cd	0.028	0.032	0.031	0.030	1.0
Ca	760	850	830	813	-
Cr	<0.01	<0.01	<0.01	<0.01	5.0
Cu	0.9	1.4	1.5	1.3	-
Fe	12	0.94	0.21	4.38	-
Pb	1.7	1.9	1.3	1.6	5.0
Mg	29	32	31	31	-
Mn	3.1	2.9	2.4	2.8	-
Hg	<0.0003	<0.0003	<0.0003	<0.0003	0.2
Mo	<0.10	<0.10	0.11	0.04	-
K	18	20	18	19	-
Se	<0.02	<0.02	<0.02	<0.02	1.0
Si	17	16	19	17	-
Ag	<0.02	<0.02	<0.02	<0.02	5.0
Sr	1.6	1.8	1.6	1.7	-
Zn	21	29	23	24	-

\* - Not regulated

for air pollution control. The leachate from bottom ash contains relatively low concentrations of sulfate and chloride, and heavy metal release is nonproblematic. The encapsulating properties of bitumen in the ash pavement are readily apparent. Generally, there is more than a 10- to 20-fold decrease in salt leachability from bitumen encapsulated material as compared with unamended bottom ash.

The EPA TCLP test data are given in Table 4. These data indicate that the bottom ash easily passes the standard regulated limits. Results from these as well as other leach tests are being compiled to create an extensive leaching data base.

## CONCLUSIONS

The following conclusions can be drawn from the work completed on this research project to date.

1. The variation of the physical properties of the ash produced during the 7-month test period is reasonably insignificant.
2. The physical properties of the ash are consistent with its use as a substitute for conventional aggregate in asphaltic concrete.
3. The properties of ash are typical of a lightweight aggregate.
4. Bottom ash mixes can be designed to meet NHDOT specifications with up to 75 percent ash substitution. The mixes have lower unit weights (increased volume with constant weight) and require more asphalt cement than conventional mixes.
5. Bitumen effectively encapsulates the bottom ash and significantly mitigates the release of low concentrations of soluble salts from the bottom ash.

A field pavement demonstration is planned after ongoing laboratory physical and field lysimeter work is completed.

## ACKNOWLEDGMENTS

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# Use of Cement Kiln Dust, Fly Ash, and Recycling Technique in Low-Volume Road Rehabilitation

CHIANG LIN AND XISHUN ZHANG

A research project on the evaluation of methods of rural road rehabilitation has been conducted at the Southern Illinois University at Edwardsville with the cooperation of the Illinois Department of Transportation since late 1987. The use of cement kiln dust, fly ash, and a recycling technique to rebuild the roadway base is one of the methods under investigation. A series of laboratory studies was conducted to develop the mix design for field demonstration. A  $\frac{1}{3}$ -mi test road was constructed to evaluate the adequacy of the mix design and the practicality of the construction procedures. The performance of the test road to date is satisfactory. No damage has been detected. It is concluded that cement kiln dust and fly ash could be used successfully for roadway base stabilization, and it is feasible to recycle the existing chip seal surface in the field.

Low-volume roads (LVR) for many years have contributed significantly to the growth of this country. At present, there are an estimated 3 million mi (1,2) of public LVR in the United States. In Illinois, a significant portion of LVR is controlled by local officials. Because of financial constraints, the majority of these LVR have been inadequately maintained. Most local governments rely on simple but ineffective resurfacing methods for maintenance. These methods only provide a temporary riding surface without solving the underlying problem, which is the improper design of the roadbed support system. It is necessary to have a rehabilitation method that can provide structural stability to LVR with low cost and simple construction procedures.

The main objective of this research is to investigate the feasibility of using two low-cost materials—cement kiln dust (CKD) and fly ash (FA)—and a recycling technique to rehabilitate the base of LVR. CKD and FA are by-products collected from bag houses in cement manufacturing plants and coal-fired power plants, respectively. These materials have been successfully used as pozzolanic stabilizers in the construction of highway bases and subbases (3,4). The reclaimed material in the recycling process is the deteriorated chip seal surface material. An extensive laboratory study followed by a field test section study was conducted between 1987 and 1989 to determine the relative applicability and resultant beneficial effects of the proposed method. The conventional chip

seal resurfacing method was also included in the study for comparison. The important data and information collected from the study are presented in this paper.

## MATERIALS CHARACTERIZATION

### Reclaimed Surface Material

A  $\frac{1}{3}$ -mi section of a deteriorated chip seal road in Pin Oak Township, Illinois, was chosen as the field test site for this project. A portion of this section was scarified initially to provide the reclaimed surface material (RSM) for laboratory study. The collected RSM was a mixture of soil, rock chips, and hardened oil/chip lumps. On the basis of the results of soil classification tests, the RSM is classified as A-1-a in the AASHTO system.

The sieve analysis was performed on five representative samples collected at different locations. Table 1 gives the test results and gradation requirements for lime-fly ash-aggregate (LFA) mixture as recommended by the Illinois Department of Transportation (IDOT) (5) and the gradation requirement for kiln dust-fly ash-aggregate (KFA) mixture as recommended by N-viro Energy Company (N-viro) (6). Figure 1 shows the gradation curve for the five collected samples. RSM meets both IDOT and N-viro gradation requirements.

There was concern that the hardened oil lumps in RSM might contain a detrimental amount of active sulfur. The sulfur could react with water and result in an acidic reaction, which could adversely affect the pozzolanic and hydration reaction. By following the ASTM Method D2976 (standard test method for pH of peat materials) (7), the pH value was found to be close to 7. Therefore, it is concluded that RSM would not have any negative effect on pozzolanic or hydration reaction.

### New Limestone Aggregate

There were two reasons for adding new limestone aggregate (NLA) to the mixture: (a) to increase the volume so that the base of the test sections could be elevated to improve drainage conditions, and (b) to improve the gradation matrix in the mixes to gain a better strength. ASTM CA-7 size aggregate was selected on the basis of its particle gradation and its being widely used in LVR.

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TABLE 1 GRADATION OF RSM AND REQUIREMENTS

Sieve Size	% Passing		
	RSM	IDOT	N-VIRO
1-1/2"	100	100	100
1"	92-100	90-100	90-100
1/2"	76-97	60-100	60-100
No.4	42-53	40-70	40-75
No.8	29-39	— *	30-60
No.40	14-22	0 -25	0 -30
No.200	9 -16	0 -10 (gravel) 0 -15 (crushed stone)	0 -12

\* Unavailable Data.

### CKD

The CKD used in this project was collected from the Dundee Cement Company in Clarksville, Missouri. The specific gravity of the CKD per ASTM C188 is 2.49. Table 2 presents the physical and chemical testing data of this CKD and the quality requirements specified by IDOT and N-viro. The CKD meets all the requirements.

### FA

The FA used in the project was purchased from the American Fly Ash Company in Chicago and is classified as ASTM Class F, which is noncementitious. Table 3 presents the physical and chemical testing results as well as the ASTM C618 specifications for Class F and Class C. The FA meets the ASTM C618 standard.

### Portland Cement

The intent of adding a small amount (less than 5 percent of total weight) of portland cement was to provide an early ce-

TABLE 2 CKD PROPERTIES AND REQUIREMENTS

Gradation % Passing	Composition*			CKD	IDOT/N-Viro Requirement
#4 100%	CaO	39.9	total reactive	12.7%	Min=10%
#30 100%	MgO	2.2	alkalies %		
#100 92.7%	Na <sub>2</sub> O	0.2	%CaO+%MgO-1.2(%LOI)		
	K <sub>2</sub> O	5.1	total reactive	5.3%	Max=6%
	SO <sub>3</sub>	5.8	alkalis		
	Insoluble	12.1	%K <sub>2</sub> O+%Na <sub>2</sub> O		
	Rapid Sugar	5.6	amount of SO <sub>3</sub>	5.8%	Max=10%
	LOI	24.6			

\* By Weight

mentitious effect to mixes that would increase the early strength. This was essential to expedite the construction process. Type I, general purpose cement, was used.

### LABORATORY EVALUATION

The laboratory tests were conducted between 1987 and 1988 to determine the final mix design for the CKD/FA/cement/NLA/RSM mixture. The process was divided into two stages: preliminary and final selections. Two main ASTM testing methods were used throughout the evaluation. The moisture-density test was conducted according to ASTM D1557 Method C with the exception that the compaction effort of three layers and 25 blows per layer was used. This compaction effort complies with IDOT specification for lime-fly ash-aggregate mixes. Standard Proctor molds with a 4-in. diameter and a 4.6-in. height were used. The optimum moisture contents (OMC) found in these tests were used to prepare specimens for strength tests. The reason for using OMC is that stabilized mixes at OMC would probably yield the highest strength. The unconfined compression test was conducted according to ASTM C593. The specimens, made by following the same procedure as used in the moisture-density test, were sealed in plastic

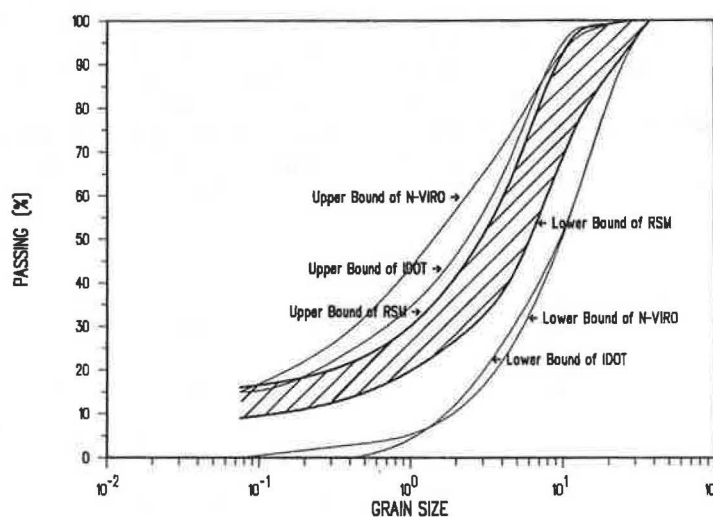


FIGURE 1 RSM grain size distribution.

TABLE 3 FLY ASH PROPERTIES AND REQUIREMENTS

Analysis		Requirements for ASTM C618 Class F & C
Pozzolantic Activity with Portland Cement, Strength at 28 Days, % of Control	90	75 min.
Water Requirement, % of Control	96	105 max.
Autoclave Expansion, %	0.5	0.8 max.
No. 325 Sieve Residue, %	17.3	34 max.
Variation, Percentage Points from Average	1.7	5 max.
Specific Gravity	2.5	
Variation from Average, %	0.8	5 max.
SiO <sub>2</sub> /Al <sub>2</sub> O <sub>3</sub> /Fe <sub>2</sub> O <sub>3</sub> , %	53.4/18.0/15.5	
Total, %	86.9	50 min.
SO <sub>3</sub> , %	1.4	5.0 max.
Moisture, %	0.3	3.0 max.
Loss on Ignition, %	0.7	6.0 max.
CaO, %	4.0	
MgO, %	0.8	
Na <sub>2</sub> O, %	1.3	
K <sub>2</sub> O, %	2.4	
Available Alkali as Na <sub>2</sub> O, %	1.3	1.5 max.

bags and containers to prevent moisture loss under the various curing conditions.

### Preliminary Selection

In this initial stage, a feasibility study was conducted to evaluate whether it would be possible, by proper proportioning of CKD, FA, and RSM, to develop a mix with strength exceeding the minimum strength requirement, which is 400 psi for the specimens cured at 100°F for 7 days (this standard is for lime-fly ash new aggregate mixes) as specified by ASTM C593. To account for normal curing condition in the field as well as evaluating the ability of early strength gain, the same tests were conducted for specimens cured under 70°F for 7 days. Table 4 presents the test results. The reported strength data for each mix is the average of three specimens. Figures 2 and 3 show the relationship between compressive strength and percent additives of CKD/FA for 7 days, 100°F curing and 7 days, 70°F curing, respectively.

The mixes cured at 100°F yielded much higher strength than those cured at 70°F, which indicates that the pozzolanic reaction between CKD and FA is strongly temperature dependent. The mixes with a higher percent of CKD and FA have higher strength. This indicates that the CKD to FA ratio of 3 to 1 could be the optimum mixing ratio for strength gain. Seven mixes (Mix 9 through Mix 15) yield strengths that exceed the ASTM C593 minimum requirement. Three mixes (Mixes 12, 14, and 15) cured at 70°F have strengths of more than 300 psi. On the basis of strength data, Mixes 14 and 15 are the best designs. However, the two mixes have as much as 40 percent additives of CKD/FA by weight. The disadvantage of using such a high level of CKD and FA is that it might

be difficult to transport and mix in the field. Therefore, it was decided to reduce the amount of CKD and FA to make the mix design practical for field application while still satisfying the engineering requirements. Final selection was based on this consideration.

### Final Selection

ASTM Standard CA-7 limestone was added in various amounts at this stage to further improve the mix designs. Tests similar to those in the initial stage were conducted. Table 5 presents the test results. In Table 5, the material proportioning was based on the total weight of materials. On the basis of the preliminary test results, a CKD to FA ratio of 3 to 1 was used through all mixes. The amount of total fine additives (CKD + FA + cement) used ranged from 20 to 30 percent by weight. All reported strength data for each mix are an average of three specimens. Compared with Table 4, for the same percent of additives of (CKD + FA) the addition of CA-7 aggregate greatly improved the strength of the mixes. Apparently, the addition of CA-7 aggregate improved the aggregate matrix of the mixes and thus increased their stiffness. Mix E, which had 15 percent CA-7 aggregate and 25 percent of total CKD and FA, had the highest strength. Therefore, it was decided to include 15 percent of CA-7 aggregate in the final mix design. Two percent of cement (by total weight) was added to Mixes H and I to improve their early strength gain. The strength values of Mixes H and I are 2.71 (1,083/400) and 2.75 (1,101/400) times the ASTM C593 standard and are 1.35 (875/650) and 1.82 (1,181/650) times the IDOT standard (650 psi cured at 70°F for 14 days). On the basis of concerns

TABLE 4 PRELIMINARY TEST RESULTS

Mix No.	(CKD+FA): RSM <sup>a</sup>	CKD:FA	Moisture Content, % <sup>b</sup>	Dry Density (lb/ft <sup>3</sup> )	7 Day Strength (psi)	
					70°F	100°F
0	0:100	----	5.8	129.8	---	---
1	15:85	1:1	6.6	124.5	170	210
2	15:85	2:1	6.3	125.4	210	320
3	15:85	3:1	7.0	124.8	150	230
4	20:80	1:1	6.2	126.4	180	310
5	20:80	2:1	6.9	126.7	170	270
6	20:80	3:1	7.3	123.6	170	330
7	25:75	1:1	7.6	124.8	180	400
8	25:75	2:1	6.6	125.7	250	330
9	25:75	3:1	7.6	125.1	240	420
10	35:65	1:1	7.4	125.8	150	470
11	35:65	2:1	8.0	125.5	270	420
12	35:65	3:1	7.6	123.6	320	490
13	40:60	1:1	7.5	125.7	210	440
14	40:60	2:1	8.5	123.6	330	590
15	40:60	3:1	7.5	126.0	370	660

<sup>a</sup> By Total Weight of Each Mix.

<sup>b</sup> Optimum Moisture Content.

— Unavailable Data.

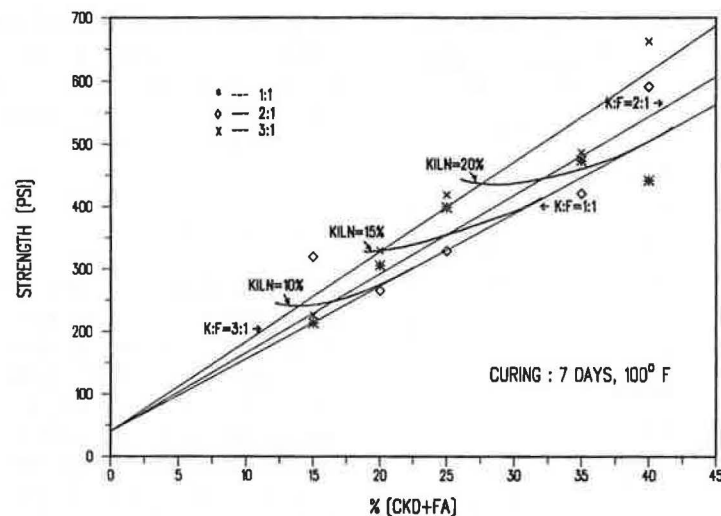


FIGURE 2 Percent (CKD + FA)-strength relation (curing at 100°F).

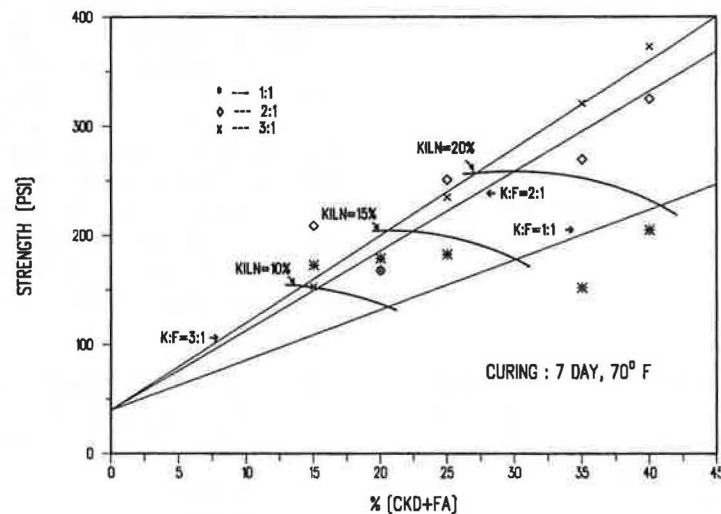


FIGURE 3 Percent (CKD + FA)-strength relation (curing at 70°F).

TABLE 5 STRENGTH DATA FOR FINAL SELECTION

Mix ID	CKD (%)	FA (%)	Cement (%)	RSM (%)	CA-7 (%)	Strength (psi)		
						70°F		100°F
						7 days	14 days	7 days
A	15	5	---	72	8	440	---	710
B	15	5	---	64	16	360	---	630
C	15	5	---	56	24	410	---	640
D	18.8	6.3	---	67.5	7.5	400	---	650
E	18.8	6.3	---	60	15	540	---	830
F	18.8	6.3	---	52.5	22.5	450	---	610
G	22.5	7.5	---	63	7	510	---	740
H	17.3	5.8	2	75	---	---	880	1080
I	17.3	5.8	2	60	15	---	1180	1100

Note: Material proportion is presented by weight.  
 "—" Unavailable Data

about variability in the field and the additional material required to elevate the roadway base for improving drainage, Mix I was selected as the final mix design for the construction of the test section.

### FIELD EVALUATION

With the cooperation of Pin Oak Township, an Illinois township located east of the Southern Illinois University at Edwardsville (SIUE) campus, a section of Goshen Road was chosen as the test site. At the time of construction the traffic volume of Goshen Road was 500 ADT (average daily traffic).

### Properties of Subgrade Soil

Laboratory tests were conducted to determine the properties of the soil samples obtained from the field. The subgrade soil

was classified as A-6(13) in the AASHTO classification system and low plasticity clay (CL) in the unified soil classification system. The laboratory maximum density at the OMC (16.5 percent) was 109 pcf. The California bearing ratio (CBR) tests (ASTM D1883) were conducted to determine the soil support value. The CBR numbers were 6.8 and 4.6 for unsoaked and soaked conditions, respectively. An average value of 5.7 was used to determine the design thickness of the base layer.

### Thickness Design of CKD/FA Section

The flexible pavement design procedure of LVR in the 1986 revised AASHTO design guide (8) was followed to determine the required thickness of the surface-treated CKD/FA layer. In the design process, the CKD/FA layer was treated as a stabilized base course. The LVR design method uses the same design approach as the highway pavement design. On the basis of a predicted future traffic volume of 1,000 ADT, a final thickness of 8 in. was determined for the CKD/FA layer, which also satisfies the minimum thickness of 4 in. as required by the AASHTO design guide.

### Construction of CKD/FA Section

The construction of the CKD/FA section was completed within 4 days (August 15 through 18, 1988). The test section remained closed for another week to allow curing. Only Pin Oak Township personnel and equipment were employed to investigate whether the construction could be carried out using only the resources of a local government. The construction equipment included two motor graders, one rubber-tired compactor, one steel drum vibratory roller, one rotavator tiller, and two water trucks.

### Scarification and Pulverization of the Surface Material

The motor graders were used to scarify the existing chip seal surface. A scarified thickness between 4 and 6 in. was reached at the end of the process. In that total thickness, the built-up chip seal layer was about 1.0 to 1.5 in. The rotavator tiller was used to pulverize the scarified material. It required six passes to obtain the proper size (approximately 1 in. maximum) of RSM.

### Preparation of the Subgrade

After pushing RSM to the outside of the roadway, water trucks, motor graders, and compactors were used to shape and compact the subgrade. A crowned surface was created for better drainage. A nearly 100 percent field to laboratory density ratio was achieved.

### Addition of CA-7 Aggregate

The CA-7 aggregate was delivered in conventional, open-bed dump trucks and was spread along the roadway by a tailgating

technique. Each truckload was spread over an equal distance to deposit a uniform layer of aggregate. The CA-7 aggregate was thoroughly mixed with RSM by use of motor graders and a tiller.

### Addition of CKD, FA, and Cement and Mixing and Compacting

The mixed coarse materials (CA-7 and RSM) were wetted before the delivery of the fine material. The fine material, which consisted of CKD, FA, and portland cement, were dry-blended at a central mix plant and then hauled to the test site in conventional open-bed dump trucks covered with canvas to contain the dust.

The construction process proceeded on a subsection by subsection basis to prevent moisture loss during mixing and compaction. The fine material was tailgated along the roadway between two windrows of the coarse material. Water trucks were used to provide the needed moisture for compaction. The graders and a tiller were used for redistributing and mixing the fine and coarse materials. Compaction was performed with both the rubber tire and steel drum rollers. Both the moisture content and dry density were monitored during the compaction process. Averages of 7.3 percent moisture content and 89.3 percent compaction ratio were measured. The entire process of adding the fines, mixing, and compaction was completed in 2 days.

### Curing Process

Proper curing is essential for pozzolanic materials and cement to attain their strength. At the end of each of the last 2 construction days, the compacted road surface was sealed with a prime coat to prevent moisture loss in the pozzolanic layer. The road remained closed 1 week for curing. During this time, the pavement temperature was monitored. It varied from 89°F to 94°F.

### Application of Chip Seal Coating

In this final step, the compacted surface was sealed with oil and chip to provide a riding surface. Because of the low porosity of the CKD/FA base, only 0.3 gal/yd<sup>2</sup> application rate of SC-800 cutback asphalt was sprayed compared with 0.5 gal/yd<sup>2</sup> for conventional surfaces. Limestone chip (3/8-in. maximum size) was spread on top of the asphalt to complete the surface treatment.

### Evaluation of Construction and Recommendation

On the basis of experience from the construction, a number of problems regarding the construction procedures and use of equipment that influenced the quality of the pavement were identified. The improvements of these problems could ensure a better CKD/FA base to be constructed in the future.



### Material Proportioning and Dusting

A comparison of the designed material proportioning and the actual material proportioning is presented in Table 6. As indicated in Table 6, significant differences exist between the quantities used in the construction and the initial design quantities. It appears that more than the initial designed amounts of fines and CA-7 aggregate were used. Two reasons contributed to these deviations. One was improper control at the central mixing plant for proportioning the fine materials. The other was that an insufficient amount of RSM was left for mixing after the preparation of the subgrade. The insufficiency resulted when a major portion of the scarified materials was incorporated into building the crowned surface during subgrade preparation. The shortage of RSM increased the percentages of the fines and CA-7 aggregate. It also reduced the overall thickness and compacted density of the completed CKD/FA pozzolanic section. Both of these problems can easily be corrected by exercising more caution and inspection during construction.

The dusting problem was created during the dumping and spreading of the dry fine material. One way to reduce the severity of this problem is to premix and prewet the fines in the central mixing plant before delivery. However, this method might produce premature chemical reactions during the transportation process that would reduce the overall strength. To lower the potential of premature reactions, the amount of water added in the plant should be kept as low as possible. A laboratory study was conducted to determine the maximum allowable delay between wetting and compaction that would not result in significant loss of strength. The results of this study are presented in the section on the postlaboratory study.

### Evaluation Based on Field Core Samples

The core sampling method for cement concrete material was conducted 30 days after construction to assess the quality of the mixture. Core samples with a 4 in. diameter were extracted from 23 locations along the test section. Some fragmentation occurred during the drilling process because of the relatively low strength (compared with cement concrete) of the mixture.

The core drilling results can be divided into three distinct groups on the basis of their appearance. They were the cores with (a) uniform and hardened mixtures, (b) excessive coarse material, and (c) excessive fines. The last two groups apparently resulted from nonuniform distribution of the fine and

coarse materials as well as improper mixing at those locations. Because of the fragmentation of the samples, only three samples could be used for the unconfined compression test. Their strength values are 410, 622, and 779 psi, respectively. The large variation is typical for the field mixing technique used in this project. As indicated by the field coring data, the average thickness of the pozzolanic layer is about 6 in., compared with the original design thickness of 8 in. Not having sufficient reclaimed surface material for mixing is the main reason for this difference.

### Material Distribution

The problems of nonuniform distribution of the materials were also observed during the field operation. Using the equipment typically owned by local governments to deposit and redistribute the fines was not effective for handling such a large amount of materials. A specially designed mechanical spreader used in soil-cement construction needs to be used in this type of operation (9). A manual check on the accuracy of the spread for both reclaimed and fine materials should be conducted to ensure that the proper quantity of materials is being provided.

### Water Application and Proper Mixing

The moisture level has a significant effect on the mixture compaction. However, the dry, hot summer weather plus the time span between the mixing and compaction made the retaining of sufficient moisture in the mixture difficult. After the compaction, several locations were found to have inadequate moisture distribution. The construction equipment used had some difficulty in handling the mixing work for such a large amount of added fines. One way to reduce the difficulty of mixing is to use a more reactive CKD, which would reduce the overall quantity of added fines. Another way would be to use a specially designed recycler or flat-transverse-shaft mixer to perform the pulverization, water spray, and mixing in a single operation. This equipment would reduce the construction time and shorten the delay between mixing and compaction.

### Field Equipment Operating Sequence

The efficiency of field operations is one of the key factors that affect the quality of construction. Carefully planning the operational procedures and properly directing the working order of each piece of construction equipment are necessary to ensure a successful and economical operation.

Several times after coming back from a reloading trip, the water trucks were blocked by other equipment. This prevented their reaching some of the drying areas. Also, it was found that the mixing distance had a profound effect on the quality of mixing. The field operation procedures recommended by the Portland Cement Association for soil-cement construction (9) can be used as an operational guide for this

TABLE 6 MATERIAL PROPERTIES (PERCENT BY WEIGHT)

Materials	Material Used (ton)	Designed (% by wt.)	Actual (% by wt.)
Fines: CKD	191	17.3	19.3
Fly Ash	115	5.8	11.6
Cement	50	2.0	5.1
Total Fines	356	25.0	36.1
CA-7 Aggregate	221	15.0	22.4
Reclaimed Surface Material	412	60.0	41.6

type of construction. Figure 4 shows a proposed sequence of equipment operation for a future construction.

### Performance and Cost Evaluation

After the construction, a combination of leveling surveys and visual inspection was used for monitoring the performance of the test sections. Use of the leveling survey was discontinued after the resurfacing action taken by the township in June 1989. No apparent elevation changes (less than 0.008 ft) were detected in the CKD/FA section between September 1988 and June 1989. In the control (chip seal) section, however, the average settlement was about five times that in the CKD section. At present, the test sections have gone through three winter-spring seasons. No freezing and thawing damage has been detected on the CKD section. On the basis of a recent visual inspection, no damage was found on the CKD section. However, several pumping spots were found on the control section, even though a special resurfacing treatment was performed in 1990.

The overall cost for the CKD section is about six times the cost for the control section. The overall construction cost includes the material, labor, and equipment costs. The major portion of the cost was from purchasing the fines material, including the price for each fine material, one-time plant operation cost, and delivery charges. As mentioned earlier, some unnecessary cost resulted from an excessive amount of cement due to improper material proportions in the central mix plant. In the future, the plant operational cost could be lower if the CKD method is acceptable and becomes widely used. The

plant would then be able to make its operation more cost-effective. Also, the modifications of mix design as well as the field operational procedures could further reduce the overall cost.

### POSTLABORATORY STUDY— EFFECTS OF TIME DELAYS BETWEEN WETTING AND COMPACTION

A prewetting process has been suggested earlier to control the dusting problem. However, the delay between prewetting and placing the material in the field may cause premature chemical reactions that could result in significant strength reduction in the final mixture. A series of postlaboratory tests was designed to evaluate the possibility of strength loss due to these premature chemical reactions. The mix proportioning used for Mix H in Table 5 was used in this study. Special laboratory procedures were set up to duplicate the effects of time delays on the test specimens. These procedures are described as follows:

1. All the fine materials were properly proportioned according to the mix design. One-half of the required mixing water was added to prewet the fine materials.
2. After thorough mixing, the wet fine mixtures were sealed in plastic bags to prevent any moisture loss. The sealed mixtures were held at room temperature (70°F) for various time delays. Two time delays, 4 and 8 hr, were selected and their results were compared with the same mixtures without time delays.

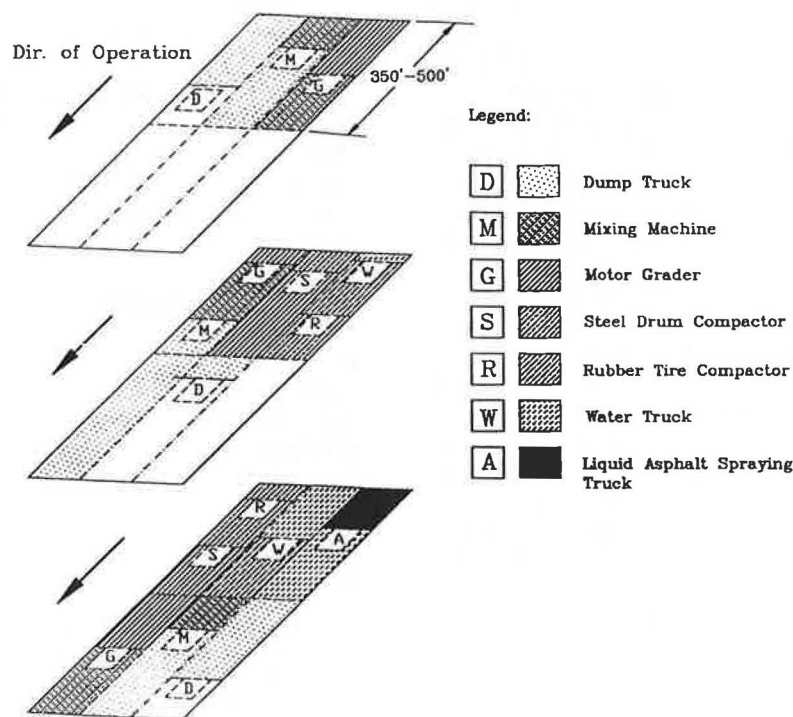


FIGURE 4 Recommended field operations.



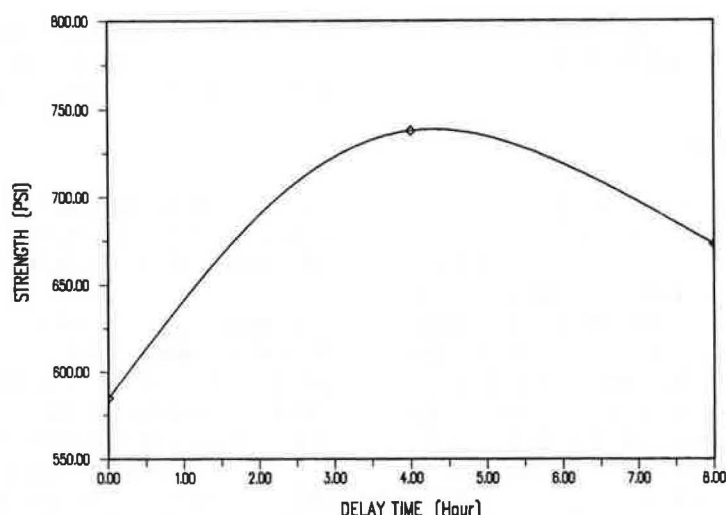


FIGURE 5 Delay time versus strength.

3. After each of the two time delays had elapsed, the reclaimed surface materials and the remaining half of the water were mixed with the wet fine mixtures.

4. Cylinder specimens were fabricated immediately after the completion of the mixing process. Then the specimens were sealed in plastic bags and cured at 70°F for a 14-day period.

5. At the end of the curing period, the unconfined compressive test was performed to determine the strength of each specimen.

Figure 5 shows the relationship between strength and time delay. The strength increased 26 percent as the time delay changed from 0 to 4 hr. Between 4 and 8 hr delay, however, the strength declined. But the strength with an 8-hr delay was still 15 percent higher than the strength with no time delay. It appears that an optimum time delay exists around 4 hr. This outcome is somewhat different from what would be expected. One possible explanation is that different methods of adding water to the mixture were used. For the specimens with no time delay, all the water was added at once to the complete mixture. For the specimens with time delay, the required water was equally divided into two parts to add to the mixtures. The first half of the water was added to the fine materials, and the mixture was thoroughly mixed. Considering that the fine materials were only 25 percent by weight of the overall mixture, the addition of one-half of the water to the fines increases the likelihood that the particles in the fines will come in contact with water. Thus, the possibility for hydration is more than doubled for those specimens with a time delay. The strength loss after a 4-hr delay could be attributed to premature cementitious reactions occurring during the extended time delay. Hence, this method of adding water should be considered in a future construction.

## SUMMARY AND CONCLUSIONS

Valuable experience was gained from the construction of the CKD/FA test section. Several difficulties were encountered

during the construction, yet the performance of the test section to date is satisfactory. The test results indicate that CKD and FA can be used successfully in place of lime or portland cement for roadway base stabilization. It is feasible to recycle the existing chip seal surface with CKD and FA. However, to ensure the quality of construction the following conditions must be met: (a) design of an acceptable mix with minimum contents of fines, (b) accurate proportioning of materials by close inspection in the central plant and proper control of the scarified depth, (c) sufficient mixing moisture, (d) thorough mixing with adequate equipment, (e) shortening of the time delay between mixing and compaction, (f) proper coordination and planning of the field operations, (g) close field inspection with a trained inspector, and (h) favorable curing conditions. Also, on the basis of the laboratory study, time delay between wetting and placing from 0 to 8 hr is likely to increase the strength of the CKD/FA/cement mix. A 4-hr delay can result in a 26 percent increase in strength, and an 8-hr delay can still allow the mix to have a higher strength than that without delay.

## ACKNOWLEDGMENTS

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# Use of By-Product Phosphogypsum in Road Construction

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Phosphogypsum is a solid by-product material resulting from the production of phosphoric acid, a major constituent of many fertilizers. It has been estimated that more than 800 million tons of phosphogypsum are stockpiled in the U.S. The current production rate exceeds 60 million tons per year, with less than 2 percent being recycled at the present time. The results of the laboratory phase of a research program to develop phosphogypsum as a road base material are presented. The research program included physical and chemical characterization of the phosphogypsum, mineralogical analysis, moisture-density relations, unconfined compressive strength testing, freeze-thaw and wet-dry durability tests, and expansion testing. A comparison is made between the engineering properties of phosphogypsum and the Louisiana Department of Transportation and Development requirements for road base materials.

Phosphogypsum is a solid by-product of phosphoric acid production. Phosphoric acid is a major constituent of many fertilizers. Of the several different phosphoric acid production processes, the wet process is the most frequently used (1). In the wet process, finely ground phosphate rock,  $\text{Ca}_{10}\text{F}_2(\text{PO}_4)_6$ , is dissolved in phosphoric acid to form a monocalcium phosphate solution. Sulfuric acid, which is added to the slurry, reacts with the monocalcium phosphate to produce a hydrated calcium sulfate, which can then be separated from the phosphoric acid by filtration.

The resulting filter cake containing the hydrated calcium sulfate is called phosphogypsum. Typically, the filter cake slurry and wash solution are piped to large stockpiles where the phosphogypsum is allowed to settle.

The crystals of calcium sulfate can exist in at least three forms: anhydrite, hemihydrate, and dihydrate. The anhydrous crystal, commonly called calcium sulfate ( $\text{CaSO}_4$ ), has no chemically combined water (water of hydration). The hemihydrate molecule contains one-half molecule of chemically combined water and is denoted by  $\text{CaSO}_4 \cdot \frac{1}{2}\text{H}_2\text{O}$ . The dihydrate molecule contains two molecules of chemically combined water and is denoted by  $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$ . In this form, the mineral is commonly referred to as gypsum. Several modifications of the wet process exist, which are distinguished by the predominant type of calcium sulfate crystal that is produced. The percentages of each crystal type in a given sample

influence, both directly and indirectly, the properties of the phosphogypsum. Goers (2) stated that crystal type and size affect the amount of phosphoric acid remaining in the filter cake after filtration, which, in turn, may affect the engineering properties of the phosphogypsum (3). Legal and Myrick (4) noted that the temperature of the reaction mass and the phosphoric acid concentrations are the controlling factors governing the degree of hydration of the calcium sulfate. Most of the wet-process phosphoric acid produced in the world is made by the dihydrate process or some modification of the process.

As a general rule, 4.5 to 5.5 tons of phosphogypsum are generated for every ton of phosphoric acid produced (5). The demand for phosphoric acid further magnifies the problem of efficiently and economically dealing with growing phosphogypsum stockpiles. In 1980, phosphogypsum was generated at an annual rate of 840,000 tons in Australia alone (6). In 1979, Japan was producing phosphogypsum at a rate of 2,748,000 metric tons per year (7). In the United States, phosphogypsum is currently being produced at a rate of 40 million tons per year. In Florida, more than 334 million tons of phosphogypsum had been stockpiled by 1980. At that time, phosphogypsum production in Florida was 33 million tons per year. Therefore, as of 1990, there are approximately 650 million tons of phosphogypsum in Florida alone. Long-term projections indicate that more than 1 billion tons will be stockpiled in Florida by the year 2000 (8). In Louisiana, more than 95 million tons of phosphogypsum have been stockpiled as of 1989 (9). Another 25 million tons of phosphogypsum also exist in the Houston area (10). Obviously, these large quantities of material must be dealt with before they become unmanageable. Similar problems with use and disposal of this material also exist in Europe and Canada (11).

Other by-product gypsums (BPG) that are available in large quantities in the United States include fluorogypsum, flue gas desulfurization (FGD) gypsum, and titanogypsum. These BPG designations are based on the specific chemical process from which the material is produced. According to the Electric Power Research Institute, 20 million tons of FGD gypsum are generated annually in the United States (D. Golden, personal communication, 1991). With the enactment of the recent Clear Air Act legislation, the current plants will probably add another 20 million tons per year of FGD gypsum. The total current inventory of the material is approximately 150 million tons. In 40 years, it is estimated that the amount of FGD gypsum will quadruple.

The amount of phosphogypsum that has been disposed of to date plus the current rate of production indicate the seriousness of the problem. The disposal of such material

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must address several factors to develop a cost-effective and environmentally sound practice. Such factors include the following:

1. Runoff from stockpiles may be acidic and a potential threat to surface and groundwater resources.
2. The long-term effects of low-level radioactive emissions from stockpiled phosphogypsum are presently unknown.
3. The availability and high cost of land for disposing of the material are becoming major concerns [stockpiles of phosphogypsum covered more than 1,773 acres of Florida land in 1972 (12)].

As stockpiles continue to grow and environmental constraints become more stringent, widespread uses of phosphogypsum must be developed. Ideally, such uses would involve large tonnages of phosphogypsum preferably in its natural or as-produced state. One such possible use could be as a base material in highway construction. To enhance the engineering properties of most phosphogypsum, some cementitious or pozzolanic binder must be added to the system.

## OBJECTIVE

The main objective of this paper is to summarize the pertinent results obtained from the laboratory testing program to evaluate the properties and behavior of cement-stabilized phosphogypsum when used as a base material. Recommendations for additional studies are also presented.

## MATERIALS

### Phosphogypsum

In Louisiana, Freeport-McMoran, Inc. (FMI) uses the dihydrate modification of the wet process to produce phosphate fertilizers. Therefore, the phosphogypsum used in this study is in the dihydrate form, that is,  $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$ .

An isolated mesa was formed on the disposal pile at the Uncle Sam plant in July 1985. This mesa, measuring approximately 500 by 600 ft, had an elevation of about +6 to +10 ft above the top of the active stack area. All phosphogypsum used in the FMI study was collected from the top few inches of the stack and from selected locations in the stockpiles. Characterization of the phosphogypsum will be discussed in later sections of the paper.

### Portland Cement

Different types of portland cement were used in the research program. They are as follows:

- Type I Citadel Demopolis cement (11 percent  $\text{C}_3\text{A}$ ),
- Type II Ideal fine grind cement (7 percent  $\text{C}_3\text{A}$ ),
- Type II Dundee cement (5.6 percent  $\text{C}_3\text{A}$ ),
- Type II Gifford-Hill cement (5 percent  $\text{C}_3\text{A}$ ),
- Type V Gifford-Hill cement (5 percent  $\text{C}_3\text{A}$ ),
- Type V Citadel Leharve cement (5 percent  $\text{C}_3\text{A}$ ),

- Type V United cement (3 percent  $\text{C}_3\text{A}$ ), and
- Type II Lone Star cement (0 percent  $\text{C}_3\text{A}$ ).

The use of various cements is necessary because of the influence of the tricalcium aluminate ( $\text{C}_3\text{A}$ ) present in the cement on the strength development and expansive characteristics of stabilized phosphogypsum mixtures. Most of the cements are commercial cements that will meet the requirements of ASTM C150.

### Sand

A fine sand locally referred to as pump sand was used in the research program. A typical gradation curve for the sand is shown in Figure 1.

## EXPERIMENTAL INVESTIGATION

FMI took the lead in investigating the potential use of phosphogypsum. From 1985 through 1989, they conducted a laboratory investigation of the phosphogypsum produced at their Uncle Sam and Faustina plants (13). Their research program included physical and chemical characterization, mineralogical analysis, moisture-density relations, unconfined compressive strength testing, freeze-thaw and wet-dry durability tests, water submersion tests, and expansion testing. The following sections summarize the results of these studies and include recommendations for further study.

### Chemical and Physical Properties

The chemical and physical characterization of phosphogypsum was carried out as part of the research program. The chemical analysis of the material is given in Table 1. The data presented in Table 1 represent a range of values obtained on

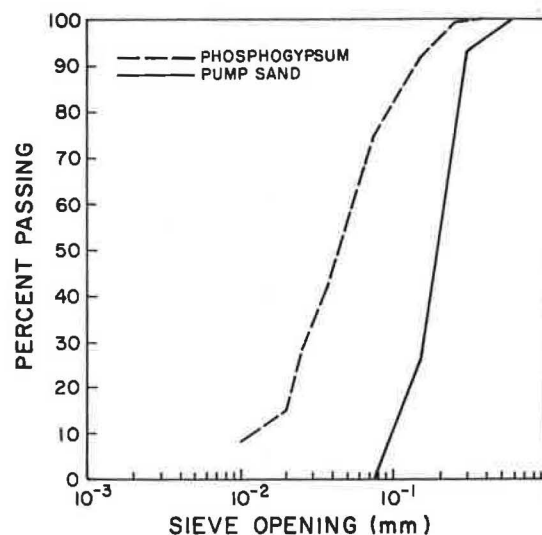


FIGURE 1 Particle size distribution of pump sand and phosphogypsum.

TABLE 1 RESULTS OF CHEMICAL ANALYSIS OF PHOSPHOGYPSUM

Constituent	Content (%)
CaO	29 - 31
SO <sub>4</sub>	50 - 53
SiO <sub>2</sub>	5 - 10
F	0.3 - 1.0
P <sub>2</sub> O <sub>5</sub>	0.7 - 1.3
Fe <sub>2</sub> O <sub>3</sub>	0.1 - 0.2
Al <sub>2</sub> O <sub>3</sub>	0.1 - 0.3
pH*	2.8 - 5.0

\*pH: not measured as a percent.

several samples. In addition, a number of trace elements are present in phosphogypsum. Typical concentrations of these elements are given in Table 2. According to Table 2, lead concentration of fresh phosphogypsum will exceed the EPA leachate standards. However, leachate samples taken from cement or fly ash stabilized phosphogypsum mixtures will meet the EPA leachate requirements.

The physical properties of phosphogypsum, such as moisture content, specific gravity, and fineness, are presented in Table 3. A temperature of 55°C (131°F) was used to determine the free moisture in the phosphogypsum. Temperatures higher than 60°C (140°F) tend to convert the dihydrate phosphogypsum (CaSO<sub>4</sub>·2H<sub>2</sub>O) to the hemihydrate form (CaSO<sub>4</sub>·½H<sub>2</sub>O). Higher temperatures will also cause a significant amount of fracture and delamination of the phosphogypsum particles. These tests were conducted on three phosphogypsum samples dried at three temperatures (room temperature, 140°F, and 257°F).

Table 3 also indicates that more than 75 percent of the phosphogypsum particles will pass the #200 sieve.

### Morphological Analysis

Morphology was investigated by optical and scanning electron microscopy (SEM). Particles were generally radiating ag-

TABLE 2 TYPICAL TRACE ELEMENT CONCENTRATIONS IN PHOSPHOGYPSUM

Element	Concentration (ppm)
As	1 - 5
Ba	50
Cd	0.3 - 0.4
Cr	2 - 5
Pb	2 - 10
Hg	0.02 - 0.05
Se	1
Ag	0.1 - 0.2
U <sub>3</sub> O <sub>8</sub>	5 - 10

TABLE 3 RESULTS OF PHYSICAL ANALYSIS OF PHOSPHOGYPSUM

Property	Value
Free Moisture (top)	8 - 18% (varies with depth)
Free Moisture (phreatic water)	25 - 30%
Specific Gravity (average)	2.35
< #200 sieve (average)	75%

glomerates of platelets. Some of the aged or weathered materials were more rounded and denser. A few samples showed unagglomerated platelets. Figure 2 shows a typical micrograph of phosphogypsum sampled from the Uncle Sam stockpile. It shows the friable texture of the phosphogypsum.

### Mechanical Properties

Strength development in phosphogypsum mixtures can be achieved through stabilization with either portland cement, lime, fly ash, or combinations thereof. To determine the strength development, the optimum moisture content and maximum dry density values were first established using a standard Proctor laboratory compaction test series. An optimum moisture content of 21.5 percent and a maximum dry density of 88.5 pcf were determined for unstabilized phosphogypsum mixtures. Typical moisture-dry density relations for phosphogypsum mixtures stabilized with 5 percent Type II Ideal fine grind cement are shown in Figure 3. Samples were either prepared in accordance with ASTM D1557—Method A (modified Proctor) or as specified by ASTM D698 (standard Proctor). The latter method is essentially the same as the Louisiana Department of Transportation and Development (LDOTD) Test Method TR418—Method B. As can be seen in Figure 3, higher maximum dry density and lower optimum moisture values were obtained with the higher compaction energy.

The following procedure was adopted in the preparation of the cement-stabilized phosphogypsum specimens. Raw phosphogypsum was initially dried at a temperature of 131°F to a constant weight. The cement was added on a dry weight basis. The phosphogypsum and cement were dry mixed for 30 sec. Water was added as necessary to bring the mixture to the required optimum moisture (water was taken as the percentage of the total dry materials). The mixture of phosphogypsum, cement, and water was mixed for about 2.5 min to obtain a visibly homogeneous blend. The materials collected at the bottom of the mixing bowl following mechanical mixing were thoroughly mixed with the rest of the material by hand. The mixture was covered at all times with a towel during the compaction process to avoid the evaporation of water.

After completing the compaction series, duplicate specimens were prepared at optimum moisture content for strength determination using the unconfined compression test (ASTM D1633). Most molded specimens were cured in plastic bags and were subjected to a 4-hr soak in water before compression testing. The 7- and 28-day compressive strength results for cement-stabilized phosphogypsum mixtures compacted in accordance with the standard and modified Proctor methods are shown in Figures 4 and 5. Modified Proctor compaction produced higher dry densities and resulted in 7- and 28-day com-





FIGURE 2 Micrograph of phosphogypsum from the Uncle Sam stockpile.

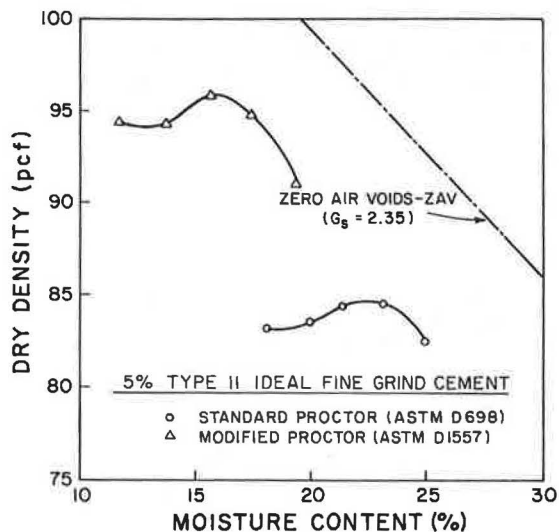


FIGURE 3 Moisture-dry density relations of cement-stabilized phosphogypsum mixtures.

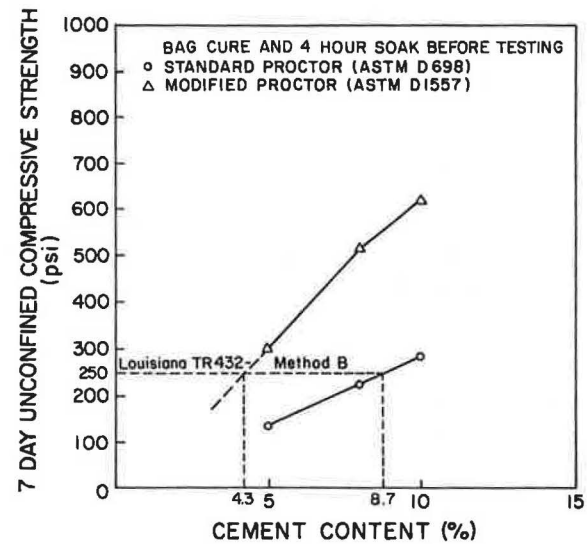


FIGURE 4 Seven-day unconfined compressive strength for Type II Ideal fine grind cement-stabilized phosphogypsum mixtures prepared under different compaction procedures.

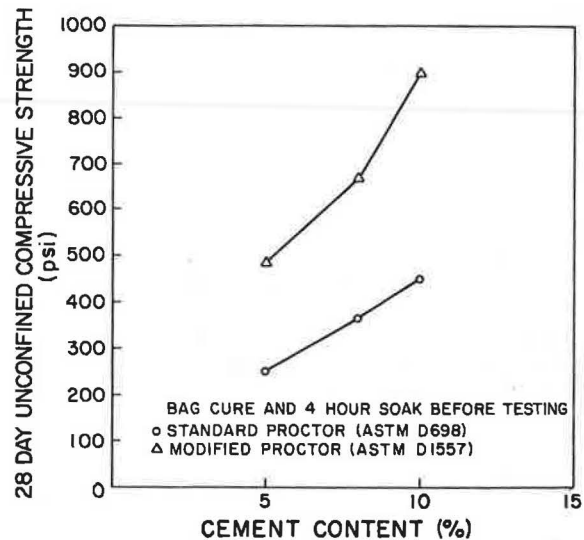


FIGURE 5 Twenty-eight-day unconfined compressive strength for Type II Ideal fine grind cement-stabilized phosphogypsum mixtures prepared under different compaction procedures.

pressive strengths about 130 and 90 percent higher, respectively, than obtained for specimens prepared using standard Proctor compaction.

LDOTD requires a minimum 7-day compressive strength of 250 psi for soil-cement mixtures (Test Method TR432—Method B). From Figure 4, a minimum of 8.7 percent cement will satisfy the required strength criteria if standard Proctor compaction is used. However, about 4 percent cement is needed to achieve a minimum 7-day strength of 250 psi if modified Proctor compaction is used. Therefore, the question of the

density that can be achieved in the field becomes of great significance.

Figure 6 indicates the effect of curing time and cement content on the compressive strength of cement-stabilized phosphogypsum mixtures. With an increase in cement content and curing period, there is a corresponding increase in the unconfined compressive strength.

Figure 7 shows that the use of either Type II or V cements will have little effect on the strength development in stabilized phosphogypsum mixtures. Both are sulfate-resistant cements, and similar engineering behavior (in particular, strength and sulfate attack resistance) should be expected when using both stabilizers with phosphogypsum.

Phosphogypsum samples collected at different depths within a stockpile may have different pH values. This will influence

the strength of various cement-stabilized phosphogypsum mixtures. Some of the added cement will be exhausted in neutralizing the more acidic phosphogypsum. Quality control is an important issue in the use of phosphogypsum stockpiles. A study that addresses the influence of pH on the strength of cement-stabilized phosphogypsum mixtures is currently under way at the Institute for Recyclable Materials.

### Water Submersion Testing

The effects of long-term water submersion on the strength of specimens of stabilized phosphogypsum were investigated. Initially the specimens were cured for either 1 or 4 weeks in double-sealed plastic bags. The test conditions then called for submersion of specimens in a water bath with the water flowing at a rate of 600 mL/min (equivalent to a replacement volume of the water in the bath every 2 hr). The specimens were dried with a towel and immediately tested in unconfined compression. This test was developed to simulate the worst possible case of complete saturation of the base material during a rainy season. Figure 8 shows that all of the specimens prepared using the Type II Lone Star cement showed a decrease in unconfined compressive strength after water submersion for as little as 1 week. Phosphogypsum stabilized with 8 percent Type II Lone Star cement showed an unconfined compressive strength of 311 psi after 1 week bag cured. However, the strength decreased to 235 psi for similar stabilized phosphogypsum mixtures submerged under water for 23 weeks. On the other hand, the specimens prepared with Type II Dundee cement and Type II Ideal fine grind cement along with the additive microsilica showed an increase in strength after submersion. Phosphogypsum stabilized with 8 percent Type II Dundee cement showed an unconfined compressive strength of 269 psi after 1 week bag cured. When similar

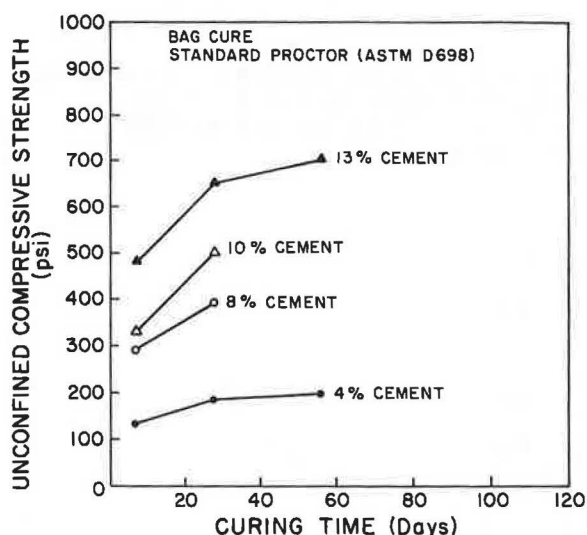


FIGURE 6 Unconfined compressive strength versus curing time for phosphogypsum mixtures stabilized with Type II Lone Star cement.

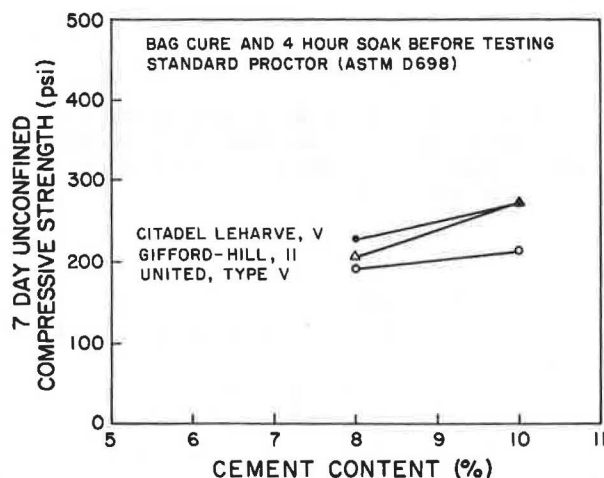


FIGURE 7 Seven-day unconfined compressive strength for phosphogypsum mixtures stabilized with Types II and V cements.

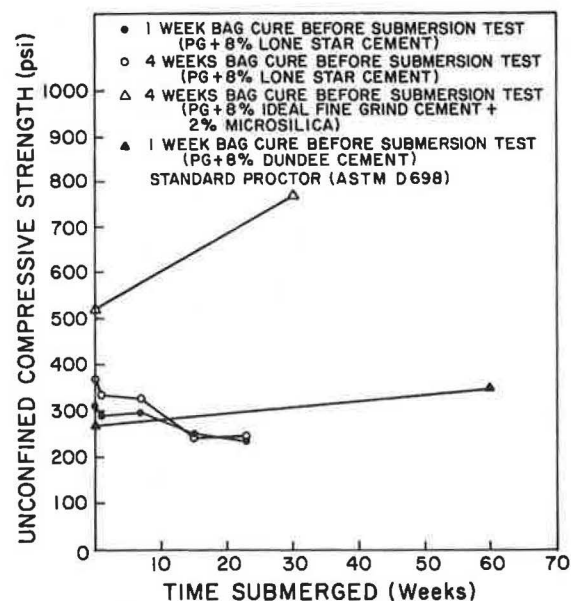


FIGURE 8 Effect of water submersion on the unconfined compressive strength of stabilized phosphogypsum mixtures.

stabilized phosphogypsum mixtures were submerged under water for 60 weeks, the strength increased to 350 psi.

### Expansion Testing

In an effort to determine the suitability of a given Portland cement for use in stabilizing phosphogypsum, a long-term reactivity test was used. The test was conducted using equipment specified by ASTM Test Methods C511 and C490. Stabilized phosphogypsum and pump sand specimens were prepared using 4-in.-diameter by 11.25-in.-high molds, and the length change of test specimens was measured along the longitudinal axis using a comparator. The specimens were cured for approximately 3 years in humidity bath cabinets at 100°F and 100 percent relative humidity. Figure 9 shows the expansion results of phosphogypsum and pump sand specimens stabilized with different types of cement. The most expansion was observed in the phosphogypsum specimens prepared using Type I Citadel Demopolis cement with 11 percent  $C_3A$  content. On the basis of these results, it is noted that there is an increase in the expansion of stabilized mixtures of phosphogypsum and cement as the  $C_3A$  content in the cement increases. Also, the expansion continues but at a decreasing rate as the curing period increases. The least expansion was observed in the pump sand specimens prepared using Type I Citadel Demopolis cement.

Scanning electron microscopy tests conducted on phosphogypsum mixtures stabilized with 10 percent Type I Citadel Demopolis cement indicated the formation of ettringite crystals. Long slender prismatic needles that characterize ettringite crystals were observed. This formation may well be related to the phenomenon of sulfate attack expansion in high sulfate mixtures prepared using cement or lime as stabilizers.

### Freeze-Thaw Test Results

Porous materials containing moisture are susceptible to damage under repeated cycles of freezing and thawing (frost attack).

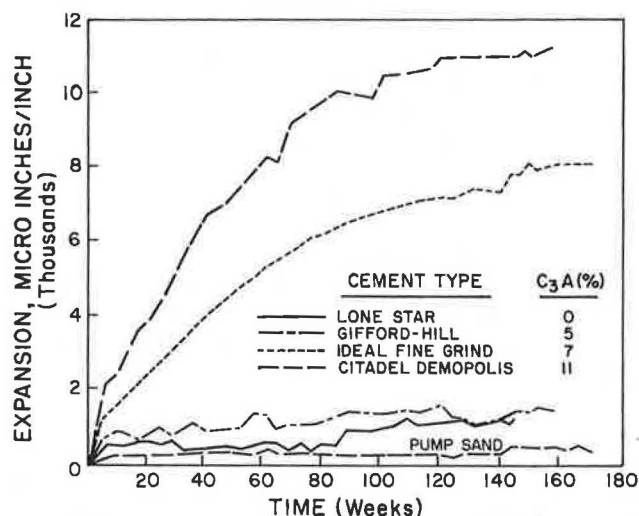


FIGURE 9 Expansion results of the cement-stabilized phosphogypsum and pump sand mixtures.

Stabilized phosphogypsum specimens were compacted at optimum moisture content using ASTM Procedure D698 (standard Proctor compaction) and were subjected to freeze-thaw cycles using modifications of AASHTO Procedure T-136. The freeze-thaw test involved subjecting the test specimens to 12 cycles of freezing and thawing followed by measurements of weight losses and volume changes. One freeze-thaw cycle was accomplished by placing the specimens at -10°F for 24 hr and then for another 24 hr at 70°F. After each cycle, some specimens were brushed and then weighed to determine weight loss. Other specimens remained unbrushed for control measurements. Final dry weights of the molded specimens were determined after placing them in an oven at 131°F. Table 4 presents the results of the freeze-thaw durability tests. The

TABLE 4 FREEZE-THAW DURABILITY TEST RESULTS FOR CEMENT-PUMP SAND-PHOSPHOGYPSUM MIXTURES

Method	Mix Design			Volume Change (%)	Uncorrected Brushed Weight Loss (%)	7-Day Strength Bag Cure (psi)	After Test	
	Phosphogypsum (%)	Cement (%)	Sand (%)				Brushed Strength (psi)	Unbrushed Strength (psi)
Brushed	100	8	0	-5.8	-4.2	283	442	621
Brushed	100	10	0	-3.6	-1.7	354	740	856
Brushed	100	12	0	-2.2	+0.1	338	800	1067
Brushed	80	8	20	-4.6	-2.6	344	542	836
Brushed	60	8	40	-3.7	-2.3	370	672	947
Brushed	0	10	100	-4.4	-2.5	N/A	464	721
Brushed	100	8*	0	-5.0	N/A	311	402	605

Notes: 1. All tests used Type II Dundee cement with exception noted.

2. \*Type II Lone Star cement.

3. Cure time for all tests using Type II Dundee cement was 651 days.

4. Cure time for the test using Type II Lone Star cement was 202 days.

5. N/A: not available.

data indicate that a maximum volume loss of 5.8 percent is produced for phosphogypsum mixtures stabilized with 8 percent Type II Dundee cement. An increase in cement content will decrease the volume change and weight loss in a freeze-thaw test. The use of 8 percent Type II Lone Star cement with phosphogypsum produced a volume loss of 5.0 percent after the completion of the freeze-thaw test. The weight loss behavior exhibited by the specimens in this test series will meet the recommended Portland Cement Association (PCA) maximum weight loss of 10 percent for siltlike materials (phosphogypsum falls under this category). Thus, these results generally indicate that stabilized phosphogypsum mixtures should perform satisfactorily as base materials subject to frost action.

### Wet-Dry Test Results

Cement-stabilized systems may be subjected to swelling and shrinkage produced by repeated wetting and drying of the hardened system. In a state such as Louisiana, where rainfall exceeds the amount of evaporation and rainfall, humidity, and temperature are high during the summer, the wet-dry durability test is of special significance.

Stabilized phosphogypsum specimens were prepared using standard Proctor compaction and were subjected to wet-dry cycles using modifications of AASHTO Procedure T-135. The wet-dry test involves subjecting the test specimens to 12 cycles of wetting and drying followed by measurements of weight loss and volume change. One wet-dry cycle was accomplished by submerging the specimens in water at room temperature for 5 hr and then for another 42 hr in an oven at 131°F. After each cycle, some specimens were brushed to determine weight loss. Other specimens remained unbrushed for control measurements. Final dry weights of the molded specimens were determined by placing them in an oven at 131°F. Table 5 presents the results of the wet-dry durability tests. The data

indicate that, with an increase in cement content, there is a decrease in the volume change and weight loss in a wet-dry test. All weight loss data will meet the recommended PCA-specified maximum weight loss of 10 percent. No brushed weight loss data are available for the phosphogypsum mixture stabilized with 8 percent Type II Lone Star cement. However, the volume loss observed for the phosphogypsum mixture stabilized with 8 percent Type II Lone Star cement was 10.2 percent, which may be significant under repeated wet-dry cycles. This cement had a lower  $C_3A$  content than the Type II Dundee cement. Thus, the amount of  $C_3A$  present in the cement may influence the results of the wet-dry test.

### CONCLUSIONS

The laboratory studies completed to date warrant the following conclusions:

1. The laboratory results indicate that portland cement-stabilized phosphogypsum mixtures can be successfully used as road base and subbase materials.
2. The type and amount of Portland cement will significantly influence the performance of stabilized phosphogypsum mixtures.
3. Moisture content and compactive effort have a significant influence on the dry density and compressive strength of stabilized phosphogypsum mixtures.
4. Durability tests indicate satisfactory performance of selected stabilized phosphogypsum mixtures.
5. Selection of portland cement type in stabilizing phosphogypsum systems should be approached cautiously. The reaction between the tricalcium aluminate present in the cement and the sulfates in the phosphogypsum can lead to the formation of ettringite and thaumate crystals. Eventually, cracking of the pavement may occur.

TABLE 5 WET-DRY DURABILITY TEST RESULTS FOR CEMENT-PUMP SAND-PHOSPHOGYPSUM MIXTURES

Method	Mix Design			Volume Change (%)	Uncorrected Brushed Weight Loss (%)	7-Day Strength Bag Cure (psi)	After Test	
	Phosphogypsum (%)	Cement (%)	Sand (%)				Brushed Strength (psi)	Unbrushed Strength (psi)
Brushed	100	8	0	-4.6	3.3	283	503	668
Brushed	100	10	0	-3.2	2.2	354	759	747
Brushed	100	12	0	-2.9	1.7	338	761	1002
Brushed	80	8	20	-4.6	3.5	344	609	876
Brushed	60	8	40	-5.0	3.7	370	605	1131
Brushed	0	10	100	-5.1	4.1	N/A	560	818
Brushed	100	8*	0	-10.2	N/A	311	393	697

- Notes:
1. All tests used Type II Dundee cement with exception noted.
  2. \*Type II Lone Star cement.
  3. Cure time for all tests using Type II Dundee cement was 651 days.
  4. Cure time for the test using Type II Lone Star cement was 202 days.
  5. N/A: not available.

## RECOMMENDATIONS

Additional laboratory studies are needed to investigate the underlying phenomenon for the expansion that was observed in the cement-stabilized phosphogypsum specimens. A study of this kind will be initiated soon at the Institute for Recyclable Materials, College of Engineering, Louisiana State University to enhance our knowledge of the fundamental chemical and physical mechanisms contributing to the stability and strength of cement-stabilized phosphogypsum mixtures. Furthermore, flexural fatigue and dynamic modulus testing will be included in the follow-up testing program.

## ACKNOWLEDGMENT

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# Stabilization of Water Treatment Plant Sludge for Possible Use as Embankment Material

M. C. WANG, J. Q. HULL, AND M. JAO

The compaction, compressibility, and shear strength behavior of an alum sludge both untreated and treated with additives are presented. The significance of the sludge behavior in terms of landfill/embankment design and construction is discussed. The additives used were a slake lime, a fly ash, and a local clay sand; the additive content was 60 percent by dry weight of sludge. The standard Proctor comparative effort and the conventional consolidation test were used for testing. The shear strength was determined using both the laboratory fall cone penetration test and the consolidated undrained triaxial compression test with pore pressure measurements. The moisture-density relation is not that of the typical one-hump curve. Instead, the dry density decreases with increasing water content from a maximum near zero water content. The additives appear to have an insignificant effect on the compaction curve. However, the additives improve the plasticity, compressibility, and shear strength behavior considerably. The untreated sludge has a high effective angle of internal friction, but the undrained shear strength is low, especially at high water content. Both the untreated and treated sludges are sensitive and highly thixotropic. It is concluded that admixture stabilization is an effective method of improving sludge properties for easier handling and also for increasing the volume of disposed sludge. Although the stabilized sludge within the range of conditions investigated is still too compressible and without sufficient shear strength for embankment construction, the potential for use of stabilized sludge as an embankment material can be enhanced through further dewatering or raising additive content or both.

Water treatment plant sludge is the residue generated from the various coagulation processes used in producing potable water (1). According to the National Academy of Sciences (2), the amount of sludges derived from treatments of both drinking and waste water has doubled between 1972 and 1990 because of population growth, industrial development, and upgrading of treatment plants as mandated by various federal and local legislation. Thus, disposal of the ever-increasing volume of water treatment sludge has become a national problem.

The traditional disposal methods for water plant sludge include ocean barging; direct discharging into streams, storm sewers, or sanitary sewers; lagooning; and land applications. Because of economic reasons and the decreasing availability of land and more stringent regulations regarding the ultimate disposal of sludges, considerable attention has been directed toward the method of landfill disposal. Besides serving the purpose of disposing of the unwanted sludge, sludge landfills

can possibly be used as roadway embankments, dikes, and for other engineering applications.

Water plant sludge has a high water content and is plastic, sensitive, and compressible. Without treatment, it is difficult to handle and to landfill to an economic height with a desired slope. This study is concerned with using the admixture stabilization method to improve sludge properties and with evaluating the possibility of using the stabilized sludge for embankment construction.

## PRESENT STATE OF KNOWLEDGE

The geotechnical properties required for design and performance analysis of sludge landfills include mainly compaction, compressibility, and shear strength. The present state of knowledge of these properties of both untreated and treated water plant sludges is presented.

In one of the few available studies on this subject matter, Raghu et al. (3) performed the modified Proctor compaction test on a lime/alum/polyamine coagulant water plant sludge obtained from the Jersey City Water Treatment Plant. The compaction water content varied from about 55 to 72 percent. The compaction curve exhibited the typical one-hump shape having an optimum water content of approximately 65 percent and a maximum dry unit weight of about 51 lb/ft<sup>3</sup> (pcf). Elsewhere, Environmental Engineering & Technology, Inc. (4) reported the results of standard Proctor compaction tests on a coagulant sludge that had been stored at a water plant in Oklahoma for an extended time. The test results also showed the typical one-hump compaction curve. The optimum moisture content and maximum dry unit weight were approximately 17 percent and 105 pcf, respectively.

Raghu et al. (3) also conducted a conventional consolidation test to determine the coefficient of permeability of the coagulant sludge. No compressibility behavior of the sludge was presented, however. Using the method of dewatering under different pressure differentials, Knocke and Wakeland (5) investigated the compressibility of four sludges—alum (low density), alum (high density), conditioned alum, and lime sludges. They reported that the average coefficient of compressibility for the first three alum sludges ranged between 0.94 and 0.97, whereas that of the lime sludge was 0.79, indicating that the lime sludge is considerably less compressible than the alum sludge. The coefficient of compressibility is not the compression index; the relation between the two is not yet available.

Available information on the shear strength property of water treatment plant sludge concentrates mainly on the undrained strength with regard to sludge handleability (or workability). Most studies used the vane shear test method to determine the strength. Novak and Calkins (6) studied five sludges using a torvane and reported that 0.04 tsf is the minimum undrained shear strength for easy handling of the sludge. This minimum shear strength value is about 2.5 times lower than 10 kPa (0.104 tsf) currently used in West Germany and the Netherlands. Using both a torvane and a viscometer, Huang (7) presented, among others, a relationship between the undrained shear strength and solids concentration for two sludges. It was pointed out by Knoke and Wakeland (5) that water treatment plant sludge undergoes marked strength increase due to thixotropic hardening. However, little information on the thixotropic behavior of sludge is currently available.

For improving the handling characteristics of water treatment plant sludge, Farrel et al. (8) investigated the effectiveness of lime stabilization. They reported that the addition of a quicklime to an alum sludge resulted in an increased shear strength because of the formation of a calcium-aluminum compound. However, research findings from Europe indicated that sludge treatment with quicklime did not always lead to an increase in shear strength to more than 10 kPa (4).

The preceding account indicates that the current knowledge on geotechnical properties of water treatment plant sludge is limited. For the effective design and performance analysis of sludge disposal landfills, fundamental properties such as compaction, compressibility, and shear strength are essential. In addition, the potential use of stabilized sludge for embankment construction cannot be evaluated without the necessary geotechnical properties of stabilized sludge. The results of a study that addresses this research need are presented in this paper.

## TEST SLUDGE AND ADDITIVES

The sludge investigated is an alum sludge obtained directly from a water treatment plant in Chesapeake, Virginia. Some sludge samples were also obtained from a landfill in Chesapeake using plastic tubes having 2-in. ID with wall thickness of  $\frac{1}{8}$  in. The test sludge is the treatment residue of high-color, low-turbidity raw water. It was dewatered using the centrifuge method. It has a specific gravity, liquid limit, plastic limit, and plasticity index of 2.26, 550 percent, 239 percent, and 311 percent, respectively. The sludge contains 66 percent silt size and 20 percent clay size particles with a median size of 0.013 mm. It is classified as CH according to the unified soil classification system. The activity is equal to 15.6, which is many times higher than that of common clay minerals. On the basis of the behavior of clays, the abnormally high activity value suggests that the test sludge is very sensitive, highly compressible, and expansive. The sludge received directly from the treatment plant has a water content of about 714 percent and a wet unit weight of approximately 67 pcf, whereas the field samples have water contents ranging between 270 and 669 percent and wet unit weights varying from 67 to 73 pcf.

The additives used for stabilization were a slaked lime, a fly ash, and a natural soil. The slaked lime was obtained from Bellefonte, Pennsylvania. The fly ash is of Class C type (i.e.,

self-cementing) obtained from Indiana and Michigan Electric Company at Rockport, Indiana. The local soil is a clayey sand and is classified as SC according to the unified classification system. It contains about 14 percent silt size and 5 percent clay size particles with a median particle size of approximately 0.6 mm; its liquid limit and plastic limit values are 39 and 19 percent, respectively.

The additive content was arbitrarily chosen at 60 percent by dry weight of sludge. The specific gravities of the treated sludges are 2.46, 2.44, and 2.44, and the liquid limits are 272, 305, and 297 percent for the lime-, fly ash-, and soil-treated sludges, respectively. The data indicate a large decrease in liquid limit due to treatment.

## COMPACTION BEHAVIOR

The standard Proctor compaction curves of the untreated and treated sludges are shown in Figure 1. The zero-air-void curve (ZAVC) is plotted using the specific gravity value of the untreated sludge. It is seen that the shape of the compaction curves is different from that of the usual one-hump curve; the dry unit weight decreases from a maximum near zero water content, then decreases as the water content increases. Such a moisture-density relation is rarely seen. The one-hump compaction curve has been obtained for other waste treatment plant sludges (3,4). Although sludge type will certainly influence the shape of the compaction curve, other factors such as the effect of drying and rewetting, as briefly described later, may also play an important role. An in-depth study of this subject is warranted.

According to the compaction curve, to reach the maximum dry density, the molding water content must be as low as possible. The data also indicate that mixing with any of the three different additives has little effect on the compaction behavior, although the addition of slaked lime slightly lowers the dry unit weight when the molding water content is low.

The compaction test was executed from the wet side rather than from the dry side as specified in the ASTM standard test procedures. This deviation is necessary to accommodate the unusual behavior of the test sludge. Upon drying, the sludge particles attract together to form flocs. Each floc is strongly held together so that it is difficult to break apart to make possible a uniform mixture with water. Furthermore, when the sludge is dried below a certain water content and rewetted, it loses its original cohesive characteristic and becomes a sandy material. For all of these reasons, the compaction test was started from a high water content, which was then decreased gradually by air drying inside the laboratory. For air drying, the sludge was spread in a drying pan and was periodically stirred and the flocs broken apart by hand to make the water distribution as uniform as possible.

## CONSOLIDATION BEHAVIOR

The consolidation test was performed on both untreated and treated sludges. The treated sludges were cured for 2 and 4 weeks. The void ratio versus logarithm of pressure relation ( $e$ -log  $p$  curve) of the untreated and treated sludges is shown in Figure 2. As shown, the shape of the  $e$ -log  $p$  curves for

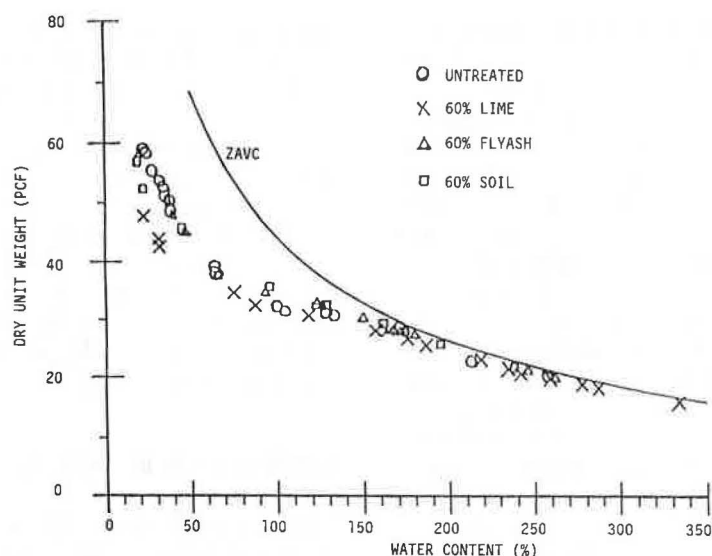


FIGURE 1 Compaction curves of test sludge with and without additives (13).

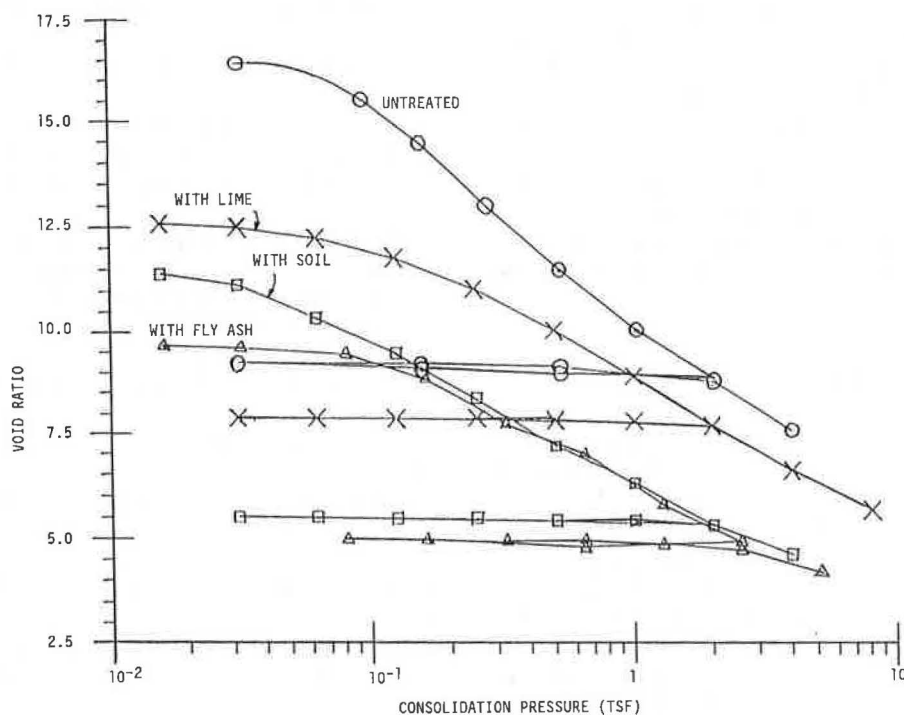


FIGURE 2 Void ratio versus consolidation pressure of treated and untreated sludges.

the treated sludges is similar to that of the untreated one, but the initial void ratio is much lower. Table 1 gives the initial void ratio together with water content and compression index. The smaller initial void ratio of the treated sludge is attributable to the decreased water content due to mixing of the additives. For a saturated soil, the initial void ratio equals water content times specific gravity. Because the specific gravity of the treated sludge is only slightly higher but the water content is considerably lower than that of the untreated one,

the initial void ratio of the treated sludge will be lower than that of the untreated one.

The water content and initial void ratio vary among the various treated sludges. The data indicate that the water content is lowest for the fly ash-treated sludge and about equal between the soil- and lime-treated sludges (Table 1). Although the reason why the water content of fly ash-treated sludge is lowest is not yet known, it may be related to possible pozzolanic reaction of fly ash. The pozzolanic reaction may

TABLE 1 CONSOLIDATION TEST RESULTS

Test Material	Water Content (%)	Initial Void Ratio	Compression Index
Untreated Sludge	766.8	17.33	6.69
Soil Treated Sludge	466.4	11.38	4.04
Lime Treated Sludge	2-wk cured	443.9	10.92
	4-wk cured	511.8	12.59
Fly Ash Treated Sludge	2-wk cured	404.1	9.86
	4-wk cured	395.9	9.66

depress the diffuse double layer and cause the original highly dispersive sludge particles to flocculate, resulting in drainage of free water. There may also be uptake of water in pozzolanic reaction. The consequence of either or both of these two effects will be a much lower water content than for sludge without pozzolanic reaction. The data also indicate only a small difference in water content between 2- and 4-week cured fly ash-treated sludges. This suggests that within 1 month of curing, the effect of curing time on water content change caused by chemical reaction, if any, is negligible.

As seen, the values of the compression index are high. Furthermore, the compression index of the untreated sludge is considerably higher than that of the treated ones. Among the various treated sludges, only a slight variation in compression index is seen. The lower compression index of the treated sludges can be attributed to the decreased interparticle repulsion caused by the additives. As a result of the reduced interparticle repulsive force, particles flocculate to form more stable flocs and, therefore, the compression index decreases.

The decompression index of the untreated sludge is extremely low, as is the decompression index of the treated sludges, as shown by the slopes of the decompression curves (rebound curves) in Figure 2. Because of the highly dispersive nature of the structure of sludge particles, the low decompression index can be expected.

## SHEAR STRENGTH BEHAVIOR

The shear strength of untreated sludge, both plant and field samples, was determined using the consolidated undrained triaxial compression test with pore pressure measurements. Three confining pressures ranging from 15 to 55 psi were used. The initial water content ranged between 326 and 477 percent, and the final water content varied from 247 to 345 percent. The laboratory fall cone penetration test was also used to determine the undrained shear strength of both untreated and the various treated sludges.

The shear strength parameters and pore pressure parameter at failure ( $A_f$ ) obtained from the triaxial compression tests are summarized in Table 2. A close agreement between the plant and field samples data is seen. The values of  $A_f$  indicate that the test sludge is behaving as a possibly normally consolidated soil with a high sensitivity, according to Skempton (9). The effective internal friction angle ( $\phi'$ ) is unusually high. As noted before, the test sludge has a plasticity index (PI) equal to 311 percent, which is also abnormally high. When  $\sin \phi'$  and PI values are entered into the correlation developed by Kenney (10) and Olson (11) and reported by Mitchell (12),

TABLE 2 SHEAR STRENGTH PARAMETERS OF UNTREATED SLUDGES

	Total Stress		Effective Stress		Pore Pressure Parameter, $A_f$
	c (psi)	$\phi$ (°)	c' (psi)	$\phi'$ (°)	
Plant Sample	0.6	19.3	1.0	42.3	0.75
Field Sample	0.7	19.0	1.2	44.0	0.80

the data point lies far above the extended correlation, as shown in Figure 3. On the basis of this comparison, it can be expected that the behavior of the test sludge cannot be predicted using the data base established for natural soils.

The undrained shear strength of the untreated sludge obtained from the cone penetration test is plotted against solids content in Figure 4. The solids content is defined as the ratio between the weight of the solid phase and the total sludge weight expressed in percent. This definition is normally adopted by the environmental engineer in dealing with sludges. According to this definition, the solids content can be related to the water content as follows:

$$\text{Solids content (\%)} = \frac{1}{1 + \frac{\text{water content (\%)}}{100}} \times 100 \quad (1)$$

The data in Figure 4 demonstrate that, as would be expected, the undrained shear strength increases with increasing solids content. The rate of increase is slow at the beginning, then intensifies at higher solids content. The curve eventually should level off at a high solids content, although no such trend appears in the range of solids content studied.

To investigate the thixotropic behavior of the test sludge, the undrained shear strength was determined for both remolded and cured specimens. The test specimens were prepared at four water contents, which were chosen arbitrarily: 511.9, 634.2, 737.1, and 804.4 percent. For each water content, the cone penetration resistance was measured at different curing times after remolding. The specimens were sealed and cured at room temperature. Details of specimen preparation, curing, and testing are available elsewhere (13). The variation of undrained shear strength with curing time obtained is shown in Figure 5. Curing results in an increase in the undrained shear strength. Initially, the strength increases rapidly; the rate of increase decreases with curing time and eventually levels off.

The curves in Figure 5 indicate that the amount of strength gain due to curing and the rate of strength increase at the initial curing stage are greater when the sludge water content is lower. From these curves, the maximum shear strength for each water content is obtained and the strength gain ratio, defined as the ratio between the cured and remolded strengths, is computed. The computed strength gain ratios are plotted against water content in Figure 6. The data indicate that the strength gain ratio increases to a maximum, then decreases with increasing water content. The maximum strength gain ratio of about 8.0 occurs at a water content of approximately 740 percent. On the basis of the range of strength



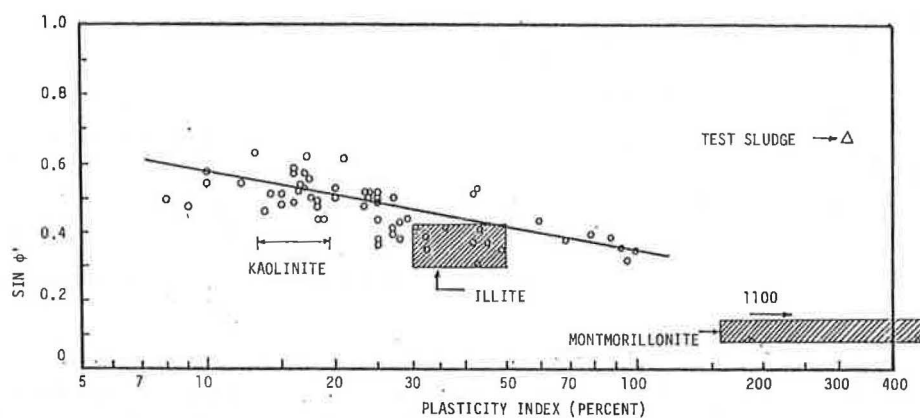


FIGURE 3 Relationship between  $\sin \phi'$  and plasticity index for normally consolidated soils (12).

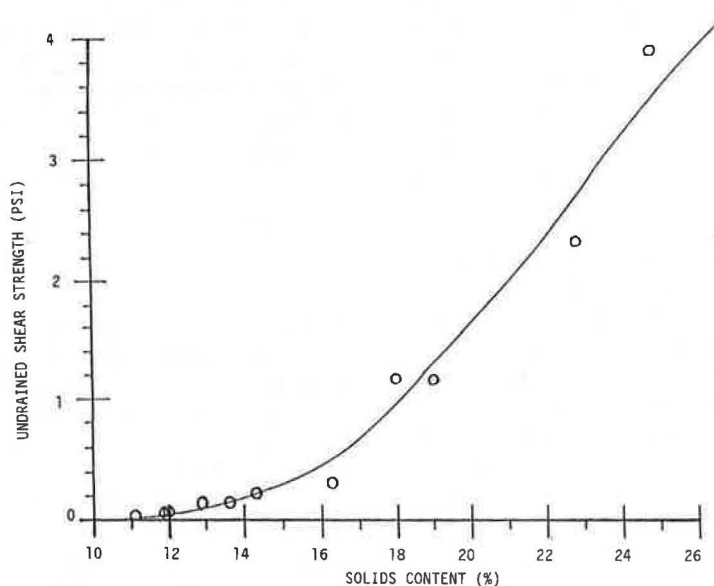


FIGURE 4 Undrained shear strength of untreated sludge determined from cone penetration test versus solids content (13).

gain ratios, if the untreated test sludge was a soil, it could be classified as very sensitive to slightly quick clay according to Rosenqvist (14).

The change in strength gain ratio with water content can be explained using Mitchell's hypothesis on thixotropy. According to Mitchell (12), the saturated clay soil can be characterized as a clay-water-electrolyte system. Before remolding, the system is everywhere in an equilibrium condition. In terms of interparticle forces, the attractive force is balanced with the repulsive force. Remolding causes particle displacement and reorientation and thus disrupts the balanced interparticle forces. After remolding, the system tends to return to the original equilibrium condition. The process of restoration may involve particle reorientation or readjustment of the diffuse double layer or both.

When the water content is low, the particles are closely spaced. At a close interparticle spacing, significant interparticle attractive force exists to catalyze restoration of the system to the original equilibrium condition. On the other hand,

however, the particles are tightly restrained, making any reorientation difficult. As a result, only a limited strength regain is possible after remolding. When the water content is high, the interparticle spacing is large so that the interparticle repulsive force dominates. Under the influence of interparticle repulsion, restoration of the system to the original equilibrium condition would be more difficult, and, therefore, strength regain after remolding would be limited. Thus, there would be an optimum water content at which a significant interparticle attraction exists, and at the same time particles are not tightly restrained for readjustment.

The curing time required to attain the maximum strength gain also varies with water content. Within the range of water content investigated, the higher the water content, the shorter the curing time, as shown in Figure 6. The shorter curing time at higher water content can be attributed to the decreased restraint of sludge particles, so that the particles can readjust themselves to the equilibrium condition existing before remolding.



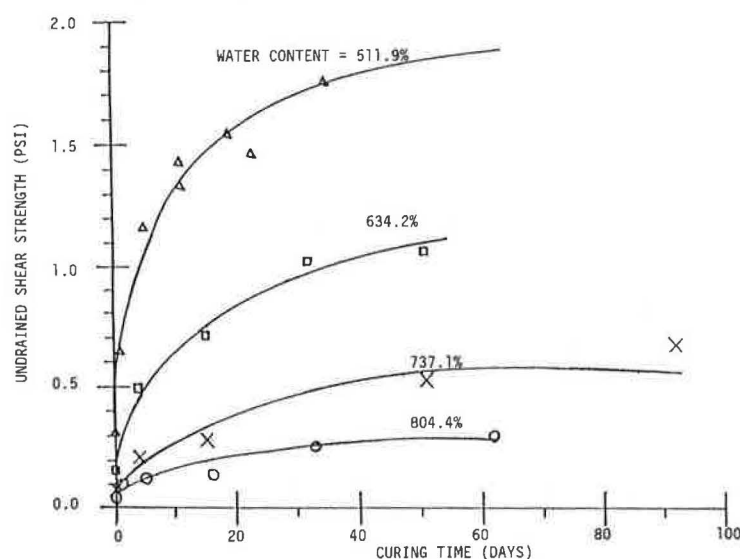


FIGURE 5 Undrained shear strength versus curing time for untreated sludge at different water contents.

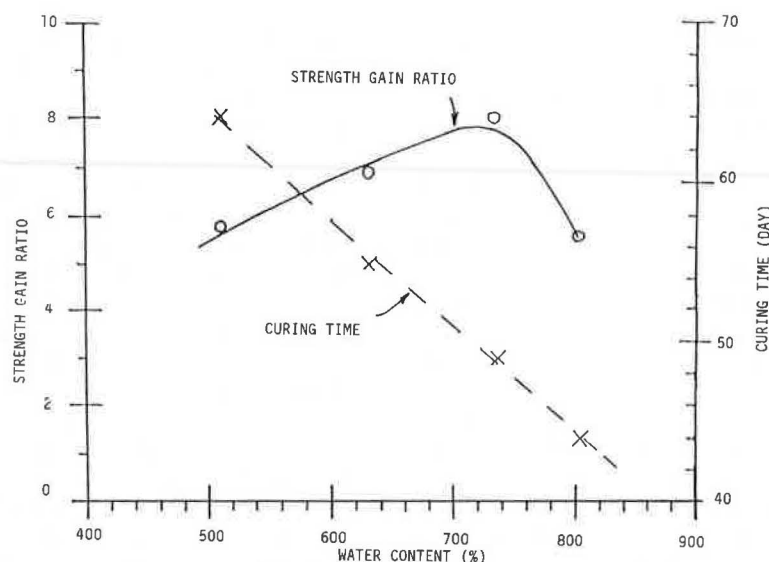


FIGURE 6 Strength gain ratio and curing time versus water content of untreated test sludge.

The effect of additives on the undrained shear strength of the test sludge is shown in Figure 7, which presents the undrained shear strength versus curing time for both untreated and treated sludges. Before treatment, the water content of the sludge equals 513.5 percent, which was arbitrarily chosen. After mixing with the additive (60 percent by dry weight of sludge), the water content became 311.5 percent. As shown, the shear strength increases with curing time following the trend of Figure 5. The data points for treated specimens are more scattered than the untreated one, possibly due to mixing. Although mixing was done by hand as thoroughly as possible, it appears that the mixture was far from uniform.

Despite data scatter, the trend is clear that the additives result in higher shear strengths; the amount of strength increase is greatest for lime, followed by fly ash, then soil. The values

of cured and uncured strength are given in Table 3. Also included in Table 3 is the strength gain ratio. The results seem to indicate that although the increased strength is greatest for lime and smallest for soil, the soil-treated sludge has the highest strength gain ratio, followed by fly ash, then lime treatment. The cause for this effect is yet to be investigated.

#### ENGINEERING SIGNIFICANCE

For landfill and embankment construction, the sludge must have sufficient strength to support its own weight plus the load induced by construction operations. The strength can be enhanced by increasing the density, which can be accomplished by either compaction or consolidation or both.

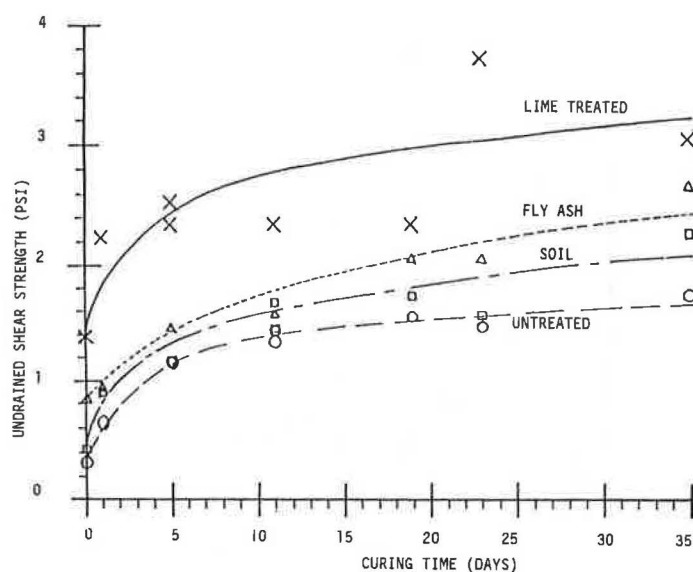


FIGURE 7 Undrained shear strength versus curing time for untreated and treated sludges.

Increasing sludge density is also necessary to maximize the volume of disposed sludge.

The compaction curve shown in Figure 1 indicates that the dry unit weight of the untreated test sludge can be quadrupled from about 15 to 60 pcf when the water content is decreased from about 350 to 20 percent. To attain the maximum dry density, the water content must be as low as possible. However, because dewatering the sludge to a very low water content requires a considerable amount of time and effort, it may be more economical to compact the sludge at a higher water content. The determination of the optimum compaction water content requires considerations of not only the dry density but also economic factors.

Regardless of the type of additive, the various treated sludges investigated behave essentially the same as the untreated sludges. One point that needs to be made clear is that admixing will decrease the water content. As a result, the dry unit weight of the sludge/additive mixture will become higher than that of the sludge alone at the premixing water content. Thus, the benefit of admixture treatment can be reaped shortly after mixing, compared with the considerable time and effort required for sludge dewatering. Furthermore, the negligible effect of additive type on compaction behavior suggests that the desired dry unit weight can be attained by mixing with any material.

Under its own weight, the sludge landfill/embankment will undergo consolidation settlement. The amount of settlement is proportional to the compression index. Since the compression index of the treated sludges is only about one-half to two-thirds of the treated sludge (Table 1), the ultimate settlement of landfills/embankments constructed of treated sludge will be only about one-half to two-thirds of untreated ones. Such a magnitude of reduction in consolidation settlement is significant when considering the potential application of treated sludge as an embankment material.

The effective internal friction angle of the untreated sludge (Table 2) is high, indicating that the sludge has good potential for long-term stability. The undrained shear strength, espe-

cially at high water content, is low but increases rapidly with decreasing water content. Thus, to make a landfill/embankment stable at a desired height and slope angle, the water content must be as low as possible. Under a constant water content, the sludge undergoes thixotropic strength gain; the strength gain makes the landfill/embankment more stable as time elapses. Because the sludge is sensitive to disturbance, the remolded strength should be used in the design and stability analysis of landfill/embankment slopes for the as-constructed condition to account for possible disturbance effects due to transportation and construction operations.

Admixing stabilization improves the remolded shear strength of sludge drastically, by a factor of more than four times for lime, almost three for fly ash, and about 1.5 for soil (Table 3). As a result of the strength improvement, the height of a landfill with a trapezoidal cross section will also increase by a factor of about 4, 3, and 1.5 for lime, fly ash, and soil treatment, respectively, assuming equal wet unit weights. The landfill slope will also become more stable with time, similar to the untreated sludge.

In summary, admixing stabilization makes the sludge less plastic and therefore easier to handle. Upon mixing, the water content decreases and dry density increases. Also, stabilization decreases the compressibility and increases the shear strength. The improvement in shear strength and dry density increases the volume of disposed sludge. The reduction in compressibility decreases the landfill settlement.

TABLE 3 UNDRAINED SHEAR STRENGTH OF UNTREATED AND TREATED SLUDGES

Sludge	Remolded	Cured	Strength Gain Ratio
Untreated	0.30 psi	1.70 psi	5.7
Treated with Soil	0.42	2.10	5.0
Treated with Fly Ash	0.82	2.50	3.0
Treated with Lime	1.39	3.25	2.3

For use of the stabilized sludge as an embankment material, the compressibility of sludge must be much lower and the undrained strength much higher. All of these may be accomplished by either further dewatering the sludge before mixing or increasing the additive content or, both.

## SUMMARY AND CONCLUSIONS

The compaction, compressibility and shear strength behaviors of an alum sludge both untreated and treated with a slake lime, a fly ash, and a local soil were presented. The additive content is 60 percent by dry weight of sludge. The standard Proctor compactive effort and the conventional consolidation test were adopted for the determination of compaction and compressibility behaviors. The shear strength was determined using both the laboratory fall cone penetration test and the consolidated undrained triaxial compression test with pore pressure measurements.

The results of the study indicate that the compaction curve is not that of the typical one-hump shape. Instead, the dry density decreases from about 60 pcf at approximately 20 percent water content to about 15 pcf at 330 percent water content. The additives appear to have an insignificant effect on the compaction curve. However, the additives decrease the compression index to about one-half to two-thirds of the untreated sludge. The untreated sludge has a high effective internal friction angle. The undrained shear strength of untreated sludge is low, especially at high water content. The addition of lime, fly ash, and soil increases the remolded shear strength by a factor of more than 4 times, almost 3 times, and about 1.5 times the untreated strength, respectively. Both untreated and treated sludges are sensitive and highly thixotropic.

It is concluded that admixing stabilization improves the workability, compaction, compressibility, and shear strength properties of the sludge tremendously. For the range of conditions investigated, the treated sludge is still too compressible and without sufficient strength for use as an embankment material. However, both compressibility and shear strength can be improved by either dewatering further or raising the additive content or both. The potential of using treated sludge for embankment construction can be enhanced by improving the compressibility and shear strength properties.

## ACKNOWLEDGMENT

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# Construction and Performance of a Shredded Waste Tire Test Embankment

PETER J. BOSSCHER, TUNCER B. EDIL, AND NEIL N. ELGIN

The construction and performance of a test embankment designed to evaluate the use of shredded waste tires as soil replacement in highway construction are described. The shredded tires offer the advantage of low unit weight and durability. The test embankment was designed and built to test key variables including chip size, confining overburden pressure, and use of chip-soil mixtures or chip-soil layering. The embankment consisted of sections, each 20 ft long, containing differing chip-soil compositions. The embankment was constructed parallel to the access road of a sanitary landfill and exposed to the heavy incoming truck traffic. Field data were collected to assess the stability and deformation of the road surface, compaction of tire chips, and quality of tire chips leachate. Observations were made to assess the potential difficulty of depositing and compacting layers of tire chips. Normal construction machinery can be used successfully with tire chips, though rubber tires can be punctured by the exposed wires at the edge of the chips. Vibratory or static compaction does not significantly induce compaction in tire chips. After an initial adjustment period, the overall road performance was similar to most gravel roads. Tire chips used as a replacement for fill under a road perform better when covered by 3-ft-thick soil caps compared with chips covered by only 1 ft of soil. Furthermore, the void ratio of the pure tire chips affects its stiffness. The leachate quality data indicate that shredded automobile tires show no likelihood of having adverse effects on groundwater quality. The findings support the use of properly confined tire chips as a lightweight fill in highway applications.

With the banning of whole tires in sanitary landfills, stockpiles of waste tires are growing in the country. For example, it is estimated that there are 20 million waste tires in Wisconsin, with an additional 4 million generated each year. Whole tires are not easily disposed of for several reasons, including their poor compressibility and their potential combustibility and associated toxic fumes. Recycling is also difficult due to the composite structure of tires: integrally combined rubber, synthetic fibers, and steel wire. Shredding is one common means of modifying waste tires to ease disposal. Finding large-volume uses of shredded tires is desirable to increase the lifetime of sanitary landfills.

The findings from an experimental test embankment designed and constructed to provide information regarding the behavior of waste tire chips as a fill material are summarized. The research specifically examined the difficulty of constructing a fill made of tire chips, the stability and deformability of tire chips used below a road surface, and the environmental acceptability of the leachate that passed through the tire chip fill (1).

## CHARACTERIZATION AND CLASSIFICATION OF TIRE CHIPS

To characterize and classify the range in size and shape of shredded tires, an inventory of the waste tire stockpiles and shredders in Wisconsin was developed by visiting most of the tire-shredding operation sites. The shredding process was videotaped at the sites visited, representative bag samples were taken, and the owners/operators were interviewed. Data were collected regarding shredding operations, type of machinery used, product cost, and product types and characteristics. Most processors use fairly small mobile shredding equipment of 30 to 100 HP. These shredders use a shearing process in contrast to the tearing process in the older versions. The shearing process produces more uniform products, makes cleaner cuts, and eliminates the partial pulling of the reinforcing wires out of the tire chips. The production rate ranged from 100 to 400 tires per hour depending on the machinery type and desired chip size. The cost of shredding ranged from \$30 to \$65 per ton, 1 ton equaling approximately 100 tires.

The size of the tire chips is dictated primarily by the design of a particular machine and the setting of its cutting mechanism. Small chips are produced by processing the material through more than one shredder, each set to produce finer cuts than its predecessor. Classifiers can also be used to separate the finer sizes from coarser ones. Usually the chips are shaped irregularly, with the smaller dimension being the size specified by the manufacturer and the larger dimension two to four times as much. Samples collected from sites visited range from 1 × 2 in. to 4 × 18 in., with the most common size chip being 2 × 3 in. Classifiers are not typically used on these sites. Figure 1 shows information concerning the minimum tire chip sizes of samples collected from four shredding sites. The names of the tire chips were assigned on the basis of the location of the shredding process.

After reviewing the tire chip samples collected, three distinct size groups were recognized. The Edgar chips were the smallest and were produced by a tearing action resulting in significant amounts of loose metal fibers. At the other extreme, the Verona chips (not plotted in Figure 1 because of size extremes) had passed through only one cycle of shredding and consisted of large striplike pieces up to a length of 12 in. Between these two extremes were the Franklin and the Rodefild tire chips. These chips had an intermediate size, the Rodefild chips being somewhat coarser than the Franklin chips. Because of the availability of the Rodefild chips at the construction site, it was decided to use the Edgar, Verona, and Rodefild tire chips with the Rodefild the main tire chips to be investigated. The large chips, referred to as the Verona

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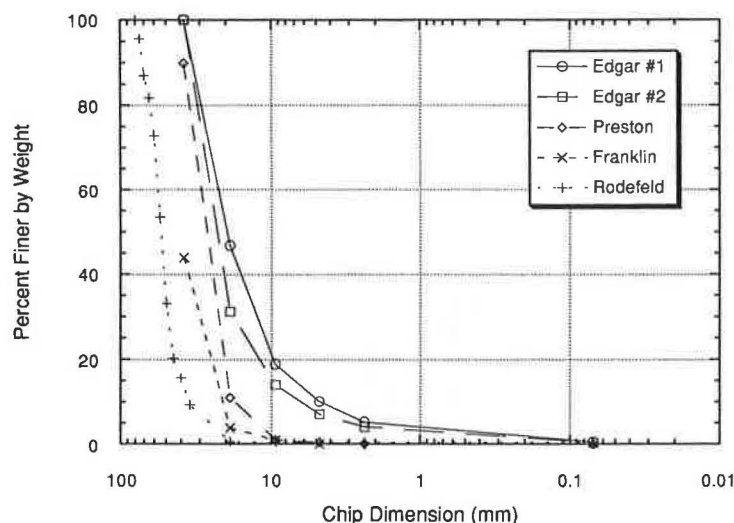


FIGURE 1 Size distribution data of tire chips.

chips, were originally sampled at the Dane County Verona Landfill. However, the first cycle of shredding at the Rodefeld site produced a product similar to the Verona chips. Therefore, in the construction of the test embankment, the coarse Rodefeld material was used. However, the name "Verona" was retained for ease of identification.

### TEST EMBANKMENT DESIGN AND CONSTRUCTION

The test embankment was constructed in Dane County Landfill No. 2 (Rodefeld Landfill) near Madison, Wisconsin. This site offered a sufficient supply of the main chip size to be tested and an abundant soil source. The test embankment was constructed parallel to the access road near the landfill entrance, allowing the diversion of a known quantity of heavy traffic (incoming refuse trucks) onto the test embankment as desired. The trucks were individually weighed as they brought their refuse to the landfill, providing a record of the traffic. Regular weather monitoring and ground and surface water monitoring at the landfill site provided environmental data.

#### Test Embankment Design

In designing the test embankment, the following factors were considered:

1. Tire chip size and type,
2. Soil type and chip-to-soil ratio for chip-soil mixture, and
3. Placement conditions (pure chips, mixed with soil, or layered).

An available soil at the landfill site (a glacial outwash gravely sand) was chosen to be mixed with the chips. The grain size curve of this sand is given in Figure 2. It is a well-graded, predominantly coarse-grained material with some fines in it. It is classified as an A-1-b(0) granular material in the AASHTO system and an SW well-graded sand with gravel in the USCS.

Three soil-chip compositions were adopted for study: pure tire chips, tire chips mixed with soil, and tire chips layered with soil. For the chip-soil mixture, a ratio of 50 percent tire chips and 50 percent sand by volume was chosen. The mixing was achieved in the field by a relatively simple operation using a backhoe. The layered tire chip and soil section was built by placing alternating 1-ft lifts of tire chips and sand. In this section, an overall ratio of tire chips to sand was targeted to be the same as used in the mixture.

In two of the sections a thicker soil cap was designed to assess its effect in reducing the deformation of the chips under traffic loading. In addition to testing tire chips, it was decided to include a fiber-reinforced soil in the construction of the test embankment. Reinforcement fibers obtained from Synthetics Industries were mixed into the sand at two different ratios and placed in two separate sections in the test embankment. This paper will focus only on the behavior of the tire chip sections of the embankment. However, reference is made to these fiber-reinforced sections for the purpose of comparison.

On the basis of site geometry and number of variables, the test embankment has eight sections, each 20 ft long as shown in Figure 3. The six tire chip-soil cross sections are shown in Figures 4a and 4b. Along with the approaches built of sand, the test embankment included a total of 10 sections to be monitored and compared. The test embankment has a nominal height of 6 ft with side slopes of 1V:2H. The crest width is 16 ft wide to permit safe passage of large trucks.

#### Field Compaction

After scraping off the surface organic soils, an elevation survey was made of the foundation base. A geotextile was then placed to separate the foundation soils from the embankment materials. A geotextile was also placed between the tire chips and the soil cap in Section 4 as well as over the whole embankment before placing the gravel wear coarse and the organic soils on the slopes. The embankment was constructed in six lifts of approximately 1 ft each. During the compaction



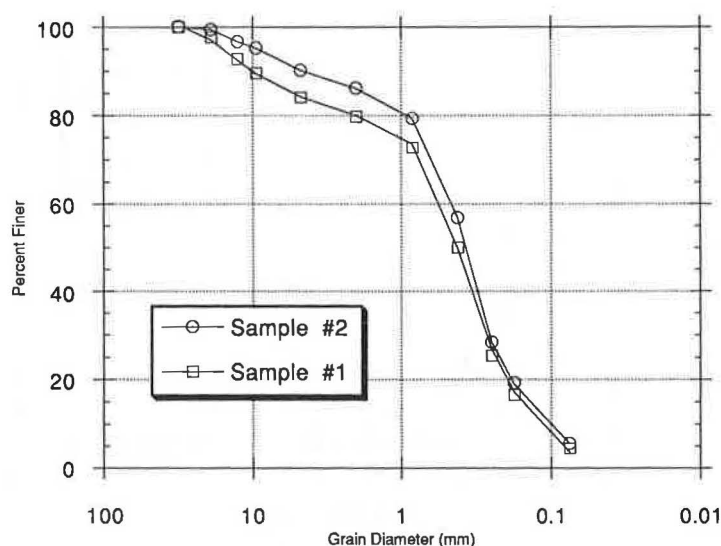


FIGURE 2 Grain size distribution of outwash sand.

of the first lift, surface elevation measurements were made after each pass of the compactor. Since the surface was irregular due to the large tire chips, these measurements were made from a  $2 \times 2$  ft steel plate placed at the center of each section. The compactor used was a Case 1102 PD 12-ton sheepsfoot roller with vibratory capability (26 ft-tons). The north half of the first lift was compacted with vibration on, whereas the south half was subjected to no vibratory action. The corresponding lift thicknesses were computed from the surface elevations.

Examination of the data indicated that vibratory action improved compaction of the tire chips somewhat (25 measurements out of 40 showed more compaction for the vibrated side), but not enough to support use of vibratory rollers. A comparison of the first pass with the fifth pass indicated no additional compaction in the tire chip sections. In general, tire chips showed little plastic deformation due to compaction, unlike the soil sections.

The density achieved during construction was monitored by computing unit weights from a record of the weights of the soils and chips used and the embankment dimensions. Figure 5 shows the calculated unit weights of each material as they were encountered in each section (e.g., S1 = Section 1). Densities of the Rodefeld and Edgar tire chips (medium and finest size) were comparable at 25 to 35 lb/ft<sup>3</sup> range, but the density of the Verona chips (coarser chips) was decidedly lower at 19.3 lb/ft<sup>3</sup> even though the Verona chips were confined with an additional foot of sand because of their large initial compression during the placement of the soil cap (1). Sand used in interlayering with the Rodefeld chips in Section 1 was not compacted as well as the top cap sand layer in Sections 3 and 4 (60 lb/ft<sup>3</sup> versus 105 to 111 lb/ft<sup>3</sup>). In Section 1 this may be due to the lack of inertial mass of soil to compact against, whereas in Sections 3 and 4 the soil mass is two or three times thicker. The Rodefeld/Sand mixture used in Section 6 had an average unit weight of 75 lb/ft<sup>3</sup>.

### Embankment Instrumentation

The instrumentation of the embankment was designed to evaluate the compressibility of the embankment. Slope stability was not deemed to be a problem because of the high friction angle of tire chips; the angle of repose of the piles of tire chips was more than 50 degrees. The compressibility behavior of the test embankment was monitored by regular surveys of surface markers and settlement plates located in the embankment. For surface surveys, target markers were placed at seven locations in the center of each section, as shown in Figure 6. The marker consists of a 2-in. square, 1/4-in.-thick plate with a 10-in.-long #4 rebar anchor welded in its center. There are 70 markers on the embankment and the two approaches. Periodic surveys of these markers provide the  $x$ ,  $y$ , and  $z$  coordinates of these points and the changes thereof.

In addition, 10 settlement plates were placed in the embankment. The settlement plates are standard Wisconsin DOT plates, consisting of a 2-ft-square plate with a rod and a friction pipe. They were placed roughly in the midheight of the embankment (on the third compacted lift from the base) in each test section. An additional plate was placed on the foundation base in Section 4 to measure foundation settlement (see Figure 4b).

Two leachate collection lysimeters used for obtaining leachate samples were constructed by cutting a 1-ft-deep, 10-ft by 12-ft hole in the sand base (see Figure 3). The collection lysimeters were placed in the base of Sections 2 and 5 because these sections contained pure tire chips and were thought to be the sections in which the leachate would be the most affected by the tire chips. The lysimeters consisted of a 10- $\times$ -12-ft square, 30-mil-thick PVC liner with 1-ft-high sidewalls. It was filled with gravel and fitted with a 4-in. pipe boot. A nonwoven geotextile covered the collection system. The base of the hole was sloped to the center and in the direction of the 4-in. pipe. The pipe conducts the collected leachate to

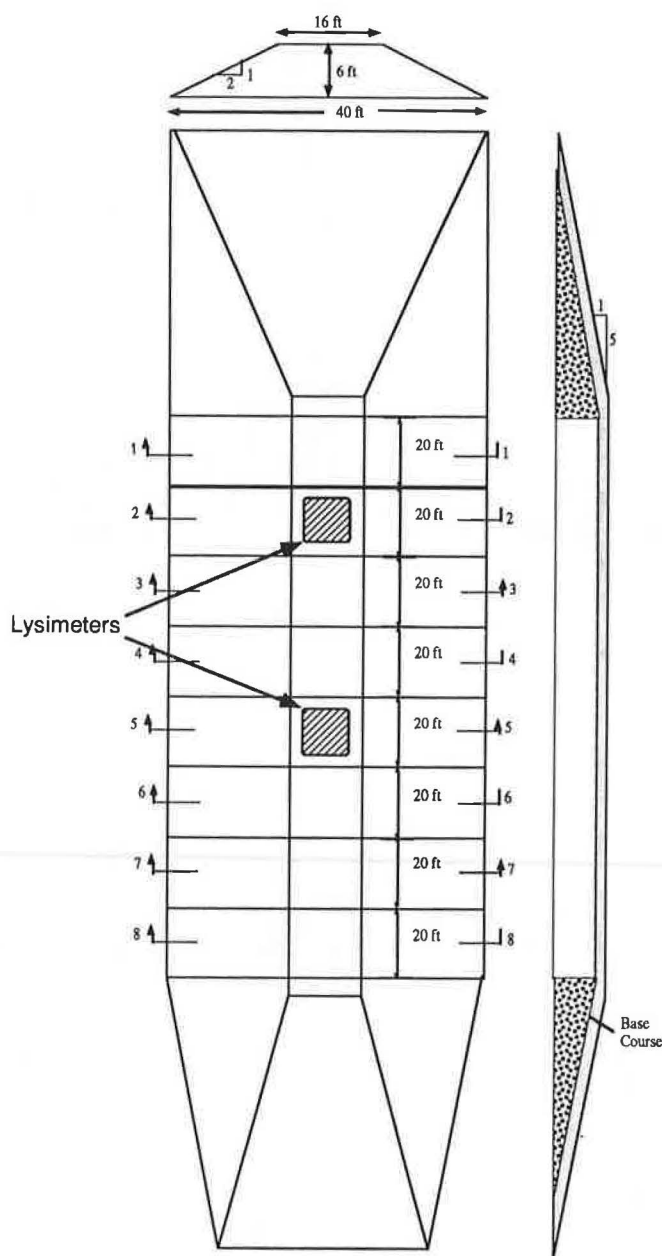


FIGURE 3 Plan view of test embankment design.

the south side of the slope into a 66-in.-deep cylindrical container of inside diameter 6 in. The container is fitted with a cap and allows retrieval of leachate samples. The pipes and the container were all built using PVC.

#### Construction Observations

The following observations were made during the construction process:

1. The handling and placement of the tire chips were not problems. The backhoe seemed more capable of spreading

the material evenly for each section than the front-end loader or grader.

2. Tracked equipment had no trouble maneuvering over the shredded tire fill, but trucks occasionally became stuck and had to be pulled out when the lifts of pure tire chips were more than 2 ft thick. Flat tires on dump trucks also occurred from driving over tire chips.

3. Vibratory compaction did not have an advantage over nonvibratory compaction because of the low inertial mass of the tire chips and their tendency to rebound.

4. Although the chips compressed with each pass of the roller, rebound of the chips was visible behind the compactor. Only the first pass appears to induce a small amount of per-

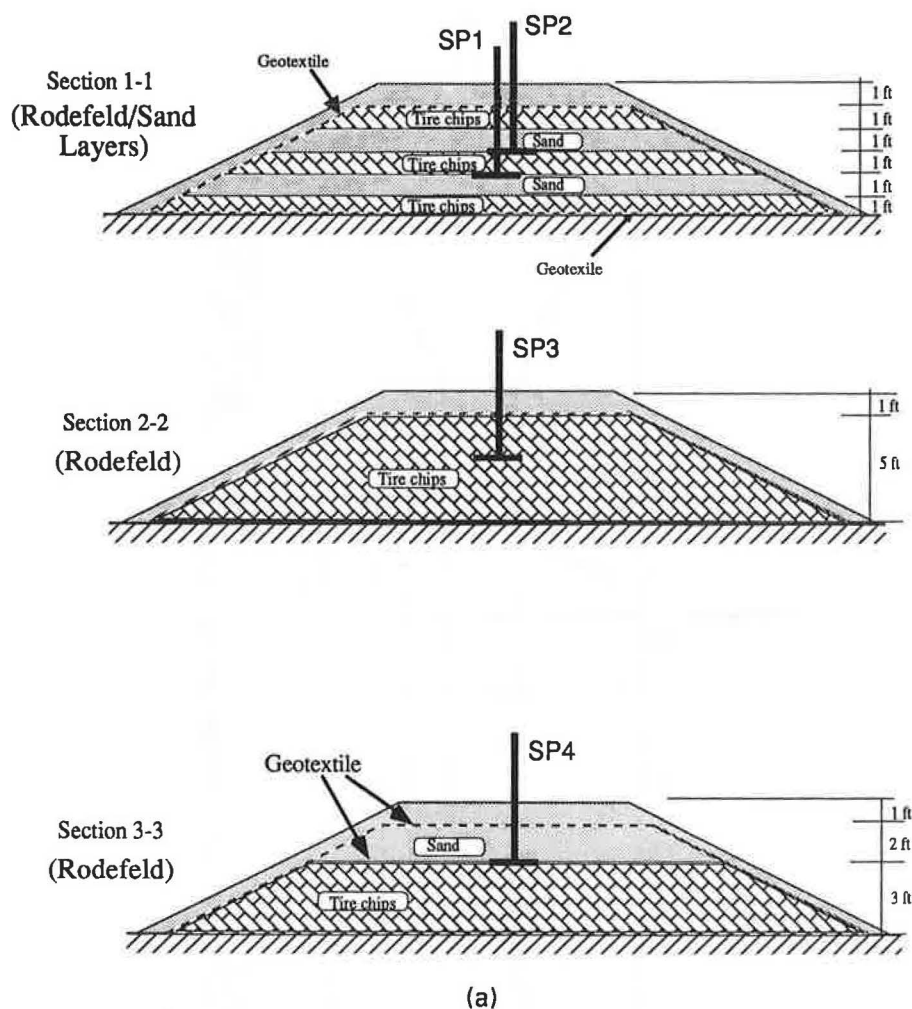


FIGURE 4 Elevation view of test embankment sections as designed: (a), Sections 1, 2, and 3; (b), Sections 4, 5, and 6. (continued on next page)

manent compaction, with the other passes being totally ineffective.

## TEST EMBANKMENT PERFORMANCE

### Traffic and Maintenance

The embankment was constructed parallel to the access road of a sanitary landfill and exposed to the heavy incoming truck traffic, which is weighed before entering the landfill. Approximately 60 to 100 trucks per day weighing an average of 21.6 tons per vehicle pass over this embankment. The standard deviation of truck weight is approximately 10 tons; the weight of some trucks is more than 45 tons. On June 4, 1990, the test embankment was opened to traffic. On June 8, 1990, the embankment required regrading because of immediate rutting under the traffic load. This was accomplished by adding 32 tons of crushed rock (base course) over the whole test embankment. From June through August the embankment

went through several cycles of regrading, opening to traffic, rutting and pothole formation, closing to traffic, and back to regrading. The west approach of the embankment was particularly affected even though it was built using soil without any tire chips. Furthermore, dust due to traffic was threatening the air quality near the landfill. On October 3, 1990, the west approach was regraded using 31 tons of base course. In addition, minor work was performed on the east approach. Later a calcium chloride treatment for dust control was applied, and the embankment was reopened to traffic on October 26, 1990. On December 3, 1990, the test embankment was closed to traffic for the winter after a record snowfall (17 in.). On April 10, 1991, potholes that developed in the east approach, especially at the contact with the asphalt pavement, were repaired, and the embankment was again opened to traffic. Traffic has been routed over the embankment throughout spring, summer, and fall 1991 with virtually no closures. At the present time, an asphalt concrete pavement is being placed on the embankment to permit traffic throughout the coming winter months and to investigate the performance of asphalt concrete pavement founded on a tire chip fill.

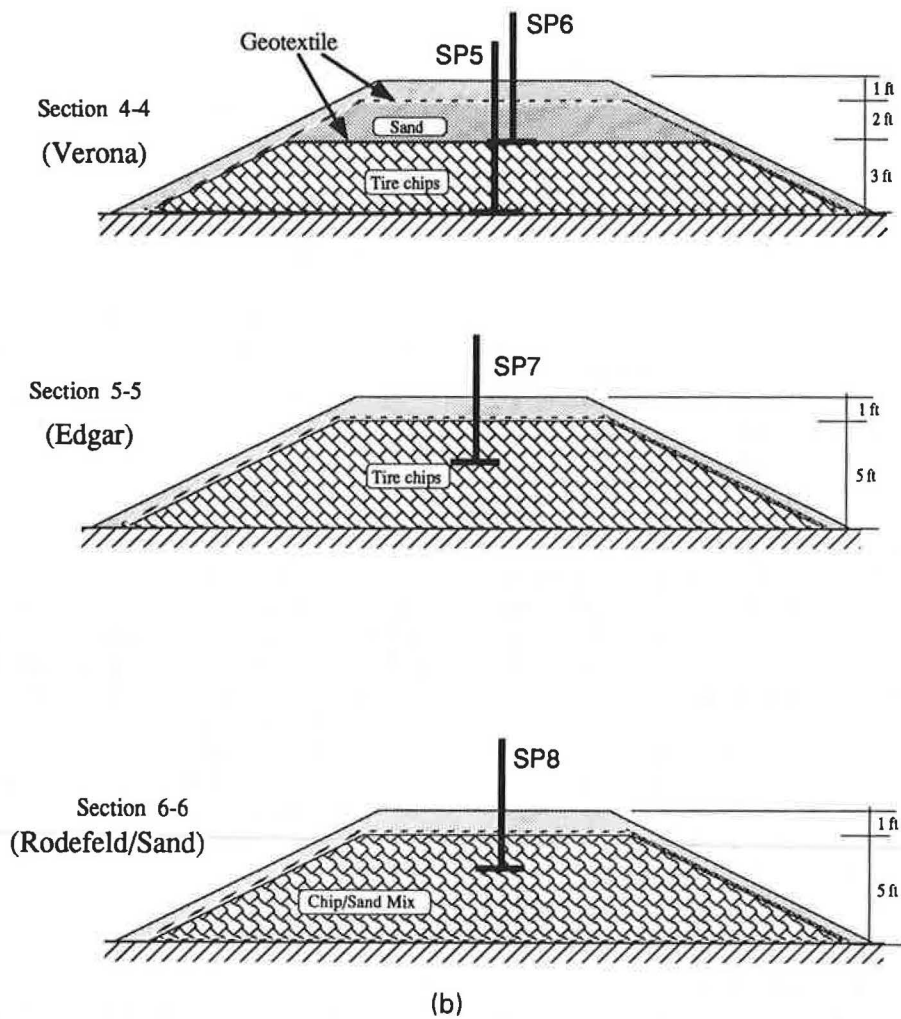


FIGURE 4 (continued)

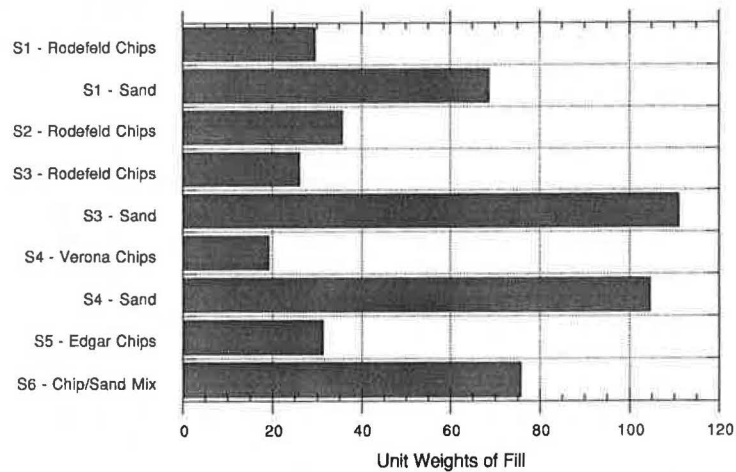


FIGURE 5 Calculated unit weights of fill components.

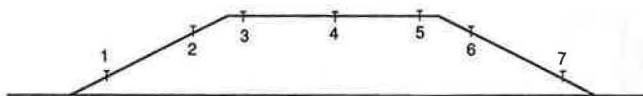


FIGURE 6 Settlement marker locations.

### Roadway Rating

Gravel roads provide service to agricultural, forestry, and recreational areas with fairly high traffic volumes. Surface evaluation and rating of such roads are needed for planning their maintenance and overall management. A system for evaluating and rating gravel roads was developed by the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL) in 1987 (2). A simplified visual method for gravel road evaluation based on the work at CRREL was prepared as a PASER manual by the Transportation Information Center of the University of Wisconsin-Madison (UW) in 1989 (3). The evaluation is based on major factors such as the road cross section, drainage, and adequacy of the gravel layer. Five road conditions are used to evaluate and rate gravel roads: crown, drainage, gravel layer, surface deformation, and surface defects.

Using the PASER system, the test embankment surface conditions were inspected and evaluated after the first year of service on April 5, 1991, by the Wisconsin DOT District 1 maintenance supervisor and UW personnel. The overall condition of the road surface was excellent (a score of 10) for drainage, good to excellent (a score of 8 to 10) for crown, and somewhat variable (a score of 6 to 10) for potholes. This over condition after the initial period of adjustment and repairs puts the test embankment among the better gravel roads. However, there were notable variations in the ratings of each section. For instance, the crown rating was 10 for all sections constructed using earth and for three of the six sections constructed using various tire chip products (Sections 3, 5, and 6). Pothole ratings showed more variation between the various sections constructed of earth or tire chips. Just about every section and the two approaches (built entirely of earth) developed potholes under the heavy garbage truck traffic.

The embankment surface was regraded one final time on April 10, 1991, rendering the rating of all sections excellent (a score of 10). After 7 months of continuous traffic, the embankment surface conditions were again assessed. The ratings were similar to those found on April 5, 1991.

### Surface Settlement

To date, five surveys have been run on (a) the ground surface elevation; (b) the surface markers placed just after the construction was completed, which became buried after additional gravel was applied; and (c) the settlement plates buried in the test embankment.

Three of these surveys were carried out with total stations permitting the collection of three-dimensional data, whereas the other two surveys obtained only the elevation of the surveyed points. The results from these surveys permit a quantitative evaluation of the performance of the embankment.

A review of the lateral movement of the individual markers indicated that there was no apparent bulging of the slopes of any sections or any noticeable longitudinal stretching. The measured lateral movements of the markers placed on the embankment crest (Markers 3, 4, and 5) also indicated relatively small movements (less than 1 to 2 in.) between the initial and final surveys. Consequently, the measured elevations of the embankment crest markers can be used in studying the overall settlement of the embankment section free of local surficial disturbances. The markers were not removed throughout the observation period, and actually the base coarse overlay was excavated to reach the markers during the surveys.

Figure 7 shows the settlement data collected from several surface markers (shown in Figure 6). The data in Figure 7 are the average settlement of Markers 3 and 5, the two surface markers located near the track made by the truck tires as a function of number of days of traffic. The data indicate that the settlement increased rapidly during the first 20 days after truck traffic was first allowed on the embankment, corresponding to the time of major pothole and rut formation. After the surface was regraded and a crushed gravel layer added, the settlement rate tapers off (20 to 60 days). After 60 days, the settlement remains relatively constant. This is further supported by Figure 8, which gives the settlement rates for each of the sections in these time intervals of traffic load.

Using the maximum values of settlement (at 152 days of traffic on Figure 7) as the measure of performance, comparisons of the sections support grouping of the sections as follows:

1. Best performance—Sections EA (east approach), 7, and 8 (Sections 7 and 8 are not shown—they are the fiber-reinforced soils);
2. Higher performance—Sections 3, 4, and 6;
3. Lower performance—Sections WA [west approach (also not shown)], 2, and 5; and
4. Poorest performance—Section 1.

Examination of these groups indicates that the best performance is found in sections composed entirely of soil or fiber-reinforced soil. The exception to this is in the west approach; however, this approach was built initially with frozen soil, which is thought to have contributed to its lower performance. On October 3, 1991, this soil was removed and the west approach was reconstructed. Since then, its performance has been similar to other earthen sections. The next-best-performing group is composed of sections having thick soil caps (Sections 3 and 4) and a section made with a mixture of chips and soil (Section 6). The presence of a thick soil cap is important to reduce the amount of plastic deformation of the chips. The mixture of soil and chips provides performance similar to the pure chip sections with a thicker soil cap; however, the amount of chips recycled in either Sections 3 or 4 is larger than that recycled in Section 6. In addition, the extra operation of soil mixing could be avoided by constructing roads similarly to Sections 3 or 4. The sections made of pure chips that did not have the thicker soil cap did not perform as well, most likely because of the lack of confinement of the surface tire chips. The worst-performing section was the layered section. The performance can be traced back to the lack



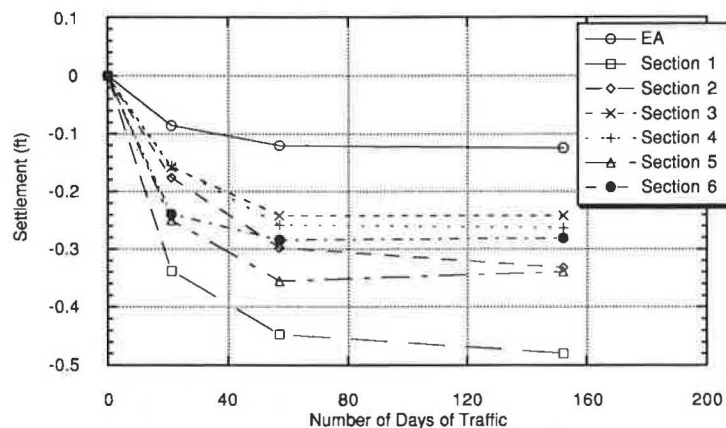


FIGURE 7 Tire track settlement versus days of traffic.

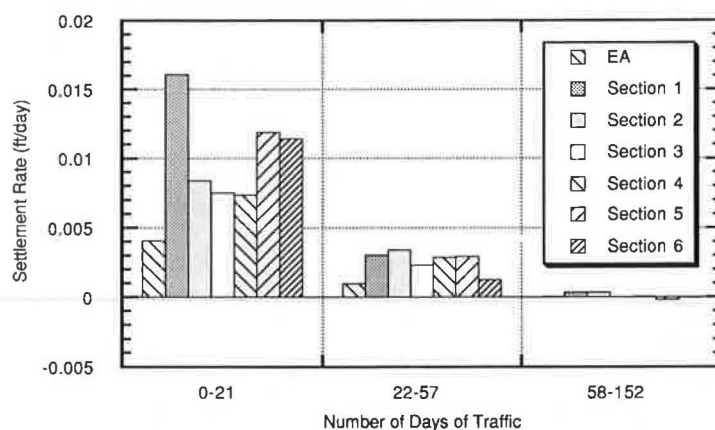


FIGURE 8 Tire track settlement rate versus days of traffic.

of compaction imparted to the soils (see Figure 5) in that section as well as the near-surface tire chips without adequate confinement.

### Compressibility of Deep Materials

The settlement of the surface markers depicts the plastic deformation associated primarily with the surface materials (crushed stone, gravel, and some tire chips), where the stress increase due to the traffic loading is the largest. The movement of the deep settlement plates describes the response of the deeper materials (chips, chip-soil mixture, or layered chips-soils) to smaller stress increases from traffic loads. A comparison of the observed surface marker settlement with the movement of the deep settlement plates indicates a much-reduced response of the deep settlement plates, though the trends are similar: higher initial movement compared with the period after 60 days. Table 1 summarizes an analysis of the stiffness of the deeper materials as measured by the movement of the deep settlement plates. The stiffness index is defined as the ratio of the overburden stress to the plastic strain. A comparison of the stiffness differences between the sections

supports the previous performance grouping based on surface settlement. The settlement plate data is useful for further differentiation within each of these performance groups. In addition to the influence of thick soil caps, the data indicate effects of the tire chip void ratio. In Table 1, the bulk unit weight gives a measure of the void ratio of tire chips except for Section 6, where the bulk unit weight is that of the chip-soil mixture. A comparison of SP4 and SP6 indicates that the Verona chips do not perform as well as the Rodefild chips even though they were subject to a higher cap overburden stress.

### Test Embankment Environmental Testing

Waste tires are essentially a solid waste, and the recycling of tires in highway applications will probably require a permit from state or federal environmental regulatory agencies. To obtain an early evaluation of potential environmental problems before construction, duplicate EP toxicity and AFS leaching tests were performed on tire chip samples by the State Laboratory of Hygiene (1). The test results indicate that the shredded automobile tire samples show no likelihood of

TABLE 1 DEEP LAYER COMPRESSION DATA FROM SETTLEMENT PLATES FOR 58 TO 152 DAYS OF TRAFFIC

Section-Settlement Plate	Description	Bulk Unit Weight	Settlement	Layer Thickness	Overburden Stress	Plastic Strain	Stiffness Index
		pcf	feet	feet	psf	%	psf
1-SP1	Layered	29.6	0.156	0.98	294	15.9	1846
1-SP2	Layered	29.6	0.205	1.45	278	14.1	1965
2-SP3	Rodefeld	35.7	0.069	1.92	273	3.6	7594
5-SP7	Edgar	31.4	0.075	1.85	328	4.1	8093
6-SP8	Rodefeld/Sand Mixture	75.7	0.080	2.27	373	3.5	10588
4-SP6	Verona + Cap	19.3	0.098	1.86	565	5.3	10724
3-SP4	Rodefeld + Cap	26.2	0.032	2.02	481	1.6	30375

TABLE 2 WATER QUALITY ANALYSIS OF LEACHATE—EAST LYSIMETER

SAMPLE	Units	5/9/90	3/28/91
pH	su	7.7	7.7
Alkalinity	mg/L	533	705
Barium	µg/L	210	350
BOD	mg/L	14	70
Calcium	mg/L	170	340
Chloride	mg/L	460	1400
COD	mg/L	170	560
Conductivity	µmhos/cm		5150
Iron	mg/L	0.05	0.7
Lead	µg/L	<3	22
Magnesium	mg/L	150	390
Manganese	µg/L	270	3200
Sodium	mg/L	220	200
Sulfate	mg/L	140	450
Total solids	mg/L	2000	4630
Zinc	µg/L	46	560
Hardness	mg/L	1100	2500

being a hazardous waste. Table 2 provides the results of the water quality analyses performed on two samples taken from the east lysimeter established under the tire chip test embankment. Samples were retrieved initially on a monthly basis, quarterly since April 1990, and are retrieved semi-annually now. The data given in Table 2 are typical of the changes observed in the water quality samples obtained.

A review of the data to date support our expectations based on earlier leach tests. The pH is stable around 7.5. Consistent with that pH, most of the parameters stay within acceptable limits. As indicated by the leach tests, there is an elevated manganese concentration in the field samples too, especially in the last samples.

To clarify the possible source of higher manganese concentration in the samples, ground and surface water data from the vicinity of the test embankment were obtained from the Dane County Public Works Department. It is believed that the geological formations here cause the higher-than-usual manganese concentration. The test embankment is located at the foot of the landfill. The volume of water pumped out of the two lysimeters and the elevations of the lysimeters indicate that surface and groundwaters are entering the lysimeters laterally through the slope cover soil. The general characteristics of the water quality at the site are reflected to a certain extent in the measured quality of the lysimeter leachate samples (1).

## CONCLUSIONS

On the basis of the results of the research program, the following can be concluded:

1. Normal construction machinery can be used successfully with tire chips, though rubber tires can be punctured by the exposed wires at the edge of the chips. Vibratory or static compaction does not significantly induce compaction in tire chips.

2. After an initial period of adjustment, the overall performance of a gravel road founded on tire chips appears similar to that of most gravel roads.

3. Tire chips used as a replacement for fill under a road perform better when covered by 3-ft-thick soil caps compared with chips covered by only 1 ft of soil.

4. The void ratio of the pure tire chips affects its stiffness. The void ratio is affected by the size of the tire chip and by the presence of soil within the tire chip voids.

5. Shredded automobile tires do not show any likelihood of being a hazardous waste. Compared with other wastes for which leach test and environmental monitoring data are available, the tire leach data indicate little or no likelihood of shredded tires having adverse effects on groundwater quality.

6. The preceding conclusions support the use of tire chips as a lightweight fill in highway applications if properly confined.

## ACKNOWLEDGMENTS

The investigators are indebted to numerous individuals for their excellent cooperation. In particular, Ken Kosciak, Dennis Norton, Gary Whited, Paul Kozier, Clyde Laughter, and Allan Rommel are acknowledged. The Wisconsin Department of Transportation together with the Wisconsin Department of Natural Resources provided the funding for this project. The Wisconsin Department of Transportation also provided the settlement plates and the survey target markers. Synthetic Industries provided additional funding as well as the geotextiles used in the construction. The Dane County Department of Public Works provided an excellent site for the test embankment. The Dane County Department of Highway and Transportation constructed the test embankment. Finally, the personnel of the Wisconsin Department of Natural Resources, in particular Robert Greffe, provided valuable input for characterizing tires as a waste material and funding for the environmental testing program.

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# Corrosion of Steel Piles in Some Waste Fills

RICHARD P. LONG

On the basis of studies conducted about 30 years ago, steel piles having a significant portion of their length driven below fresh groundwater tables showed only minor losses of metal due to corrosion. Previous investigators attributed the lack of corrosion to global cathodic self-protection of driven piles. Recent construction in Hartford, Connecticut, uncovered piles installed more than 30 years ago. Some showed losses of cross-sectional area greater than 50 percent. These piles were driven through layers of waste materials called miscellaneous fill. The results of an investigation into this unusual corrosion loss are presented. The cause of the corrosion is traced to a macro cell developed between contiguous layers in the miscellaneous fill that overrides the global protection normally found at most pile installations in fresh water. Electrical characteristics of the macro cell are presented. Techniques for detecting areas that might develop these macro cells are outlined, and methods of protecting piles from their effects are discussed.

Previous research indicated that steel piles driven with more than about half of their length below the groundwater table experienced only nominal amounts of corrosion with negligible loss of metal (1-5). Romanoff (1,2) of the National Bureau of Standards concluded that the large area of steel pile below the groundwater table is anodic and develops a weak cathodic protection for the upper portion exposed to oxygen in the fill (2). As a result, designers usually avoided the expense of protective coatings for steel piles and were justified in doing so in most cases (6). Reconstruction of the I-91 to I-84 interchange in Hartford, Connecticut, however, revealed certain fill conditions that caused steel H piles to lose large amounts of cross-sectional area to corrosion during the 30 years since their installation. The research reported here identifies the conditions that led to this unusual corrosion loss and tests to show the aggressiveness of certain fills.

## DISCOVERY OF THE LOSS

The design of the new I-84 to I-91 interchange in Hartford required the enlargement of some existing pier foundations. The construction of these enlarged foundations required excavating beneath the original pile caps, cleaning the exposed portion of the original HP 12 × 53 mild steel piles by sand blasting, driving some new piles to carry the additional load, and forming and placing concrete for a new pile cap to include the old and new piles.

Excavation below the original pile cap at Pier 12 under the I-91 southbound lane uncovered severe corrosion of the piles

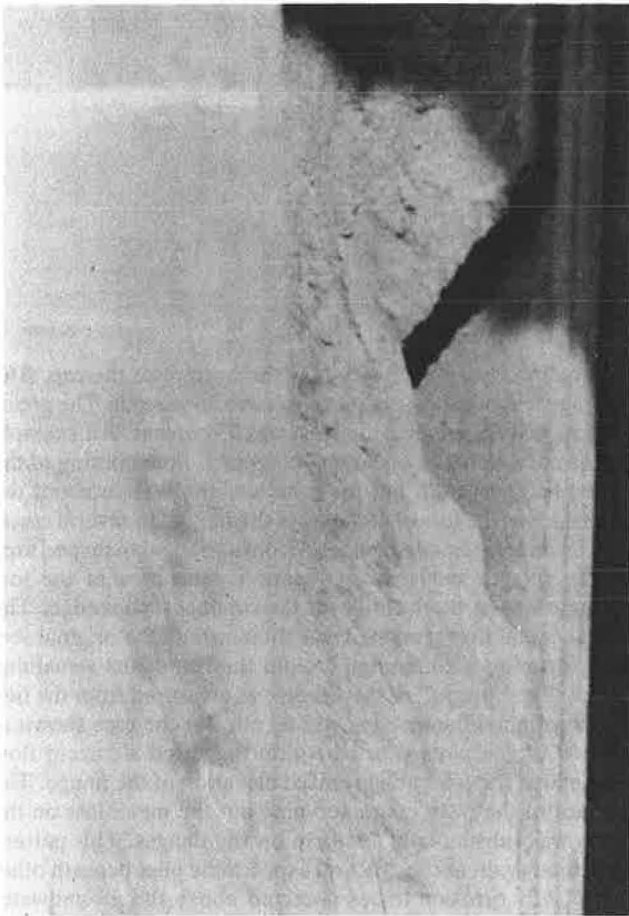
approximately 1 to 1.2 m below the bottom of the cap. The average loss of cross-sectional area was 55 percent. The greatest loss was 65 percent, the least was 35 percent. An example of the corrosion loss is shown in Figure 1. Some pitting of the metal had occurred, but the most striking feature about the damage was the loss of metal from the flange. In several cases, corrosion had removed the steel from a triangular-shaped area in the flange, as shown in Figure 1. The base of the loss triangle was at the location of the original flange edge. The flange metal that remained was thinner than the original section, forming a knife edge around the rim of the remaining metal. The "height" of the triangle as measured from the line of the original flange edge was 60 mm for the case shown in Figure 1. The pattern of corrosion suggested a current flow that exited the piles at a localized elevation of the flange. The surface of the webs contained pits, but the metal loss on the webs was substantially less than on the flanges. This pattern appeared again as construction exposed the piles beneath other piers. All corrosion losses occurred above the groundwater table.

The corrosion loss at Pier 12 occurred in a layer of fill having 94 percent of its particles smaller than the No. 200 sieve. The layers above and below the corrosion loss contained a high percentage of ash and cinder particles. These piles had been driven through a repository of waste materials commonly called miscellaneous fill.

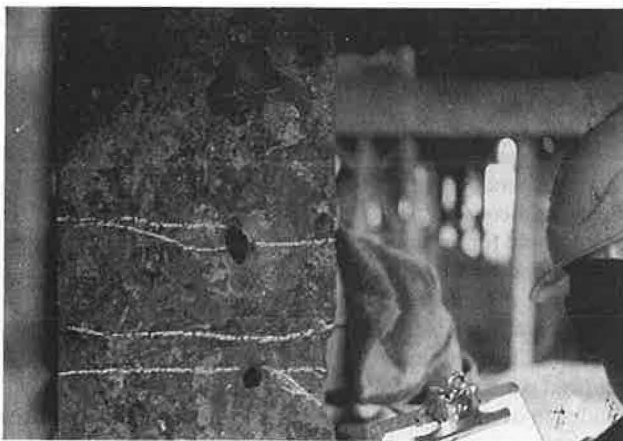
The design of the original pier used a maximum fiber stress of 6 ksi for these piles. The remaining cross sections of the piles were adequate to carry the existing loads. The pier was never in danger of failure.

This interchange is in downtown Hartford, where traffic is heavy and construction could not be delayed. The consultant responsible for design retained a corrosion specialist who recommended coating the new piles with coal tar and attaching sacrificial zinc anodes. This is a treatment often used for pipelines that are completely buried in disturbed soils (7-9). These solutions, usually considered conservative for steel piles (1,2), allowed construction to continue. Measurements indicated no evidence of stray current. The existing piles were reinforced by welding structural shapes to the remaining steel in the piles, and sacrificial anodes were attached. Pouring the new pile cap encased the piles to a depth about 1.5 m below the elevation at which the loss occurred. The research described herein addressed the reasons for the unusual corrosion loss.

Significant cross-sectional losses occurred at several other pile groups within the project. An example of the losses at Pier 7 are shown in Figure 2. The damage in Figure 2 includes loss of metal from the flange edge and some holes corroded through the flanges. Measurements taken on the remaining



**FIGURE 1** Typical corrosion loss, Pier 12, I-91 southbound, Hartford, Connecticut.



**FIGURE 2** Corrosion loss at Pile 5, Pier 7, I-91 southbound, Hartford, Connecticut.

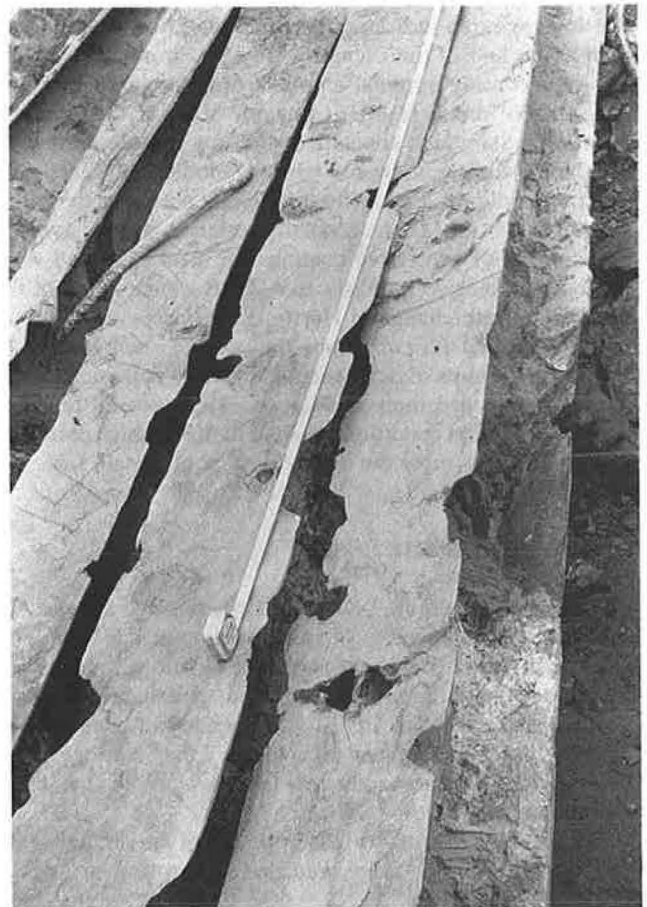
thickness of the piles under Pier 7 indicated losses of cross-sectional area up to 32 percent.

The piles under the existing west wing wall at Abutment 2 had to be removed because they were in the way of the proposed construction. During extraction with an air-activated pile hammer, several of the piles did not have enough cross

section left to allow them to be pulled out of the ground. Damage to the piles under the west wing wall of Abutment 2 is shown in Figure 3, which shows that there was considerable loss on piles that could be removed. As will be shown, this unusual corrosion had to do with the fill surrounding the upper portion of the piles.

Not all the piles had large corrosion losses. For example, at Abutment 2 only the piles under the wing wall had significant corrosion loss. Under the portion of the abutment carrying the roadway, the piles were in excellent condition. Pier 10, about 200 ft south of Pier 12, required a similar enlargement scheme, and the excavation occurred at about the same time as Pier 12. The piles at Pier 10 showed minor losses, although the layering appeared similar. The boring logs for both sites indicated miscellaneous fill above the natural deposits of varved clay and sands. Tests indicated negligible sulphur and pH range between 7.6 and 9.4. The layers below Pier 12, near the corrosion loss, had resistivities below 1,000 ohm-cm (10). The resistivities of the layers below Pier 10 were larger than 1,000 ohm-cm. A few samples were taken for laboratory tests, and a strategy was developed to investigate the metal loss.

Most of the other pile groups had small losses of cross section, usually as surface pits. For example, the piles at Pier 8, surrounded throughout most of their length with natural clay, had only a few minor pits.



**FIGURE 3** Piles extracted from beneath Abutment 2, I-91 southbound, Hartford, Connecticut.



## PREVIOUS RESEARCH ON CORROSION IN SOILS

Corrosion of iron and steel has been the subject of many research studies (11-15). These investigations have found that steel corrodes in the atmosphere, sea water, or disturbed soil at rates requiring protection to remain serviceable. The study by the National Bureau of Standards (NBS), as the National Institute of Standards and Technology was then known, on metals buried in disturbed soils extended over a 30-year period and indicated that the corrosion of metals in disturbed soils correlates with specific properties of the soil-water system, such as soil resistivity, pH, and chloride and sulfide content (11). Results of a recently published study indicate that the most important conditions are electrical resistivity of the soil-water system and the presence or absence of oxygen (12).

Subsequent research by NBS addressed driven steel piles (1,2). These investigations examined steel piling, both H and sheet piling, installed in soils for periods up to 40 years. The results indicated minor damage from corrosion. As part of the study, investigators measured the loss of material from cross sections of the piles. The cross-sectional areas had reductions of only a few percent. The losses did not correlate with any of the conditions such as pH, and chloride and sulfate concentrations were observed to promote corrosion loss in buried metals (2,11). In fact, the prediction of metal loss based on techniques from the NBS study on buried metals were conservative for steel piles in that they predicted much more metal loss than actually occurred (1,2).

Only a few studies on corrosion of steel piles driven in fresh water locations have been conducted since the NBS study (3). The results of these studies confirmed the conclusions of Romanoff (2). The electrochemical models that emerged from these studies are valid for conditions at many sites. In the case of buried steel, the metal is entirely in the fill, and the availability of oxygen promotes corrosion. For driven steel piles, the large portion below the groundwater table gives cathodic protection to the portion in the homogenous fill (2). Obviously, something in the site conditions at the intersection of I-84 and I-91 at Hartford disrupted this pattern.

Research on pipelines noted a tendency for corrosion to occur where soil regions with different characteristics were in contact with metal (16). The corrosion characteristics of this situation, called a macro cell, have been reported (17).

## MISCELLANEOUS FILL

Land in cities has always been valuable. Low areas were often filled to make them more desirable. The filling was usually done with cheap waste materials, which were applied over time and gave the site a subsurface profile having thin layers. The layers may consist of construction debris and rubble, waste soil from excavations, and cinders and ash. Usually the process excluded organic materials, but layers containing organics are sometimes seen.

The sites containing these materials are difficult to describe or quantify with the few words that fit on a boring log. Therefore, drillers in Connecticut came to refer to these materials as miscellaneous fills. The profiles in miscellaneous fills vary from location to location at the same site. Any large structure built on these locations is usually supported on a deep foun-

dation, so few details about these fills were requested. As the excavations in Hartford show, there are conditions existing in these miscellaneous fills that can aggressively attack driven steel piles.

## INVESTIGATION OF THE MECHANISM OF CORROSION IN THE WASTE FILL

### Role of the Macro Cell in Corrosion

Corrosion is an electrochemical process requiring a difference in electrical potential between points on a metal surface (18). The potential difference can be caused by differences in grain orientation, inclusions, imperfections, and other conditions on the surface of the metal (13). It can also be caused by inhomogeneities in the electrolyte, including differences in aeration, temperature, and local depletion or accumulation of chemicals in the soil (13). The potential difference causes a flow of current. If the electrical resistance of the soil-water system is low enough, the flow of current can cause a significant loss of metal. The flow of current takes place in two forms. Inside the metal the current is envisaged as the movement of electrons. In the soil the current is carried by ions moving through the pore water. The most interesting phenomenon occurs at the interface of the steel and the pore water. At the cathode, hydrogen gas forms. At the anode, the passage of current is accompanied by the dissociation of ferrous or ferric ions from the main member, causing loss of metal.

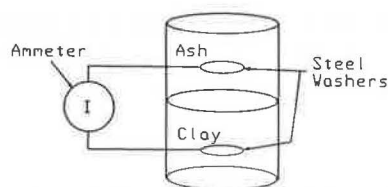
The electrochemical conditions that support corrosion led to the hypothesis of a macro cell in the miscellaneous fill counteracting the global cathodic protection of the piles set up by the groundwater table. This investigation studied the possibility of a macro cell formed by contiguous layers causing the observed losses of metal.

### Possible Causes of the Electrical Potential Difference

In a miscellaneous fill, many conditions capable of developing a difference of electrical potential between contiguous layers can exist. Contiguous layers of cinders and clay, for instance, may contain different amounts of oxygen, forming what is known as an oxygen concentration or aeration cell. There may be different species of chemical constituents present in each contiguous layer producing a similar potential difference. Electrical properties of organisms playing a part in corrosion have also been reported (19,20). A difference in electrical potential between various layers of earth materials, called self-potential, has been observed by geologists and used for the past 100 years to measure the thickness of layers in a borehole (21). A large self-potential is about 0.5 V, but values as large as 1.3 V have been reported (22). None of the mechanisms advanced to explain the cause of these differences in electrical potential is adequate (23,24). However, the method continues to be used satisfactorily despite the uncertainties (25).

Several approaches to reproduce a macro cell in the laboratory were tried. The simplest demonstration used remolded varved clay and cinders from the vicinity of Pier 12. This setup is shown in Figure 4. A layer of cinders 76 mm thick was





**FIGURE 4** Laboratory demonstration of the macro cell using materials from a miscellaneous fill.

placed in an acrylic cylinder 150 mm in diameter over a clay layer of similar thickness. One steel washer was buried in each layer. Connecting the lead wires from the washers to an ammeter completed the circuit. The circuit was interrupted periodically and the open circuit potential measured. The electrical potential was measured at 0.3 V and remained approximately constant. The current began at 1.1 mA and decreased to about 0.6 mA. The sample became dry on two occasions, and the current stopped. Upon rewetting with tap water the flow of current resumed. This demonstrated that contiguous soil layers can develop a macro cell capable of producing enough electricity to cause the observed losses. It also showed that periodic drying and wetting, which may occur in the field, does not destroy the cell.

#### Measurement of Soil Resistivity

The resistivity of the soil is an important property in the study of corrosion. The amount of metal lost is directly proportional to the amount of current passed, and the soil resistivity is important in the control of current flow. The resistivity of soil, cinders, and other particulates of interest must be measured.

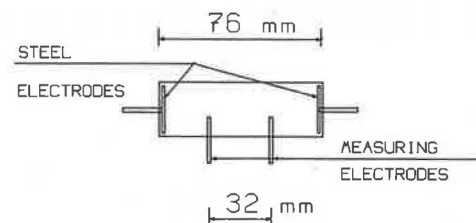
The resistivity of soil is illustrated with the device used to measure it in the laboratory. This device is shown in Figure 5 and is similar to devices used by other agencies (10,26). The soil at the desired water content is placed in a container made of acrylic plastic. All electrodes are made of steel and polished with fine emery paper before each measurement. A known alternating current is passed through the soil, and the potential drop is measured between the pin electrodes placed in the sides of the container. The resistivity of the soil is computed from the following equation:

$$R = \frac{dV}{I} = p \frac{L}{A} \quad (1)$$

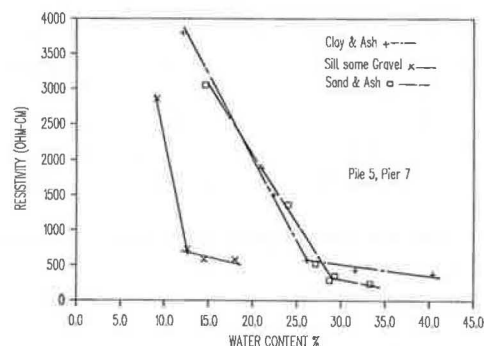
where

- $R$  = the resistance of the soil (ohms),
- $p$  = soil resistivity (ohm-cm),
- $dV$  = measured potential difference (volts),
- $L$  = length of soil (cm) across which  $dV$  is measured, and
- $A$  = cross-sectional area (cm<sup>2</sup>) carrying the current  $I$ .

Typical values of resistivities measured for samples taken from beside Pile 5 of Pier 7 are shown in Figure 6. As can be seen from Figure 6, the soil resistivity is sensitive to water



**FIGURE 5** Test cell for measuring resistivity and contact potential.



**FIGURE 6** Plot of resistivity versus water content for materials sampled from Pier 7.

content of the material. At low water contents, the resistivity of the soils is relatively high. As water is added, the resistivity drops quickly, then remains fairly constant. The low resistivity values of all three of these soils are below 1,000 ohm-cm.

This low resistivity region is of interest. The samples were mixed with distilled water and lightly compacted into the device shown in Figure 5. To ensure that all measurements were made in the low resistivity region, enough water was added so that free water was observed on the surface of the soil sample. The resistivities reported here are all from the low region and are referred to as approximate minimum resistivities. Resistivities of soil samples taken from Pier 7 are given in Table 1 as well as the open circuit potentials developed between contiguous layers. Note from Table 1 that the loss of pile occurred where the approximate minimum resistivities were well below 1,000 ohm-cm and the measured potential was 0.34 V.

#### Measurement of Open Circuit Potential

The appearance of the corrosion damage of the piles indicated that current was being passed between contiguous soil layers and through the piles. Tests indicated that layers near the metal loss had low resistivity. The flow of current also needs a difference of electrical potential. The measurements of potential difference used the same containers as those for resistivity; one-half of the cell was filled with soil or material from one layer and the other half with the material from its contiguous layer. Distilled water was added to both soils until they appeared saturated. The open circuit potential between

TABLE 1 PROPERTIES OF CONTIGUOUS LAYERS SURROUNDING PILE 5, PIER 7, I-91 SOUTHBOUND

Layer No.	Approx. Layer Thickness	Description	Approx. Minimum Resistivity	Potential Difference	pH	Remarks
	(cm)		(ohm-cm)	(v)		
1	50.8	Coarse Sand, Some Silt	4864		8.4	
				0.012		
2	25.4	Red-Brn Silt, Some Gravel	3846		7.5	
				0.045		
3	15.2	Red-Brn Silt, Some Gravel	578		2.3	
				0.34		
4	45.7	Clay w/Coal And Ashes	377		3.8	Greatest Metal Loss
				0.012		
5	30.5	Sandy Silt Some Ashes	238		7.6	
				0.145		
6	38.1	Brown Clay	1578		6.9	

the end electrodes was measured with a high-resistance voltmeter.

## RESULTS AND ANALYSIS

Additional laboratory results are given in Tables 2 and 3. All samples are disturbed. Because the excavations had been open for various lengths of time before the samples could be retrieved, undisturbed moisture contents could not be determined. All the tables indicate soil resistivity close to their minimum values, because this appears to be the best condition for comparing results. Table 1 gives the results for contiguous layers around Pile 5 in Pier 7, the most heavily damaged pile in the group. The thickness of each layer is shown in the second column from the left. The greatest metal loss occurred in Layer 4, where there was low resistivity and high potential difference with Layer 3 above it. The measured pH next to the pile is also given in Table 1. The pH in the vicinity of the metal loss was low but increased with distance from the pile,

rising to about 7.0 approximately 0.6 m from the corroded surface.

Table 2 gives the soil properties beneath the center of Abutment 2. The piles in this section had only minor pitting. The saturated resistivities are high and the potential differences between contiguous layers low except for Layers 5 and 6. No corrosion loss was observed because the layers in this area, being protected from rainfall by the roadway, probably had a resistivity in the field that was greater than the laboratory values reported in Table 2.

Less than 50 m away from the location containing the samples of Table 2 is the wing wall of Abutment 2. The properties of soil from beneath the wing wall of Abutment 2 are given in Table 3. The piles in this area, exposed to both macro cells and deicing salts, experienced severe corrosion. There was no opportunity to see the layering details under the wing wall. The samples for testing were taken from the soil that adhered to the piles at the location of the major corrosion loss and from portions above and below. As can be seen from Table 3, the layers near the metal loss had a low resistivity and a

TABLE 2 PROPERTIES OF CONTIGUOUS LAYERS BENEATH ROADWAY ABUTMENT 2, I-91 SOUTHBOUND

Layer No.	Approx. Layer Thickness	Description	Approx. Minimum Resistivity	Potential Difference	pH
	(cm)		(ohm-cm)	(v)	
1	22.9	Brown Silty Sand	1385		8.1
				0.016	
2	25.4	Brown Coarse Sand	2881		8.7
				0.093	
3	50.8	Cinders	2458		6.7
				0.012	
4	20.3	Clay	659		8.6
				0.020	
5	17.8	Ashes & Silty Sand	1082		8.2
				0.213	
6	15.2	Gray Clay	739		7.1

TABLE 3 PROPERTIES OF CONTIGUOUS LAYERS IN CORROSION ZONE OF THREE PILES FROM THE WING WALL AT ABUTMENT 2

Pile No.	Location in Corrosion Zone	Description	Sat. Resistivity (ohm-cm)	Pot Diff. V.	pH	Concentration		Remarks
						Cl ppm.	SO <sub>4</sub> ppm.	
12	Top	Cinders	1489		4.2	324	185	
12	Middle	Silty Clay	903	0.29	4.6	396	48	Anode
12	Bottom	Ash	408		7.1	257	32	
13	Top	Clay	301		4.8	407	1648	
13	Middle	Cinders	504	0.34	4.4	130	1625	
13	Bottom	Ash	247		4.2	590	1374	
15	Top	Slag	808		5.5	434	82	
15	Middle	Ash	425	0.13	3.2	476	15	
15	Bottom	Clay	2707		5.5	88	33	Anode
-	-	Varved Clay	-	-		24	40	

high potential difference. Also given in Table 3 are the chloride and sulfate levels in the layers at the wing wall piles as well as the values from a remolded varved clay representing background values. Table 3 indicates that the levels of chloride and sulfate under the wing wall were substantially higher than the background found in the clay. One effect of these chemicals is to reduce the resistivity of the soil or cinders by making the pore water more conductive to electricity.

Other combinations of layers were tried in the laboratory to determine the range of potentials that may develop in a miscellaneous fill. Some of these values are given in Table 4. The largest potential difference is between crushed coal and coal ash. Coal, perhaps because it is almost pure carbon, develops the greatest potential with other materials. The data in Table 4 are presented to illustrate that various combinations of particulate materials tend to develop a difference in electrical potential when placed in contiguous layers.

#### MODELING THE CORROSION LOSS

To check the viability of the measured electrical properties causing the observed metal loss, a first approximation macro cell model was developed, as shown in Figure 7. Assuming

TABLE 4 ELECTRICAL PROPERTIES BETWEEN CONTIGUOUS LAYERS

Anode	Cathode	Potential Difference (V)	Samples
Silt	Conc. Sand	0.11	
Clay(lay.6)	Sand(lay.1)	0.14	from Pile 5, Pier 7
Silt(lay.3)	Sand(lay.1)	0.04	from Pile 5, Pier 7
Coal Ash	Coal	1.05	Both passing No.10 Sieve
Coal Ash	Cinders	0.06	
Coal	Cinders	0.82	
Varved Clay	Cinders	0.39	
Conc.Sand	Conc.Sand	0.01	

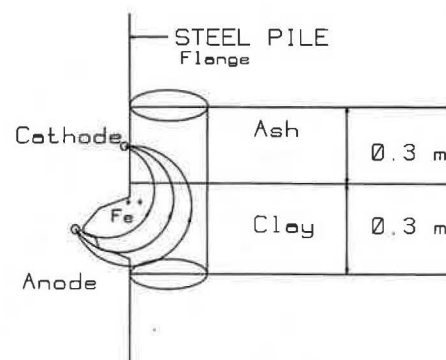


FIGURE 7 First approximation corrosion model of a macro cell in miscellaneous fill.

that the corrosion is caused by contiguous layers 0.3 m thick and that the center of the cathode and the center of the anode are in the center of these layers, the average length of current travel is between these centers, approximately 0.3 m apart. The area through which the current can travel is assumed to be circular and 0.15 m in diameter. A potential difference of 0.25 V between the electrodes and a soil resistivity of 500 ohm-cm allow enough current to pass during a 30-year period to remove nearly half the cross-sectional area. This model demonstrates that a macro cell with these properties is able to cause the observed metal losses.

#### INVESTIGATION OF POTENTIALLY AGGRESSIVE FILLS AND LOCATIONS

Locations whose depth profile contains thin layers, especially layers of ash and cinders interspersed with clay, should be

investigated carefully before designing a deep foundation with steel piles. At the present time the best way to determine whether the layers in a given area will be aggressive to steel piling is to plan a boring program for taking continuous samples and recover a specimen from each of the layers for laboratory testing. The specimens should be tested for resistivity of each layer and electrical potential between contiguous layers.

The corrosion observed here occurred in layers in which the soil resistivity was less than 1,000 ohm-cm and the measured potential difference was at least 0.10 V. These conditions identify aggressive corrosion locations for steel piling.

## CONCLUSIONS

1. Significant corrosion loss can occur from steel piles driven through layered fills, especially fills containing layers of cinders, ash, and clay.

2. The layering in nonhomogeneous fills can cause a macro cell that overrides the normal protection provided the upper portion of the pile by the anodic portion below the groundwater table.

3. The local galvanic cell requires a potential difference of at least 0.10 V and a soil resistivity less than 1,000 ohm-cm to produce the losses observed in this study.

## ACKNOWLEDGMENT

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# Recycled Plastics for Highway Agencies

LAWRENCE L. SMITH AND RICHARD M. RAMER

A description is presented of ongoing research by the Florida Department of Transportation on feasibility studies to use recycled plastic wastes for fence line posts and guardrail posts as mandated by a 1988 state law. All test work focused on acquisition of data for selected essential properties and characteristics, including durability; resistance to insects, soil microorganisms, and environmental exposure; water absorption; and flexural and tensile strength. Initial studies entailed property measurements of high-density polyethylene sheeting (milk bottles) used in A-frame barricades. Low strain rate tests produced failure at a strain of approximately 1.0 for flawed material but did not fail for flaw-free material at a strain of 2.6. Large-diameter posts showed major variations in composition and properties. Exposure tests showed warpage on posts with small cross sections. Flexure to failure whole post tests indicated that some posts were stronger than concrete and were possible candidates for guardrail posts. Resistance tests for soil microorganisms showed no effect. Boiling water and boiling saline had no effect on dimensional changes on most post discs. All discs absorbed liquid after 8 to 10 hr of exposure. Insect exposure tests (fire ants and termites) indicate that posts containing cellulosic material may not be suitable in termite-infested areas due to high weight loss. A fence post specification framework is proposed. Other components currently under testing and evaluation are listed.

It has been estimated that the construction, rehabilitation, and maintenance of the nation's highways annually requires 350 million tons of construction materials (natural and manufactured). This includes 20 million tons of asphalt, 10 million tons of portland cement, and 320 million tons of natural aggregates, paving mixtures, coatings, and so forth (1). Many state agencies have developed or are developing and implementing procedures to include a variety of waste materials in construction and rehabilitation processes. This is in response to increasing environmental concerns about solid waste disposal, environmental hazards, and location of new disposal sites.

The generation of large amounts of solid waste has created much concern at local and state government levels in Florida. The situation is aggravated by Florida's sensitive environment, vulnerable ground water supplies, and continuing need to find additional landfill capacity (2).

In 1988, the Florida legislature passed a comprehensive solid waste management bill (Senate Bill 1192) (3), which directed the Florida Department of Transportation (FDOT) to expand, where feasible, its use of recovered waste materials in highway programs. The legislation directed FDOT to initiate research using available expertise in the Florida state university system and to build demonstration projects for de-

termining the feasibility of using various solid wastes in construction projects. If use of these materials was found feasible, the FDOT was to develop appropriate specifications requiring the use of such waste materials.

The Florida legislation addressed, in part, the following specific waste by-products and applications for use in road construction:

- Recycled oil products,
- Ground rubber tires in asphalt pavement,
- Glass and glass aggregates,
- Recycled steel for rebar and I-beams,
- Recycled mixed plastic materials for guardrail posts and fence posts,
- Fly ash (Class F) in portland cement concrete construction, and
- Fly ash in soil stabilization.

The intent of the legislation was to promote waste by-product use to reduce the environmental impact of disposing of these wastes in Florida landfills. Current waste disposal practices encompass the potential for air and water pollution, health hazards, lack of suitable disposal sites, collection and disposal costs, and the loss of resources that may be economically and beneficially recovered through recycling (4).

Florida currently generates 15.8 million tons of municipal solid waste (MSW) annually. Of this amount, 8.8 percent, or 1.4 million tons, represents plastic wastes available for recycling. In 1988, only 1 percent of all plastic wastes was recycled. Despite this low figure, the plastics industry has pointed out that there is no technical reason preventing the recycling of plastics in the MSW stream (5). This paper focuses on plastic recycling in Florida and the potential and current use of recycled plastics by FDOT.

As a result of the 1988 legislation, recycling plastics became a popular topic. However, the effective use of recycled plastics requires that the plastics' property changes occurring because of the recycling be documented. Commingled (mixture of waste plastics) postconsumer plastic scrap is currently melt-extruded into post and board shapes. FDOT is investigating the possible use of recycled commingled plastics for highway applications such as guardrail posts and fence posts. These applications require that the range of mechanical properties be known so that specifications can be established. It is suspected that the commingled plastic materials may be flawed and have inadequate properties for some of these applications. Ultimately, the goal is to acquire reliable data to develop specifications for procurement of the materials through construction contracts. An initial maintenance contract to supply 5,000 fence posts is in progress.



Table 1 gives typical plastics currently in use and their applications. The table includes the recycle symbol and recycle code number. There are three major types of plastics most easily collected and recycled at this time: polystyrene (PS), polyethylene terephthalate (PET), and high-density polyethylene (HDPE). These three make up approximately 50 percent of the available recyclable plastics (2).

No testing methods or testing protocols existed at the beginning of this research effort. Also, it was recognized that if recycled plastics were to be used within a reasonable time, accelerated testing of some sort would be required. This can be accomplished by using the accepted principles of time/temperature superposition (e.g., correlate excessive heating for brief periods with longer-term ambient temperature exposure). Such correlations may require several years. Test data would have to be used to predict behavior over time (without available long-term performance data). Since legislation mandated the investigation of recycled plastic fence and guardrail posts, characteristics and properties deemed essential to predict real-world performance for these items were as follows:

- Flexural strength,
- Tensile strength,
- Insect resistance (fire ants, termites),
- Soil microorganism resistance,
- Water absorption,
- Fire resistance,
- Exposure resistance to the hot and humid Florida environment, and
- Accelerated aging (oven test).

Plastic photo degradation due to ultraviolet exposure from sunlight was not expected to cause concern. Two years of post exposure have apparently justified this early decision. The initial specification for fence posts has been written addressing this concern with manufacturers.

## PRELIMINARY RESEARCH

FDOT sponsored research (6) initially focused on the stress-strain property evaluation of extruded sheets produced from recycled HDPE milk bottles. The sheets were produced by waterbath cooling of melted HDPE passed through rollers. This testing helped establish baseline parameters and guidelines for further evaluations and property measurement reproducibility. These materials are currently used commercially in Florida by two manufacturers of A-frame highway barricade panels.

The recycling process for plastics has the potential for incorporating many contaminants in the recycled plastic products. The contaminants may act as flaws and thus adversely affect the mechanical properties of the recycled plastic product. Clearly, such variations in properties will affect the specifications that can be established for any application. The use of single feedstock material and a continuous manufacturing process suggested that this recycled product had properties close to virgin plastic.

Stress-strain tensile testing indicated that property variations at deformation strains up to the yield point (a strain of approximately 0.2) within one batch of samples is less than 10 percent, and the mean property variations between batches are small (4 percent). However, the presence of dirt and other contaminants affects the failure behavior in a significant fashion. For low strain-rate testing (0.2 to 2 in./min), failure occurred at a strain of approximately 1 for the specimens containing flaws, whereas the specimens that are nearly flaw-free did not fail at a strain of 2.6 (Figure 1). These early efforts suggest that ultimate control for extruded recycled plastics must be built into the quality control scheme.

Nearly all recycled plastic post suppliers are using an unvented batch extrusion process resulting in posts with large voids (Figure 2). These voids (plus particles of metal and unmelted polymer) act as flaws and reduce the mechanical strength. Preliminary stress-strain testing indicates that these

TABLE 1 TYPICAL PLASTICS AND APPLICATIONS

Materials	Recycle		Products
	Symbol	Code #	
Polyethylene Terephthalate	PET	1	Computer and Video Tapes, Soda Bottles (2 Liter)
High Density Polyethylene	HDPE	2	Milk Bottles, Bleach Bottles
Vinyl	V	3	Water Pipe, Hose, Floor Tile
Low Density Polyethylene	LDPE	4	Supermarket Bags, Films
Polypropylene	PP	5	Soda Bottle Caps, Cassette Boxes, Battery Cases
Polystyrene	PS	6	Fast Food Containers, Utensils
ABS, PMMA, Nylon, etc.	Other	7	Toys, Contact Lenses, Rope

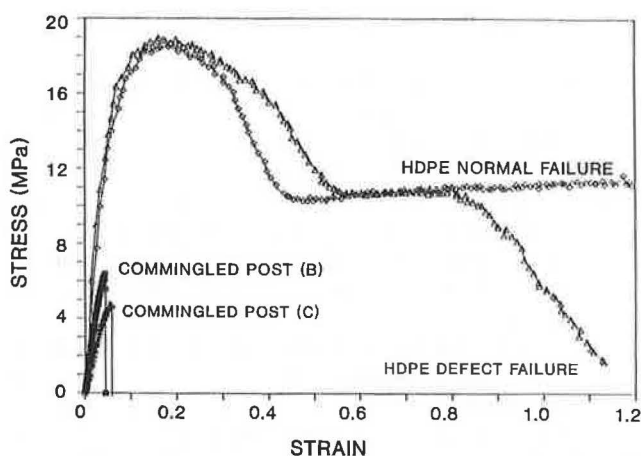


FIGURE 1 Stress-strain curve for industrial HDPE and commingled plastics.

commingled materials are more brittle than the early HDPE tests (100 percent milk bottles), as expected (Figure 1). Properties of samples taken from different portions of the posts were evaluated to determine variations due to nonuniform processing conditions. Many posts have been installed for the past 2 years at FDOT for outdoor weathering evaluation (Figure 3). It is obvious from Figure 3 that the cross-sectional size and uniformity of composition (after extruding) has an effect on long-term warpage resistance (i.e., the smaller the cross section, the greater the likelihood of warpage with age and exposure). At this time, however, post height does not appear to affect warpage within the range of post lengths being considered for use by FDOT.

### RESEARCH IN PROGRESS

Flexural testing using full-size recycled posts indicates that failure loading for certain recycled plastic posts would be greater than for concrete posts of the same size (Table 2).

This suggests that large-diameter posts may be suitable for use as guardrail posts. Ultimately, after additional laboratory tests, FHWA-mandated impact tests and crash tests will be conducted. Posts passing the crash test will be permitted for use in Florida. On the basis of preliminary whole post flexure tests, it appears that a certain amount of post void content may be desirable to help distribute the momentum of the vehicle impact.

To simulate exposure to soil microorganisms,  $\frac{1}{8}$ -in.-thick discs cut from the posts were weighed and measured, then placed at ambient temperature in five terraria each containing 25 lb of one of five unanalyzed soil types common to Florida. After being buried for several months under normal room humidity, the discs were reweighed and measured. Results indicate no mass loss or dimensional change, suggesting that soil microorganisms apparently have no effect on the posts.

Since some fence posts will probably be exposed to various water systems (brackish and fresh), it was necessary to ascertain the absorptive properties of the posts under accelerated conditions. Thus,  $\frac{1}{8}$ -in. post discs (including a wood comparison) dried, weighed, and measured, were immersed

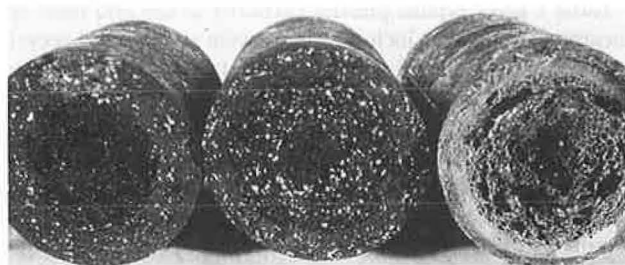


FIGURE 2 Voids and skinning effects in 6-in.-diameter posts influence the strength of the posts.

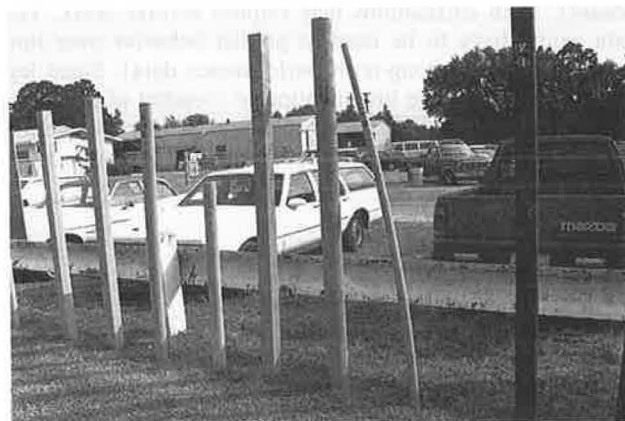


FIGURE 3 Two-year exposure test at project inception.

in boiling deionized water for up to 600 min and in boiling 5 percent saline in deionized water under pressure (to maintain constant concentration) for 500 min. The percentage changes in disc diameter (dimensional) and weight uptake are given in Table 3. It is obvious that in boiling water, dimensional changes are within acceptable limits, although Post D exceeded absorptive values for the wood disc. This is surprising but not unexpected, considering that Post D contains 50 percent sawdust. It was hoped that the plastic would act as an encapsulating agent for the sawdust and inhibit water uptake. This obviously was not the case as indicated by the high water uptake compared with the uptake of the other posts. Post D, however, is still one-ninth that of the wood control (E). In the boiling saline tests (to simulate brackish water), dimensional changes were relatively small, although Post D again gave values greater than the wood control. In terms of liquid uptake, weight changes were generally greater in boiling saline than in boiling water except for Post A. Post D again absorbed more than twice as much liquid as the other posts but was well below that of the wood. This increased water uptake may also affect insect infestation, which seems to be actuated by the addition of water.

Since the plastic posts will be exposed for very long periods to environmental conditions common to Florida, it was necessary to measure the resistance to insect attack. By design, none of the posts contains any insecticides or treating chemicals (e.g., CCA) used for wood protection. Tests were run

TABLE 2 ULTIMATE FLEXURAL STRESS OF RECYCLED PLASTIC SAMPLES FROM DIFFERENT PRODUCERS

SAMPLE CODE NAME	SHAPE	DIMENSIONS (IN)				EFFECTIVE MOMENT OF INERTIA (I) (IN <sup>4</sup> )	SECTION MODULUS (S) (IN <sup>3</sup> )	MOMENT (LB.IN.)	ULTIMATE FLEXURAL STRESS PSI
		OVERALL		CORE					
		T <sub>O</sub>	W <sub>O</sub>	T <sub>C</sub>	W <sub>C</sub>				
B2-E	R	3.5	7.5	---	---	26.8	15.3	44,637	2,915
B2-C	R	3.5	7.5	1.5	5.5	25.3	14.4	65,593	4,546
C1-C	C	6	6	4.5	4.5	43.5	14.5	58,981	4,069
C14-E	C	6	6	3.5	3.5	56.3	18.8	75,641	4,034
E7-C	R	5.5	7.5	3.5	5.0	86.1	31.3	97,717	3,120
E7-E	R	5.5	7.5	4.0	5.5	74.7	27.1	107,578	3,962
E9-C	C	6	6	4	4	51.1	17.0	78,965	4,640
E9-E <sup>+</sup>	C	6	6	4.5	4.5	43.5	14.5	57,188	3,945
BC	S	3.6	3.6	2	2	12.7	7.0	21,403	3,042
CONCRETE	S	6	6	---	---	108	36	29,200	810

- Notes:
1. R = Rectangular, C = Circular, S = Square
  2. For circular samples T<sub>O</sub> = W<sub>O</sub> = Overall diameter and T<sub>C</sub> = W<sub>C</sub> = Core diameter
  3. Length of Sample = 30 in.
  4. I (effective) = I overall - I core

TABLE 3 ABSORPTION TESTS

% DIMENSIONAL DIAMETER CHANGE - 600 MINUTES IN BOILING WATER	
A. * 0.28	D. 4.8
B. 0.29	E. 4.3 (WOOD)
C. 0.53	F. 0
% WEIGHT CHANGE - 600 MINUTES IN BOILING WATER	
A. 4.7	D. 14.8
B. 6.4	E. 122.8 (WOOD)
C. 6.3	F. 2.6
% WEIGHT CHANGE - 500 MINUTES IN BOILING 5% SALINE	
A. 3.7	D. 25.9
B. 9.1	E. 149.4 (WOOD)
C. 7.0	F. 4.1
% DIMENSIONAL DIAMETER CHANGE-500 MINUTES IN BOILING 5% SALINE	
A. 0.3	D. 5.8
B. 0.1	E. 4.1 (WOOD)
C. 0.2	F. 0.4

\*Coded reference for post manufacturers.

for termite and fire ant resistance, since two post producers incorporate cellulosic material. One manufacturer uses sawdust, and the other uses manufacturing-rejected diapers. The latter includes some cellulosic absorptive material obtained during plastic liner recovery. The fire ant tests were run at the United States Department of Agriculture (USDA) Insect Labs in Gainesville, and the Formosan and Eastern termite tests were performed at the University of Florida Research and Education Center in Ft. Lauderdale (6). Both test sets exposed dried, weighed 1/8- to 1/4-in.-thick discs cut from the

posts to each test organism for 2 weeks. The tests were run in triplicate in open containers designed to keep the insects within the container. At the end of the test cycle, the discs were again dried and weighed. Any weight loss was expressed as a percentage. The results in Table 4 suggest that Posts C and D may prove unsatisfactory in a termite environment. Posts A and B also showed weight loss that may be cause for concern.

To date, fire ants have not shown any use for the materials as a food source and do not pose a threat in this respect, but Dr. Wojcik (USDA) has pointed out that fire ants may choose

TABLE 4 TERMITE DAMAGE

SAMPLE	R AVERAGE WEIGHT LOSS (G)	F AVERAGE WEIGHT LOSS (G)	PROJECTED PERCENT WEIGHT LOSS/YR
A	0.054	0.074	15.3
B	0.065	---	8.3
C	0.182	0.335	47.5
D	0.419	0.469	65.2

R - Eastern subterranean termite (*R. flavipes*).

F - Formosan subterranean termite (*C. formosanus*).

Unspecified - not distinguished between F or R.

Projected weight loss is for F.

to inhabit them. It may be necessary to add an infestation inhibitor during post manufacture. Insect testing is continuing and will be repeated, in part because of improved manufacturing processing over that used for the 2-year-old posts tested. In all the tests described, several years will be required to correlate real-world exposure tests with laboratory exposure tests.

### STAPLE PULLOUT

As a consequence of the physical property measurements of the posts, it was decided to measure the resistance to removal of fence staples in plastic posts compared with wood posts. In general, it was discovered that the staples hold up to five times more strongly in plastic posts. Moreover, the resistance to removal increases with increasing temperature. This observation and the expected durability of the plastic posts indicate that maintenance and replacement costs could possibly be zero. Hence, cost over the lifetime of the fence post would be reduced.

### SPECIFICATION FRAMEWORK FOR FENCE POST

On the basis of the testing described, a tentative fence specification has been proposed for recycled plastic line posts. The proposed specification will be modified on the basis of field performance of the plastic posts.

Because of the potential for using recycled plastic posts and the current lack of approved (ASTM or AASHTO) test protocols, the post manufacturers will be asked to certify that their products

- Shall be at least as durable as wood posts (35-year minimum lifetime);
- Are composed mostly of waste stream material (70 percent minimum plastic);
- Shall not exceed a specified void content;
- Shall show no chipping, cracking, flaking, peeling, or splintering;
- Shall be resistant to soil organisms and insects, especially termites and fire ants, such that no more than 10 percent loss in strength occurs over the post lifetime;
- Shall not absorb excessive water; and

- Shall contain no chemicals or additives except those designed to inhibit photo or biochemical degradation or insect infestations.

Posts from five manufacturers generally complying with these specifications have been in place for real-world exposure since April 1990 (Figure 4).

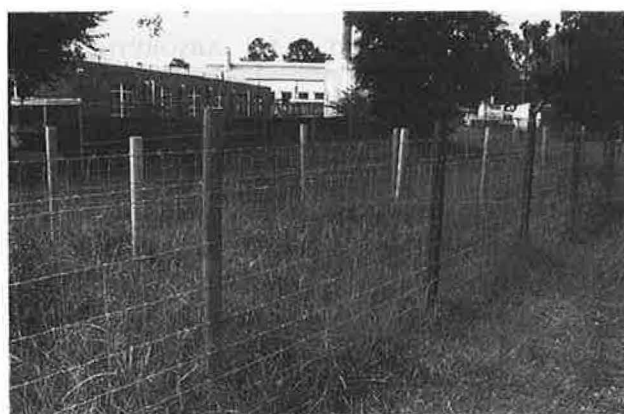


FIGURE 4 View of plastic posts from five producers with attached fencing in Gainesville, Florida.

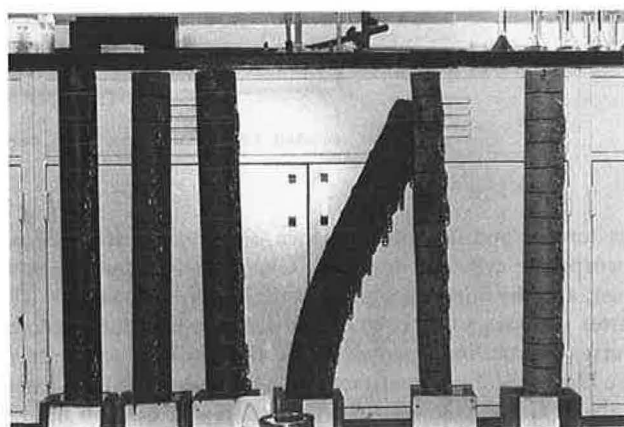
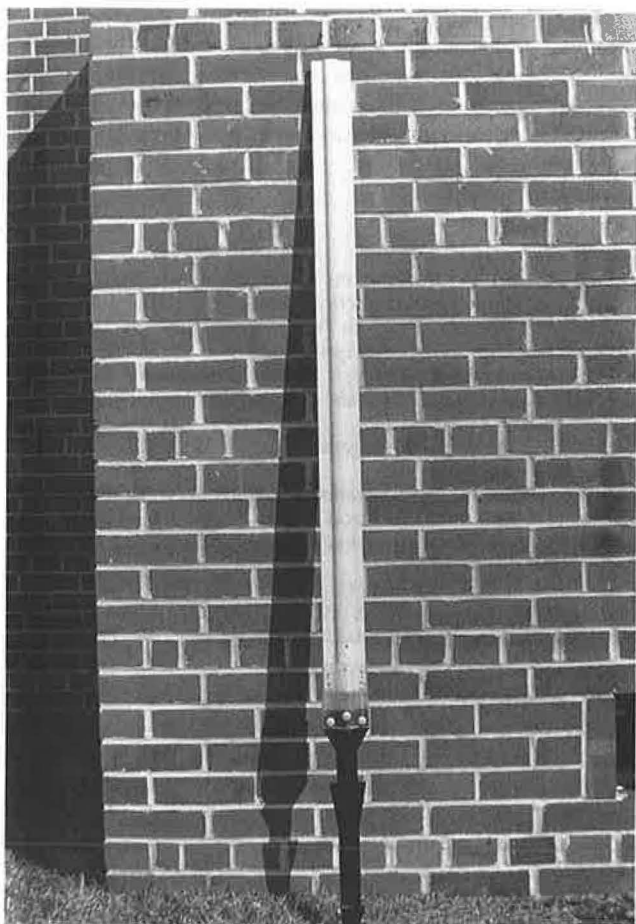


FIGURE 5 Effect of accelerated aging by oven tests. Attached chain is part of staple removal test.





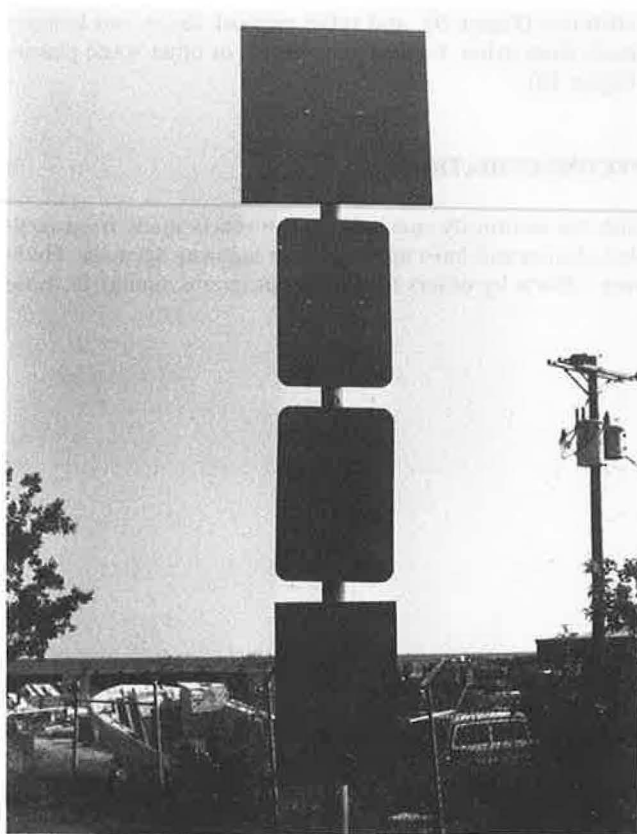
**FIGURE 6** Flexible delineator post approved for limited state use.



**FIGURE 7** Local road containing recycled plastic flexible delineator posts.



**FIGURE 8** Recycled plastic offset blocks after 6 months' exposure.



**FIGURE 9** Some recycled plastic sign substrates exposed in Marathon (Florida Keys) up to 1 year.

For the larger-diameter posts required for fence pull posts and guardrail posts, flexure tests suggest that a certain amount of void content may be beneficial, since pull posts are under tension. A field evaluation of fence pull posts will be initiated without line posts. The pull posts and braces from each manufacturer will be spaced about 10 ft apart and strung with

wire. Any residual stresses in the posts should be manifested by warpage due to fence tension and the hot Florida sun. Accelerated testing of line post sections at elevated temperatures (oven test) gives some insight into this concern (Figure 5). Since these early tests, post quality has improved to the point where high-temperature warpage (260°F, 7 hr) no longer





**FIGURE 10** Rebar support chairs approved for Florida use. *Left*, polystyrene foam—1,000-lb capacity; *middle*, another producer's chair made from polycarbonate/ABS; and *right*, conventional chair made from steel that rusts.

appears to be a concern. This may in part result from the new posts containing 70 percent minimum plastic. The earlier posts contained as little as 50 percent.

#### OTHER RECYCLED PLASTIC COMPONENTS

A wide variety of other recycled plastic FDOT components are also under study, including flexible delineator posts (Figures 6 and 7), guardrail offset blocks (Figure 8), sign substrates (Figure 9), and rebar support chairs and bolsters (made from nylon, foamed polystyrene, or other waste plastic) (Figure 10).

#### RECOMMENDATIONS

One can cautiously conclude that products made from recycled plastics will have application in highway agencies. However, efforts by others to develop programs similar to those

of FDOT should recognize environmental differences; for example, cold weather resistance is a problem in northern states, whereas fire ants are a concern in southern states. Eventually, collective data must be used to develop more definite specifications than are currently being used by FDOT.

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# Effect of Chloride and Sulfate Contamination in Soils on Corrosion of Steel and Concrete

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The durability performance of plain and blended cements in sabkha soils (soils contaminated with high concentrations of chloride and sulfate salts) was investigated. Specimens placed in the sabkha soil were obtained at periodic intervals and evaluated by conducting strength loss measurements, corrosion potentials monitoring, and corrosion rate measurements at regular intervals. Results indicate that the strength reduction after 540 days of exposure to sabkha soils was greater in silica fume cement mortar specimens compared with plain and other blended cements. Reinforcement corrosion was, however, much lower in silica fume cement concrete than in plain, fly ash, and blast furnace slag cements. This indicates that use of silica fume cement in structural components placed in sabkha soils will be beneficial from the viewpoint of reinforcement corrosion. Additional protective measures, such as application of a water-resistant epoxy-based coating, will have to be adopted to minimize deterioration related to sulfate salts.

Sabkhas are saline flats underlain by sand, silt, or clay and often encrusted with salt. These subaerial evaporative areas border partially landlocked sea (coastal sabkhas) or cover a number of continental depressions (inland or continental sabkhas). Both types form under hot, arid climates and are associated with a shallow groundwater table. Sabkha soils are generally unconsolidated, heterogeneous, layered or unlayered sedimentological framework bathed in highly concentrated subsurface brines. These types of soils normally have a loose, rather porous, and permeable, gritty structure, and their surfaces are highly hygroscopic (1). These soils typically exist in the form of alternating cemented and uncemented layers. The cementation is brought about by the presence of saturated brines coupled with excessive evaporation rates (2).

Several potential geotechnical problems are associated with sabkha soils (2). The problems emerge principally from the high salt content in the soil and groundwater. The sulfate and chloride concentrations in the sabkha soils are approximately five to six times as high as those in typical seawater (3). The high salt concentrations, particularly those of chlorides and sulfates, are highly corrosive to both concrete and steel. Moreover, the conjoint prevalence of chloride and sulfate salts

poses several unresolved questions regarding their concomitant and interactive effect on the durability of reinforced concrete structural components placed in such an environment. The problem is significantly magnified by the coexistence of moisture with this high salt content, which may lead to crystallization of salts in concrete pores, thereby leading to its disintegration. This form of crystallization-induced disintegration usually occurs above the water table, where crystallization is enhanced by the prevailing high rates of evaporation.

Geotechnical engineers usually investigate the load-bearing and compressibility characteristics of sabkhas and are rarely concerned with the durability performance of concrete in such environments. This unintentional ignorance of the aggressive service environment has led to a substantial reduction in the useful service life of concrete structures. The normal practice in the Arabian Gulf region is to use ASTM C150 Type V cement to resist deterioration due to sulfate salts. The structural components placed in sabkhas, however, are subjected to both sulfate and chloride attack. In such situations, use of Type V cement, though providing adequate protection against sulfate attack, fails to remove free chlorides from concrete. To inhibit reinforcement corrosion, cement with a high tricalcium aluminate ( $C_3A$ ) content should be used (4). Rasheeduzzafar et al. (5) indicated better performance by high- $C_3A$  cements in chloride environments. Although the use of cement with high  $C_3A$  phase, more than 12 percent per se, is useful from a reinforcement corrosion standpoint, it poses the danger of deterioration and disintegration of concrete due to the reaction between sulfate salts and the compounds of cements. A useful approach to solve this problem seems to use a medium  $C_3A$  cement in conjunction with suitable admixtures to decrease the permeability of concrete, thus retarding the ingress of the aggressive species (6).

Mineral admixtures like natural pozzolan, fly ash, and industrial by-products like silica fume and blast furnace slag are used in concrete to make it dense and impermeable. Whereas there are considerable data on the sulfate resistance performance of these cements (7–9) and some data are available on the performance of these cements in chloride environments (10–13), data are fairly sparse at this time on the durability performance of these cements in environments characterized by the interaction of chloride and sulfate ions. In the absence of data on the performance of blended cements in the chloride-sulfate (sabkha) environment, a more rigorous approach

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to the selection of appropriate composite binder components is required before their use can be recommended in such aggressive environments.

This investigation was carried out to evaluate the performance of three types of ASTM C150 cements (Types I, II, and V) and Type I cement blended with fly ash, silica fume, and blast furnace slag in the sabkha soils. The plain and blended cement concrete and mortar specimens were made with water-cement ratios of 0.35 and 0.50. The cement mortar specimens were buried in sabkha soils and retrieved after 3, 6, 12, and 18 months of exposure. The performance of plain and blended cements in the sabkha environment was evaluated by determining the reduction in the compressive strength and analyzing the crushed mortar specimens to measure chloride and hydroxyl ion concentrations. The reinforcement corrosion was evaluated by conducting accelerated laboratory studies using simulated sabkha groundwater. The corrosion activity was monitored by measuring corrosion potentials and corrosion current density at regular intervals.

## MATERIALS AND TECHNIQUES

### Materials

ASTM C150 Type I, Type II, and Type V cements were used in preparing plain mortar specimens. ASTM C618 Class F fly ash was used as 20 percent replacement by weight of Type I cement to prepare portland fly ash blended cement. For the silica fume mortar specimens, silica fume was used as 10 percent replacement by weight of Type I cement. In addition, one blast furnace slag cement, containing 60 percent granulated blast furnace slag, was used. A sand-to-cement ratio of 2.75 was used for mortar specimens, whereas a cement content of 350 kg/m<sup>3</sup> was used in the reinforced concrete specimens used for evaluation of steel corrosion. The coarse aggregate was 19 mm maximum size crushed limestone of bulk specific gravity 2.42 and average absorption 3.77 percent. A coarse to fine aggregate ratio of 2.0 by weight was kept invariant in all the concrete mixes.

### Techniques

#### *Casting of Specimens*

Mortar cube specimens 25 mm in size were used to monitor the effect of sabkhas on compressive strength. Concrete cylinders 75 mm in diameter and 150 mm in height, with a centrally embedded 12 mm reinforcing bar were used to investigate reinforcement corrosion. The steel bar was coated with an epoxy paint at the concrete-air interface and at its end, which was embedded in concrete. An effective length of 75 mm of uncoated steel was enclosed in concrete. The concrete ingredients were mixed in a revolving mixer, whereas mortar was mixed in a mortar mixer. After casting, the specimens were covered with wet burlap for 24 hr before demolding.

After demolding, the specimens were cured in potable water for a further period of 13 days. The field specimens were placed in plastic cages, which were placed below the groundwater table. The placement of the specimens was properly documented, and the pit was refilled with the native soil. For

investigating reinforcement corrosion, three reinforced concrete specimens representing similar mix constituents were immersed in the simulated sabkha solution (15.7 percent Cl<sup>-</sup> + 0.5 percent SO<sub>4</sub><sup>2-</sup>). Sodium chloride was used to provide the chloride ions, and sodium sulfate and magnesium sulfate were used to provide the sulfate ions. The latter two salts were proportioned to provide 50 percent of the sulfate concentration from each of them.

#### *Reduction in Compressive Strength*

Deterioration due to exposure to the aggressive environment of the sabkha soils was documented by evaluating the reduction in compressive strength. Mortar specimens (25-mm cubes) were tested in compression after 3, 6, 12, and 18 months using a special 200-kN automatic machine suited for testing small cubes. At the scheduled time, the specimens were retrieved from the exposure site and rinsed in distilled water for about 15 min to remove any salt deposits and loose materials. They were air dried for a period of about 24 hr and tested in compression according to ASTM C39.

#### *Chemical Analysis*

After retrieval from the field, the specimens were cleaned of the salt deposits, air dried, and crushed to fine powder (passing #100 sieve). The crushed portion was used to determine the following: (a) water soluble chlorides, by titrating against mercuric nitrate (14), and (b) hydroxyl ion concentration, by dissolving 10 g of the powder sample in 500 mL distilled water and titrating the filtrate against 0.01 M sulfuric acid.

#### *Corrosion Monitoring*

Reinforcement corrosion was monitored by measuring corrosion potentials and corrosion current density at regular intervals. The corrosion potentials were measured using a high-impedance voltmeter and recording the potentials with respect to a saturated calomel electrode (SCE). Linear polarization resistance technique was used to measure the corrosion current density. The polarization resistance ( $R_p$ ) was determined by conducting a linear polarization scan in the range of  $\pm 10$  mV of the corrosion potential. A microprocessor-based potentiostat/galvanostat was used for polarizing the steel. A stainless steel frame placed outside the specimen was used as a counterelectrode, whereas an SCE was used as a reference electrode. A scan rate of 0.1 mV/sec was used. Positive feedback technique was used to compensate for the ohmic drop (IR) between the reference electrode and the reinforcing bar. The corrosion current density  $I_{\text{corr}}$  was determined using the Stern Geary formula (15):

$$I_{\text{corr}} = B/R_p$$

where

$$\begin{aligned} I_{\text{corr}} &= \text{corrosion current density } (\mu\text{A}/\text{cm}^2), \\ R_p &= \text{polarization resistance } (\text{ohms} \cdot \text{cm}^2), \text{ and} \\ B &= (\beta_a \cdot \beta_c) / [2.3(\beta_a + \beta_c)]. \end{aligned}$$

$\beta_a$  and  $\beta_c$  are the anodic and cathodic Tafel constants. For steel in concrete, values of  $B$  equal to 52 in the passive condition and 26 in the active condition are used. Gonzalez and Andrade (16) have demonstrated a good correlation between corrosion rates determined by linear polarization resistance technique and weight loss measurements for active and passive steel in concrete using these values of  $B$ .

## RESULTS

### Strength Development

The effect of the high salinity of the sabkha groundwater and soil on the integrity of the cement mortar specimens was evaluated by measuring the compressive strength after exposure periods of 3, 6, 12, and 18 months. The compressive strength after these exposure periods was compared with the compressive strength before placement in the exposure site. In normal situations, concrete strength generally increases with the period of curing. Any reduction in strength with increasing period of curing indicates deterioration of the internal structure, which could be caused by surface softening or expansion caused by the reaction between sulfate salts and the cement. The compressive strength development in plain cement mortar specimens made with a water-cement ratio of 0.5 is shown in Figure 1. The data indicate that the compressive strength in mortar specimens made with Type I, Type II, and Type V cement continues to be higher than the 14-day value even after an exposure period of about 540 days. The data on strength development in cement mortar specimens made with Type I and Type V cements and a water-cement ratio of 0.35 are shown in Figure 2. These data indicate an initial increase in the strength of these specimens in the range of 120 percent over the 14-day value up to an exposure period of about 3 months. The strength development appears to decrease after this exposure period (i.e., 3 months). After an exposure period of about 540 days, the compressive strength of these specimens was 70 to 90 percent of the 14-day value. The reduction in strength in specimens made with a water cement ratio of 0.35 may be attributed to the reaction between the sulfate ions in the groundwater and the compounds of cement.

The strength development in cement mortar specimens made with a water-cement ratio of 0.5 and blended with fly ash (20 percent replacement of cement), silica fume (10 percent cement replacement level), and blast furnace slag cement (60 percent granulated blast furnace slag and 40 percent Type I cement) specimens is shown in Figure 3. These data indicate an increase in compressive strength with period of immersion for mortar specimens made with fly ash and blast furnace slag cement. The strength development with increasing period of immersion was found to be higher in fly ash cement mortar specimens compared with blast furnace slag cement mortar specimens. In the silica fume mortar specimens, compressive strength reduction was indicated after an exposure period of about 200 days. Figure 4 shows the strength development in cement mortar specimens made with Type I cement and a water-cement ratio of 0.35 and blended with fly ash and silica fume. The data indicate a trend similar to that indicated by the strength data in Figure 3. The strength reduction was again observed to be higher in silica fume mortar specimens compared with that in the fly ash mortar specimens. The strength

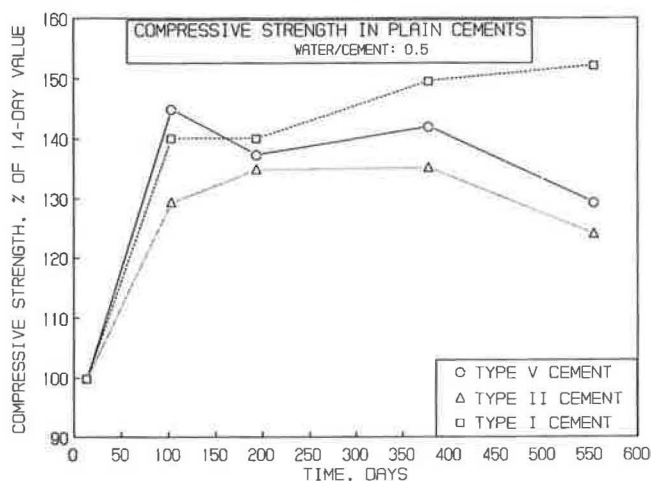


FIGURE 1 Compressive strength development in plain cements (w/c: 0.50).

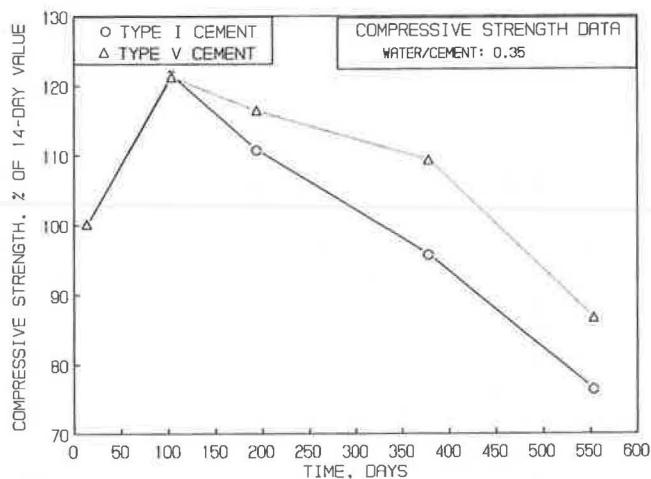


FIGURE 2 Compressive strength development in plain cements (w/c: 0.35).

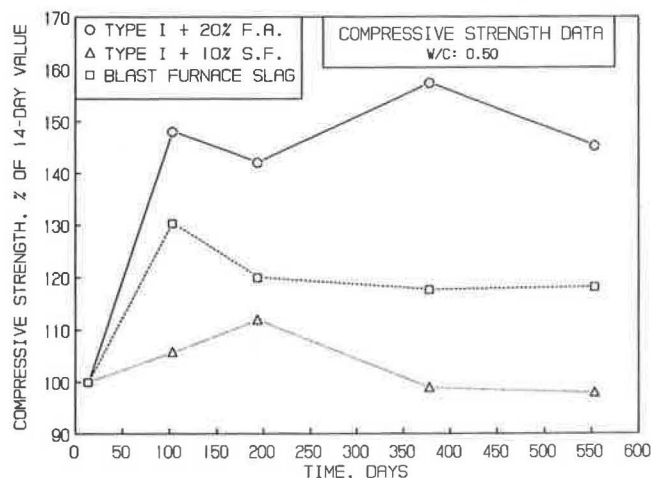


FIGURE 3 Compressive strength development in blended cements (w/c: 0.50; Type I cement).



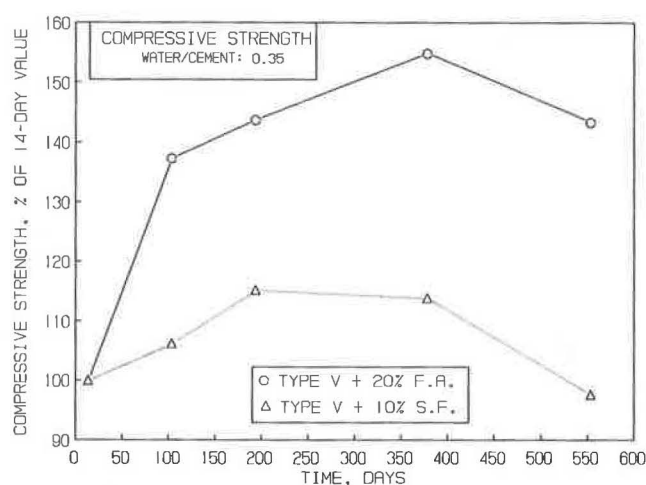


FIGURE 4 Compressive strength development in blended cements (w/c: 0.35; Type I cement).

development data for specimens made with Type V cement and blended with fly ash and silica fume and a water-cement ratio of 0.5 are shown in Figure 5. These data indicated a trend similar to that shown in Figures 3 and 4.

#### Chloride and Hydroxyl Ion Concentration

The data on hydroxyl and chloride ion concentration are presented in Table 1. The  $\text{OH}^-$  ion concentration in the plain cements was in the range of 8 to 10.5 percent by weight of cement. The  $\text{OH}^-$  ion concentration in mortar specimens made by blending Type I cement with silica fume, fly ash, and blast furnace slag cement mortar was 5.54, 6.56, and 6.00 percent by weight of cement, respectively. The  $\text{OH}^-$  ion concentration of mortar specimens made by blending Type V cement with silica fume and fly ash was 4.9 and 5.3 percent, respectively.

The water-soluble chloride content was approximately 6.28 percent by weight of cement in Types I, II, and V cements. This indicates that the effect of cement type was not very significant on water-soluble chloride concentration. The water-

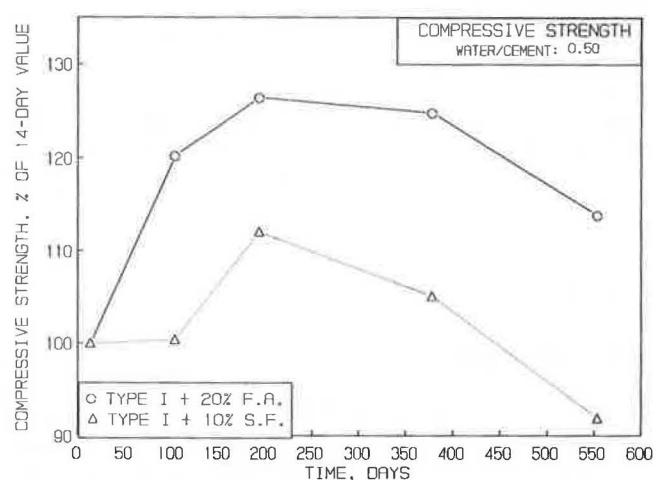


FIGURE 5 Compressive strength development in blended cements (w/c: 0.5; Type V cement).

soluble chloride concentration was 5.36, 4.62, and 5.54 percent in specimens made with Type I cement blended with fly ash, blast furnace slag, and silica fume, respectively. The chloride concentration in Type V cement mortar specimens blended with fly ash and silica fume was 6.28 and 5.73 percent, respectively. The chloride concentration in Type I and Type V cement made with a water-cement ratio of 0.35 was 4.80 and 4.62 percent, respectively. The chloride concentration in specimens made with a water-cement ratio of 0.35 and blended with fly ash and silica fume was 3.70 and 2.59 percent, respectively. The lower chloride concentration in specimens made with silica fume and fly ash and using a water-cement ratio of 0.35 is due to the densification of the paste matrix resulting in the use of the mineral admixtures and the low water-cement ratio. The water soluble chloride concentration in all the specimens is, however, higher than the allowable value of 0.15 percent suggested by ACI Committee 318.

#### Reinforcement Corrosion

Figure 6 is a corrosion potential-time record for plain and blended cements made with a water-cement ratio of 0.50 and for Type I cement with a water-cement ratio of 0.35. The corrosion potentials were measured for three specimens representing similar composition, and the potentials indicated in Figure 6 are the average of these three values. The corrosion potentials provide a qualitative indication of reinforcement corrosion. Potential values more negative than  $-270$  mV SCE indicate initiation of reinforcement corrosion. The corrosion potential-time record (Figure 6) indicates the initiation of reinforcement corrosion in plain and blended cements in the range of 75 to 150 days. The average corrosion potential of steel embedded in silica fume mortar specimens was approximately  $-270$  mV, indicating no initiation of corrosion, even after an exposure period of 450 days. The corrosion potentials of steel in blast furnace slag cement mortar specimens were more than  $-270$  mV at the time of immersion of these specimens in the test solution. This indicates that the ASTM C876 criterion of  $-270$  mV SCE is not useful for detecting the

TABLE 1 HYDROXYL AND CHLORIDE ION CONCENTRATION IN MORTAR SPECIMENS PLACED IN SABKHA ENVIRONMENT

Cement Type	$\text{OH}^-$ ion (% wt. cement)	Soluble Chloride (% wt. cement)	$\text{Cl}^-/\text{OH}^-$ (Ratio)
Type I	9.0	6.28	0.70
Type V	9.12	6.28	0.69
Type I + F.A.	6.56	5.36	0.82
Blast furnace slag	6.00	4.62	0.77
Type I + S.F.	5.54	5.54	1.00
Type II	10.10	6.28	0.62
Type I (w/c: 0.35)	8.74	4.80	0.55
Type V (w/c: 0.35)	10.54	4.62	0.44
Type V + F.A.	6.46	6.28	0.97
Type V + S.F.	4.96	5.73	1.12
Type I + 20% F.A.	5.38	3.70	0.69
Type I + 10% S.F.	5.32	2.59	0.49



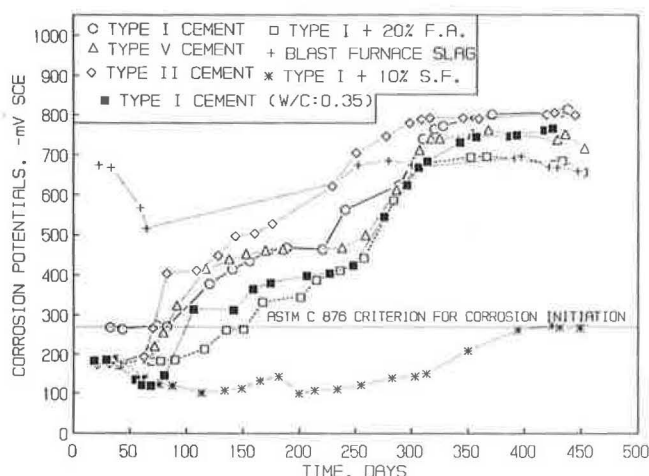


FIGURE 6 Corrosion potential-time record.

onset of corrosion in blast furnace slag cements (17). Another criterion normally used to detect corrosion initiation is a substantial increase in corrosion activity marked by a sudden increase in the corrosion potential-time curve. Such a transition has not been observed in the potential-time curve for bars in blast furnace slag cement concrete. This indicates that reinforcement steel in the blast furnace slag cement concrete is in a passive condition (i.e., corrosion has not yet begun).

The data on corrosion current density for bars embedded in specimens tested in this investigation are shown in Figure 7. The corrosion current density was 3.0 to 3.87  $\mu\text{A}/\text{cm}^2$  in Type I, Type II, and Type V cements (w/c: 0.5). The corrosion current density on bars in fly ash blended cement concrete was about 1.04  $\mu\text{A}/\text{cm}^2$ . The corrosion current density on steel in blast furnace slag and silica fume cement concretes was 0.25 and 0.08  $\mu\text{A}/\text{cm}^2$ , respectively. The corrosion current density on steel in concrete made with a water-cement ratio of 0.35 was 2.17  $\mu\text{A}/\text{cm}^2$ . These data indicate that blending of portland cement with either mineral admixtures or industrial by-products was beneficial from a reinforcement corro-

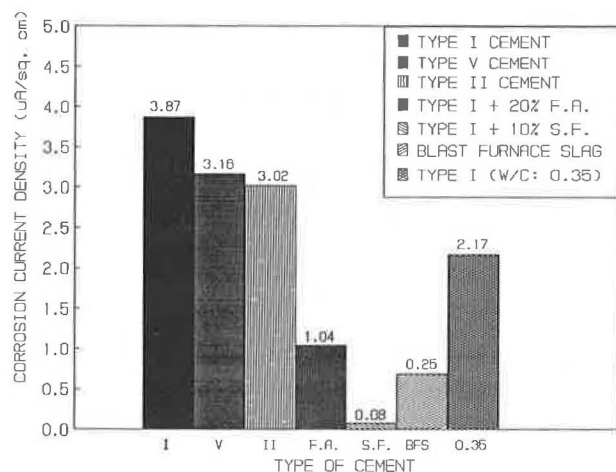


FIGURE 7 Data on corrosion current density on steel in plain and blended cements.

sion viewpoint. Reinforcement corrosion was minimum in silica fume cement concrete, followed by blast furnace slag and fly ash cement concretes in decreasing order. Reinforcement corrosion activity was higher in plain cement concretes than in blended cement concrete specimens. Even an improvement in concrete quality brought about by the reduction of the water-cement ratio from 0.5 to 0.35 did not significantly improve its performance compared with incorporation of the mineral admixtures (fly ash) and industrial by-products (blast furnace slag and silica fume).

## DISCUSSION OF RESULTS

The performance of plain and blended cements in sabkha soils was evaluated by determining strength development, measuring the chloride and hydroxyl ion concentrations, and evaluating reinforcement corrosion. The compressive strength data indicate insignificant reduction in compressive strength with period of exposure in plain cement mortar specimens made with a water-cement ratio of 0.5. The effect of the cement type (i.e.,  $C_3A$  variation) on strength development was not significant. The compressive strength in plain cements made with a water-cement ratio of 0.35 was between 75 and 90 percent of the 14-day value after an exposure period of 540 days. This indicates that specimens made with a water-cement ratio of 0.35 may deteriorate more rapidly than those made with a water-cement ratio of 0.5. Similar results have been reported by Ben-Yair (18). In his experiments, mortar specimens made with a water-cement ratio of 0.3 and exposed to 3.5 percent NaCl + 0.4 percent  $\text{Na}_2\text{SO}_4$  solution for more than 8 years expanded more than specimens made with a water-cement ratio of 0.5. The authors attribute this behavior to the very fine pore structure formed in the low water-cement ratio mixes compared with those made with a water-cement ratio of 0.50. The salts of crystallization formed as a result of reaction between cement and sulfate salts are not well accommodated in the finer pore structure in the low water-cement ratio specimens. These salts exert considerable pressure, resulting in greater expansion and deterioration in the low water-cement ratio mixes than in concretes made with a high water-cement ratio whose coarser pore structure can accommodate the salts of crystallization.

The compressive strength data for fly ash and blast furnace slag blended cements indicated an increase in strength in fly ash and blast furnace slag cement mortar specimens over the 14-day value, even after an exposure period of about 540 days. The strength development was found to be higher in fly ash concrete than in blast furnace slag cement specimens. Mortar specimens made with silica fume blended cements, however, indicated a reduction in strength over the initial values after about 200 days of exposure. The reduction in strength in silica fume cement mortar specimens is attributed to softening of the cement matrix due to reaction of the secondary calcium silicate hydrate (C-S-H) with magnesium sulfate associated with the groundwaters. This sort of deterioration is manifested by softening and deterioration of the surface skin leaving the aggregates exposed to environment (scaling). Such a failure was more prominent in specimens made with blast furnace slag and silica fume cement specimens. Surface scaling was noticeably higher in blast furnace slag cement concretes than

in plain cements in marine exposure studies carried out by one of the authors (19). Schroder and Smolzyk (20) relate the surface deterioration to the glass content of the slag. A minimum glass content of the slag ( $> 61$  percent) was necessary to improve the sulfate resistance of the blend.

The reduction in strength in silica fume cement mortar specimens is attributed to the presence of  $Mg^{++}$  cations associated with sulfate salts.  $Mg^{++}$  cation is destructive to the secondary C-S-H gel by partially converting it to a cohesionless, porous, reticulated magnesium silicate hydrate (M-S-H) gel. The enhanced deterioration of silica fume blended cement paste in the magnesium sulfate environment has also been observed by Cohen and Bentur (21). They reported that brucite (MH) was not traceable in silica fume blended hardened cement pastes that had undergone deterioration due to magnesium sulfate attack. MH is relatively insoluble in water and is known to block the pores of the hardened cement paste matrix, thereby providing a protective shield to the ingress of sulfate ions. In view of this position, it may be hypothesized that the absence of MH in the deteriorated silica fume blended cements, in contrast to its presence in plain cements, presents a more open structure, thereby making silica fume blended cement mortar/concrete more vulnerable to magnesium-based sulfate attack.

The concentration of free chloride (water-soluble chloride) was considerably more than a value of 0.15 percent (by weight of cement) recommended by ACI 318 for concrete structures exposed to chloride-bearing waters for avoiding risk of corrosion. According to Hausmann (22), the risk of corrosion is greater when the  $Cl^-/OH^-$  ratio is above 0.6. The  $Cl^-/OH^-$  ratio in most of the cement/mixture proportions investigated is more than this value (see Table 1 for the values of the  $Cl^-/OH^-$  ratio for each of the mixes investigated). This indicates that the changes of corrosion are elevated in reinforced concrete structures placed in the aggressive environment of the sabkha. The other factor for the corrosion to proceed is the availability of oxygen. The rate of oxygen diffusion is significantly affected by the extent to which the concrete is saturated with water. Since the availability of oxygen is limited in structures placed below ground level, reinforcement corrosion is assumed to proceed at a slower pace, even in the presence of high concentrations of chloride ions. However, at the soil-air interface, where abundant oxygen is available, corrosion proceeds at a rapid rate. This phenomenon appears to be of importance in structures submerged deep in the ground and only a small portion of which are exposed above ground. If these structures are placed in highly contaminated soils, reinforcement corrosion is initiated at the soil-air interface, whereas the buried portion remains free of corrosion because of non-availability of oxygen. The small anode (the soil-air interface) compared with a large cathode (submerged portion) leads to localized corrosion at the anodic sites. Reduction in the diameter of reinforcing steel, in deep foundations placed in sabkha soils, to the extent of more than 50 percent has been observed (23).

The reinforcement corrosion data indicate superior performance by blended cements in resisting reinforcement corrosion compared with plain cements. The corrosion rate of steel in fly ash, blast furnace slag, and silica fume concretes was 3, 12, and 38 times lower than in plain cement concrete. The corrosion rate of steel in blended cements made with a water-

cement ratio of 0.50 was lower than in plain Type I cement ( $C_3A$ : 8.5 percent) concrete made with a water-cement ratio of 0.35. The corrosion current density on steel in Type I cement (w/c: 0.35) was 2, 9, and 27 times that of corrosion current density on steel in fly ash, blast furnace slag, and silica fume cement concrete specimens made with a water-cement ratio of 0.50. Furthermore, the corrosion potential-time record does not indicate initiation of reinforcement corrosion in silica fume cement concrete specimens, whereas reinforcement corrosion was active in all the plain and fly ash cement concrete specimens. The authors believe that the time to initiation of corrosion is an important parameter controlling the service life modeling for structures suffering from reinforcement corrosion. The other parameter used in the life prediction models is the propagation period. The propagation time remains fairly constant in all the cement types. A longer time period for corrosion initiation observed in silica fume cement is indicative of its usefulness in increasing the useful service life of concrete structures compared with other cements.

The corrosion data indicate a superior performance by silica fume blended cement concretes, implying that surface softening leading to a reduction in the compressive strength in this cement does not accelerate the reinforcement corrosion process. Furthermore, the magnitude of deterioration observed in the mortar specimens is probably due to the specimen size. It is possible that the reduction in strength due to sulfate ions in actual structures may not be as high as that observed in the small laboratory specimens. The surface deterioration can be mitigated by applying a water-resistant epoxy-based coating.

## CONCLUSIONS

Mortar specimens prepared using plain cement (Type I, Type II, and Type V), and blended cements (blended with fly ash, blast furnace slag, and silica fume) were placed in sabkha soil for a period of more than 540 days. The performance of these cements was evaluated by conducting compression tests, analyzing the crushed specimens for chloride and hydroxyl ion concentrations, and monitoring reinforcement corrosion. The conclusions from the data developed in this study are as follows:

1. The compressive strength development in plain cements was not affected by placement in the aggressive sabkha soils. The compressive strength measured after about 540 days of exposure to the sabkha environment was not affected in fly ash and blast furnace slag cements, whereas the compressive strength in silica fume blended cements was 70 to 80 percent of the 14-day value.
2. The compressive strength after about 540 days of exposure was 75 to 90 percent of the 14-day value in specimens made with a water-cement ratio of 0.35. The compressive strength in cement mortar specimens made with a water-cement ratio of 0.5 was more than 100 percent of the 14-day value.
3. The water-soluble chloride concentration in all the cement mortar specimens placed in the sabkha was more than the normally accepted threshold value of 0.15 percent by weight of cement. Similarly, the chloride/hydroxyl ion ratio was more

than 0.6 in most of the cement mixtures investigated. This indicates that the chances of corrosion in all the cements placed in sabkha environments are greatly increased.

4. The data on reinforcement corrosion in plain and blended cements developed by measuring corrosion potentials and corrosion current density indicate that the corrosion activity was lower in blended cement concretes than in plain cement concretes. The corrosion current density on steel in fly ash, blast furnace slag, and silica fume concretes was 3, 12, and 38 times lower than in plain cement concrete. The reinforcement corrosion in Type I cement ( $C_3A$ : 8.5 percent) made with a water-cement ratio of 0.35 was more than that in blended cement concretes made with a water-cement ratio of 0.5. The best corrosion protection was offered by plain cements blended with 10 percent silica fume.

5. Data developed in this investigation indicate that blending of cement with silica fume protects the reinforcing steel from chloride-induced corrosion. However, protective measures like coating the exterior surface with a water-resistant epoxy-based coating will be necessary to protect concrete from deterioration due to magnesium-based sulfate salts.

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# Permeability and Leaching Characteristics of Fly Ash Liner Materials

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Rigid-wall column tests are performed at a hydraulic gradient of 100 on 21 compacted mixtures containing a Class F fly ash, sand, bentonite, and cement. Results indicate that combinations of the admixtures can lower the permeability of the fly ash by up to an order of magnitude. There appears to be an optimum percentage of fly ash that minimizes the permeability of fly ash-sand mixtures. Specimens from four mixtures, as well as 100 percent fly ash, are tested at four hydraulic gradients to establish different flow rates. Tap water is used as the permeant fluid, and the effluent is sampled every quarter pore volume of flow and tested for 13 metal concentrations, pH, and electrical conductivity. For most of the metals, most of the metal mass is leached within the first pore volume of flow. However, some metals (Ba, Ca, and Sr) are not leached in significant quantities until two or three pore volumes of flow have passed through the test specimens. These two leaching behaviors suggest that there is dissolution competition between metal ions. No simple trend in leaching rate as a function of flow rate was observed for all mixtures and metals. However, for specific elements between different mixtures at approximately the same flow rate, an increase in the fly ash content tended to increase the leaching rate of most metals. A 2.5 percent cement content was more effective than a 10 percent bentonite content in reducing the leaching rate of metals for mixtures containing the same amount (75 percent) of fly ash.

Fly ash is the finer portion of the noncombustible residue produced during the operation of coal-fired boiler units. It is composed primarily of fine-grained particles ( $< 0.075$  mm in diameter), most of which are glassy spheres, with some crystalline matter, carbon, and scoria. The glassy spheres are relatively inert and immune to dissolution due to their elemental composition and glasslike structure (1). However, on the surface of the spheres, there exist molecules of trace metals that can dissolve in the presence of liquid (1). The potential for leaching of these metals causes concern regarding the environmental consequences of using fly ash.

Despite the self-hardening characteristics that allow fly ash to be used as a soil stabilizer and as an additive in portland cement, only 25 percent of the fly ash produced in the United States currently is used for productive purposes (2), whereas the rest is being disposed of in landfills and surface impoundments. The safe use of fly ash has potential for great economic benefits.

Recently, several studies have investigated the application of fly ash in waste containment liner systems (3-6). Perme-

ability and leaching tests have been performed on mixtures containing fly ash. The studies have shown that permeabilities on the order of  $10^{-7}$  cm/sec can be obtained for stabilized fly ash, though no definite trends of metal leaching were found. None of the previous studies considered the effect of the leaching fluid flow rate on the leaching of metal species from mixtures containing fly ash. The flow rate, which is dependent on field conditions or laboratory procedures, may affect metal leaching. Investigation of the characteristics of metal leaching from fly ash is needed to prove its potential as a liner material for waste containment.

The objectives of this study are to (a) determine if flow rate affects the leaching of heavy metals from fly ash and stabilized fly ash; (b) investigate the effects of sand, bentonite, and cement admixtures on the permeability of fly ash; and (c) determine the effects of these admixtures on the leachability of metals associated with fly ash.

## MATERIALS AND METHODS

### Material Mixtures

For this study, 21 combinations of various percentages of fly ash, sand, bentonite, and cement were chosen. The mixes were grouped into Stage 1 (pure fly ash), Stage 2 (fly ash with sand), Stages 3A and 3B (fly ash with sand and bentonite), and Stage 4 (fly ash with sand and cement). The percentages, by dry weight, of each of the fly ash-soil-cement mixes are given in Table 1.

### Properties of Materials

#### Admixture Materials

Results of specific gravity (ASTM D854), grain size (ASTM D421, D422), and Atterberg limits (ASTM 4318) tests were performed on each of the materials and are presented in Table 2. Also given in Table 2 are results of tests to determine the cation exchange capacity (CEC) of the bentonite and cement. The measurable CEC for cement probably is due to the pozzolanic nature of cement.

The fly ash used in this study was a well-graded Class F pozzolan with the majority of its particles in the silt-size range (0.002 to 0.075 mm). About 28 percent of the fly ash is retained on the No. 325 (0.045-mm) sieve; hence, it is a fine-grained material.

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TABLE 1 FLY ASH-SOIL-CEMENT MIXTURES USED IN TESTING PROGRAM

Stage	Mix #	Percentage of Constituent (by dry weight)			
		Fly Ash	Sand	Bentonite	Cement
1	1	100	0	0	0
2	2	75	25	0	0
2	3	50	50	0	0
2	4	25	75	0	0
2	5	15	85	0	0
2	6	5	95	0	0
3A	7	5	85.5	9.5	0
3A	8	15	76.5	8.5	0
3A	9	25	67.5	7.5	0
3A	10	50	45	5	0
3A	11	75	22.5	2.5	0
3B	12	5	85	10	0
3B	13	15	75	10	0
3B	14	25	65	10	0
3B	15	50	40	10	0
3B	16	75	15	10	0
4	17	5	85.5	0	9.5
4	18	15	76.5	0	8.5
4	19	25	67.5	0	7.5
4	20	50	45	0	5
4	21	75	22.5	0	2.5

TABLE 2 PROPERTIES OF MATERIALS USED IN TESTING PROGRAM

Property	Fly Ash	Sand	Bentonite	Cement
G <sub>s</sub> (ASTM D854)	2.28	2.65	2.60	3.25
USCS Classification (ASTM D2487)	ML	SP	CH	—
Liquid Limit (%) (ASTM D4318)	NP	NP	465	NP
Plasticity Index (%) (ASTM D4318)	NP	NP	425	NP
CEC (meq/100g)	—	0	76	6
% Passing #200	89.0	0	100	100

NP - non-plastic

Property of Nixon Fly Ash *	Value
Percent Retained on #325 (Fineness)(%)	28.2
Silicon Dioxide, SiO <sub>2</sub> (%)	57.0
Aluminum Oxide, Al <sub>2</sub> O <sub>3</sub> (%)	27.1
Iron Oxide, Fe <sub>2</sub> O <sub>3</sub> (%)	5.3
Calcium Oxide, CaO (%)	6.4
Loss on Ignition (%)	0.86
<b>Pozzolanic Activity Index (see ASTM C311)</b>	
Portland Cement at 7 days % of Control	71
Portland Cement at 28 days % of Control	93
Lime at 7 days (psi)	1004

\* Testing by Resource Materials Testing, Inc., of Lakewood, CO

A clean, medium sand identified as 20-40 silica sand was used. It is a poorly graded sand with approximately 95 percent of the particles between the No. 20 (0.085-mm) and the No. 40 (0.425-mm) sieve sizes.

Ecco Gel, the bentonite, purchased from Eisenman Chemical Co., Greeley, Colorado, was warranted to contain at least 81.5 percent sodium bentonite and 4.5 percent free silica.

The cement was a low alkali portland cement purchased from Ideal Cement Co., Denver, Colorado. The cement was a fine-grained material with 100 percent of its particles finer than 0.075 mm.

### Permeant Fluid

Tap water was used as the permeant fluid. Background concentrations of metals in the tap water were determined using inductively coupled plasma optical emissions spectrometry (ICP-OES) (7). This method also was used, in conjunction with extraction procedures, to determine background metal concentrations in the soils, cement, and fly ash. Average metal concentrations of nine water samples were below detection limits or drinking water standards (DWSs) (8) for all metals except Pb. The average background concentration of Pb was 0.07 mg/L, whereas the DWS is 0.05 mg/L. The electrical conductivity of the tap water was 27.9  $\mu$ mhos/cm at 25°C, and the pH was 6.3 at 25°C.

### Selection of Metal Species for Tracing During Column Tests

On the basis of values of leaching potential for 21 metal species, 13 were chosen to be traced during the column tests either because of a high background concentration in the fly ash or DWS (9) consideration. The traced metals are Al, B, Ba, Ca, Cd, Cr, Cu, Fe, Mn, Mo, Pb, Sr, and Zn.

Results from two extraction procedures on each of the materials, using tap water and ammonium bicarbonate diethylenetriaminepentaacetic acid (AB-DTPA), were analyzed to determine the approximate metal concentrations for mass balance analyses.

Metal quantities available for leaching were calculated for the mass balance analyses used during the flow-rate testing phase. Available metal masses were calculated for each admixture constituent. Because neither AB-DTPA extracts nor tap water extracts consistently resulted in the highest concentrations, the maximum concentration value for each metal from either of the two procedures was used in the mass balance analyses. Since tap water was used as the permeant fluid, the percentage of the maximum available mass of a given metal species leached in three pore volumes of flow may be somewhat low.

### Methods

The following is a summary of the methods and equipment used in the testing program. Details are given elsewhere (10).

### Proctor Tests

Five-point Proctor curves were developed using a Soiltest Mechanical Compactor (Model CN-4230). The constituents were mixed by hand until the blend was homogeneous. Tap water was added and mixed before compaction. Virgin fly



ash, cement, and soil were used for each separate Proctor point to ensure that particle deformation due to compaction did not affect densities.

### Permeability and Flow-Rate Tests

The test equipment (Figure 1) uses a pair of plexiglass accumulators on either side of a soil column. The influent liquid is pressurized in the inflow accumulator by means of compressed air acting on an air-water interface. The water flows through the soil column to the outflow accumulator, which is vented to atmospheric pressure. The outflow head stays constant because of the elevated exit tube contained within the outflow accumulator. The inflow head drops as the liquid infiltrates into the soil cell. The test procedure actually models a falling-head situation. However, considering the maximum drop in tap water height and the air pressure applied to the water surface, the gradient changed by less than 2.5 percent during the preliminary permeability tests and by a maximum of 4.2 percent during the lowest gradient flow-rate tests. This was the basis for the constant gradient assumption.

The coefficient of permeability was calculated for each measurement interval and for the cumulative outflow by using Darcy's law:

$$k = \frac{Q}{iA} \quad (1)$$

where  $k$  is the hydraulic conductivity or coefficient of permeability,  $Q$  is the volumetric flow rate,  $i$  is the hydraulic gradient, and  $A$  is the cross-sectional area of the sample. Stainless steel, standard Proctor compaction molds were used as the soil columns. For the flow-rate tests, dry unit weights were

allowed to vary by 1.5 percent within each stage, and the water contents were required to be within 1 percent water content of optimum. The compacted samples were cured at room temperature in double ziploc plastic bags. Time for curing was 1 day for the preliminary tests, but 7 days for the flow-rate tests. Laboratory temperature varied between 20°C and 27°C. All permeability tests used the same tap water for the permeant fluid as was used in the compaction of the samples. Leachate samples were collected every quarter pore volume of flow during the flow-rate tests. For the flow-rate tests, three pore volumes of effluent, based on the total, initial porosity of the mixture, were collected before the test was stopped. All of the preliminary tests were conducted at a gradient of about 100, whereas the flow-rate tests were performed at gradients of approximately 25, 50, 100, and 150.

## RESULTS AND DISCUSSION

### Proctor Tests

The moisture-density curves resulting from Proctor tests on the mixtures containing fly ash are similar to those for typical soils. The maximum dry unit weights ranged from 86.5 pcf (100 percent fly ash) to 111.0 pcf (25 percent fly ash, 7.5 percent cement, and 67.5 percent sand). Optimum water content of the mixtures ranged from 9.0 to 19.3 percent.

### Preliminary Column Tests

The preliminary column tests on the 21 mixtures resulted in steady-state permeabilities ranging from  $1.4 \times 10^{-4}$  to  $1.0 \times 10^{-7}$  cm/sec. Water contents, degrees of saturation, mixture

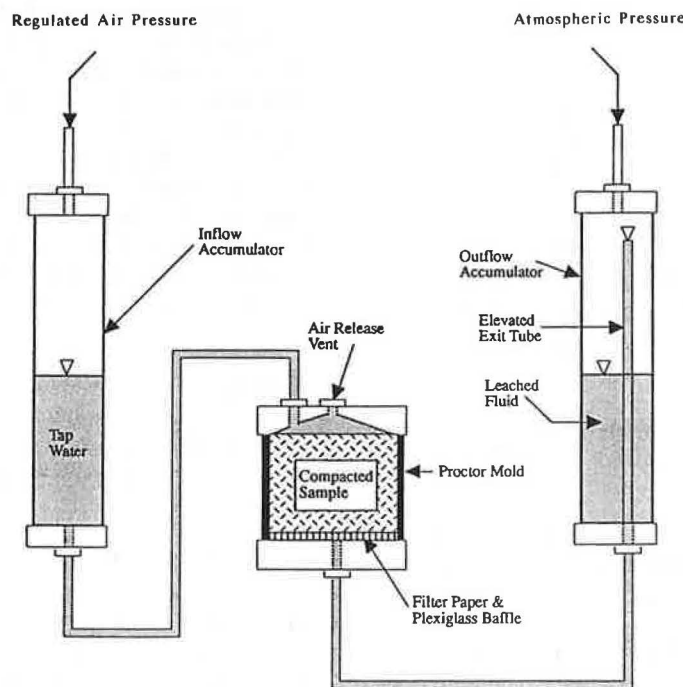


FIGURE 1 Schematic of column testing unit.

porosities, dry unit weights, and steady-state permeabilities are summarized in Table 3. The permeabilities obtained correspond well with those reported in the literature (3,5).

For all but one of the mixtures, the permeability increased slightly before leveling off at a steady-state value. In the case of 100 percent fly ash (Mix 1), the permeability increased slightly, then dropped significantly before reaching a steady-state value of  $1.0 \times 10^{-7}$  cm/sec. This permeability value is contradicted by results of the 100 percent fly ash flow-rate tests (reported later), in which permeabilities ranged from  $1.2 \times 10^{-6}$  to  $2.4 \times 10^{-6}$  cm/sec.

For all stages except Stage 3B, there appeared to be an optimum percentage of fines that minimized the permeability and maximized the dry unit weight of a given mixture of materials. An increase in fines over the optimum percentage resulted in a higher permeability and a lower maximum dry unit weight. For the Stage 3B specimens, the permeability continued to decrease with increasing percentage of fines.

### Flow-Rate Column Tests

The criteria for choice of mixtures used in the flow-rate testing phase were permeability and percent fly ash. Mixes 3, 10, and 16, from Stages 2, 3A, and 3B, respectively, were chosen because their permeabilities were substantially lower than the permeabilities of the other mixtures in the respective stages (see Table 3). Mix 21, from Stage 4, was chosen because it consisted of a higher percentage of fly ash than Mix 20, which had approximately the same permeability. Mix 1, the 100 percent fly ash mixture, also was chosen.

Column tests were performed on test specimens prepared from the five chosen mixtures to investigate the effect of flow rate on the leachability of metals from mixtures containing fly ash. Each of the five mixtures was tested at four different hydraulic gradients, producing four flow rates (i.e., fluid flow flux). Although the applied gradient was kept constant, the permeability varied during each test, resulting in slight variations in the flow rate.

### Mixture Permeabilities

As indicated in Table 4, the steady-state permeability values of the mixtures varied from  $1.5 \times 10^{-7}$  to  $2.4 \times 10^{-6}$  cm/sec. Like previous studies (3,5), this study indicated that bentonite is the most effective of the three admixture materials in reducing the permeability value for the fly ash mixtures. The permeability values between preliminary and flow-rate tests varied by no more than a factor of about two for all of the mixtures except 100 percent fly ash.

### DWSs

The average concentrations for the metal species in the DWSs (9) based on three pore volumes of flow are plotted versus mean flow rate in Figure 2. For the 100 percent fly ash specimens (Figure 2a) and the 50 percent fly ash/50 percent sand specimens, the average concentrations are greater than the DWS for B, Ba, Cd, Cr, and Pb, regardless of the flow rate. Except for Ba, the same trend is evident for the two fly ash/sand/bentonite mixtures (Figures 2b and 2c). Ba apparently is preferentially adsorbed by the bentonite.

For the mixture containing cement (Figure 2d), Ba was the only metal species that exceeded its DWS for all four flow rates. Five other metals (B, Cr, Fe, Mn, and Pb) were leached in sufficient quantities to exceed the DWS level for at least one flow rate, typically the lowest or the highest flow rate, or both.

### Leaching Behavior

In the flow-rate tests, two distinct shapes for the elution curves, or leaching behaviors, are apparent for the majority of the metal species traced. "Early" leaching behavior is characterized by concentrations that initially are high, peak before one pore volume of flow, then decrease, sometimes leveling off, for the remainder of the three pore volumes of flow. "De-

TABLE 3 DATA FROM PRELIMINARY COLUMN TESTS

Stage	Mix #	Fly Ash (%)	Fines <sup>a</sup>	G <sub>s</sub>	γ <sub>unit</sub> (pcf)	n	w <sub>unit</sub> (%)	w <sub>water</sub> (%)	S <sub>sat</sub> (%)	S <sub>unit</sub> (%)	θ <sub>unit</sub> (%)	θ <sub>sat</sub> (%)	Permeability (cm/sec)
1	1	100	100	2.28	87.6	0.384	14.1	19.2	52	70	20	27	$1.0 \times 10^{-7}$
2	2	75	75	2.36	95.4	0.352	11.4	15.6	50	68	18	24	$2.0 \times 10^{-4}$
	3	50	50	2.45	102.9	0.327	7.6	13.3	38	67	12	22	$6.4 \times 10^{-7}$
	4	25	25	2.55	104.1	0.346	12.1	13.6	58	66	20	23	$7.5 \times 10^{-4}$
	5	15	15	2.59	97.1	0.400	15.0	16.4	58	64	23	26	$7.5 \times 10^{-5}$
	6	5	5	2.63	88.0	0.464	19.5	21.6	59	65	27	30	$6.4 \times 10^{-4}$
3A	7	5	14.5	2.63	93.6	0.430	16.3	20.7	57	72	25	31	$2.1 \times 10^{-4}$
	8	15	23.5	2.59	99.6	0.384	13.4	16.6	56	69	22	26	$2.0 \times 10^{-4}$
	9	25	32.5	2.55	103.4	0.350	13.2	14.5	63	69	22	24	$2.7 \times 10^{-5}$
	10	50	55	2.45	102.4	0.330	11.1	14.2	55	70	18	23	$4.5 \times 10^{-7}$
	11	75	77.5	2.36	93.3	0.366	12.8	16.8	52	69	19	25	$1.1 \times 10^{-4}$
3B	12	5	15	2.63	97.4	0.406	15.7	18.9	60	73	24	30	$7.5 \times 10^{-5}$
	13	15	25	2.59	96.7	0.402	15.1	17.7	58	68	23	27	$1.4 \times 10^{-4}$
	14	25	35	2.55	98.4	0.382	15.8	16.7	65	69	25	26	$8.2 \times 10^{-4}$
	15	50	60	2.46	97.2	0.367	12.7	NA	54	NA	20	NA	$1.7 \times 10^{-4}$
	16	75	85	2.37	90.0	0.391	15.9	NA	59	NA	23	NA	$5.2 \times 10^{-7}$
4	17	5	14.5	2.67	100.7	0.396	14.0	13.8	57	56	26	22	$4.8 \times 10^{-5}$
	18	15	23.5	2.62	107.8	0.341	10.8	NA	55	NA	19	NA	$9.2 \times 10^{-4}$
	19	25	32.5	2.58	109.0	0.323	8.9	9.8	48	53	16	17	$4.0 \times 10^{-4}$
	20	50	55	2.47	103.2	0.330	10.3	11.8	52	59	17	19	$4.4 \times 10^{-7}$
	21	75	77.5	2.36	95.2	0.354	13.7	13.9	59	60	21	21	$4.5 \times 10^{-7}$

<sup>a</sup>Fines include fly ash, bentonite and/or cement. NA - data not available. Symbols: γ-unit weight; w-water content; S-degree of saturation; θ - volumetric water content; n - porosity; G<sub>s</sub> - specific gravity of solids of mixture.

TABLE 4 FLOW-RATE TEST DATA

Mixture	Hydraulic Gradient	$\gamma_{sat}$ (pcf)	n	$W_{sat}$ (%)	$W_{final}$ (%)	$Sr_{sat}$ (%)	$Sr_{final}$ (%)	$\theta_{sat}$ (%)	$\theta_{final}$ (%)	Permeability (cm/sec)
Mix #1 100% FA	24	84.5	0.406	14.8	19.9	49.4	66.4	20.1	27.0	$1.2 \times 10^{-4}$
	51	85.3	0.400	14.9	NA	51.0	NA	20.4	NA	$1.8 \times 10^{-4}$
	103	85.3	0.400	14.7	17.0	50.3	58.1	20.1	23.2	$2.4 \times 10^{-4}$
	154	85.6	0.392	15.2	18.3	53.8	64.1	21.1	25.4	$2.1 \times 10^{-4}$
Mix #3 50% FA, 50% S	24	105.0	0.313	10.1	10.9	54.3	58.6	17.0	18.3	$6.5 \times 10^{-7}$
	51	104.6	0.316	10.0	11.2	53.3	59.7	16.8	18.9	$7.7 \times 10^{-7}$
	103	104.1	0.318	10.1	11.2	53.1	58.9	16.8	18.7	$1.8 \times 10^{-6}$
	154	104.3	0.318	9.9	11.8	52.0	62.0	16.5	19.7	$1.6 \times 10^{-6}$
Mix #10 50% FA, 45% S, 5% B	24	102.0	0.333	11.3	13.0	55.5	63.8	18.5	21.2	$1.5 \times 10^{-7}$
	51	102.1	0.332	11.2	13.5	55.2	66.6	18.3	22.1	$2.3 \times 10^{-7}$
	103	102.2	0.332	11.2	13.1	55.2	64.6	18.3	21.4	$2.7 \times 10^{-7}$
	154	101.3	0.338	11.7	14.0	56.1	67.2	19.0	22.7	$3.4 \times 10^{-7}$
Mix #16 75% FA, 15% S, 10% B	24	99.4	0.328	16.0	19.1	77.7	92.7	25.5	30.4	$2.3 \times 10^{-7}$
	52	99.2	0.329	16.5	19.3	79.8	93.3	26.3	30.7	$3.1 \times 10^{-7}$
	103	99.8	0.325	16.2	19.0	79.7	93.5	25.9	30.4	$3.3 \times 10^{-7}$
	155	99.8	0.325	16.2	17.3	79.7	85.2	25.9	27.7	$3.1 \times 10^{-7}$
Mix #21 75% FA, 22.5% S, 2.5% C	24	103.9	0.295	13.3	15.2	75.0	85.7	22.1	25.3	$4.4 \times 10^{-7}$
	52	104.5	0.291	13.3	14.7	76.5	84.5	22.3	24.6	$6.2 \times 10^{-7}$
	103	104.8	0.289	13.3	14.4	77.2	83.6	22.3	24.2	$9.3 \times 10^{-7}$
	155	104.8	0.288	13.3	15.5	77.6	90.7	22.3	26.1	$8.9 \times 10^{-7}$

NA - data not available. Symbols:  $\gamma$  - unit weight; n - porosity; w - water content; Sr - degree of saturation;  $\theta$  - volumetric water content; FA - fly ash; S - sand; B - bentonite; C - cement.

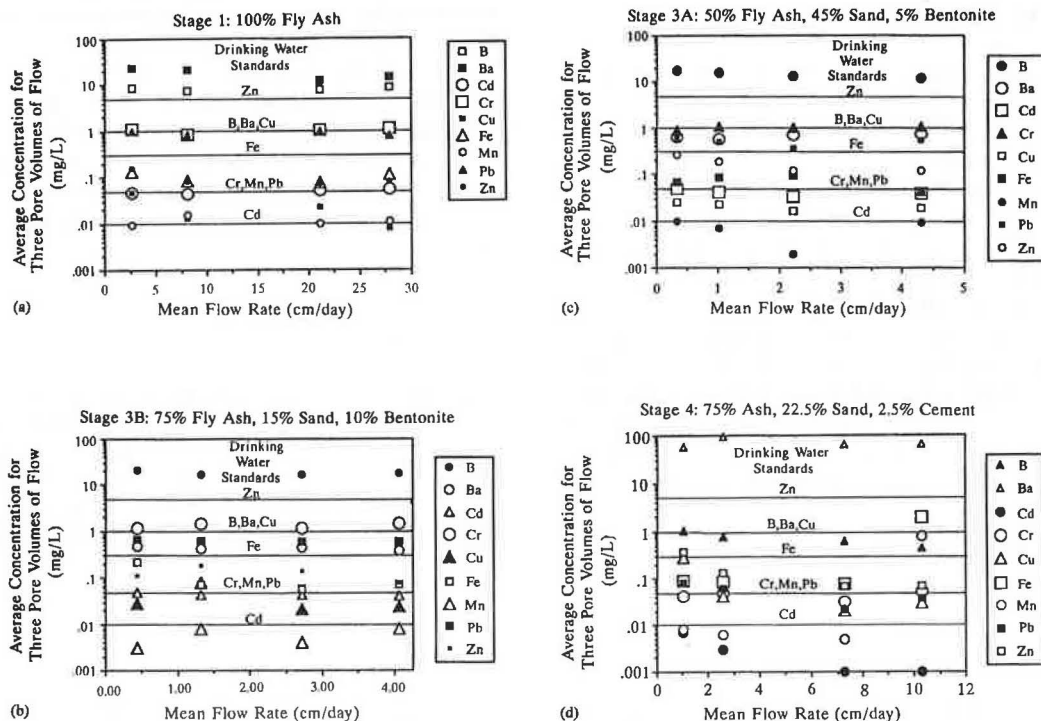


FIGURE 2 Average metal concentrations versus mean flow rate.

layed" leaching behavior is characterized by concentrations that start at relatively low values, increase, then either level off or peak before decreasing. In some cases, the metal concentrations were still increasing at three pore volumes of flow. Figure 3 shows typical early and delayed leaching elution curves found in this study.

Of the 13 metals traced, only Ba, Ca, and Sr show delayed leaching behavior. These three metals appear in the second column of the periodic table and are divalent ions in their most stable oxidation states. Comparisons of the relative charge densities of the metal ions monitored in this study indicate that  $Ba^{2+}$ ,  $Ca^{2+}$ , and  $Sr^{2+}$  possess the three smallest charge

densities. The consistent relationship between the two types of leaching behaviors with respect to the charge densities of the metal ions suggests that the equilibrium chemistry of the leaching fluid may play a significant role in the rate at which metal ions dissolve into solution (i.e., it appears that the ions with higher charge densities dissolve into solution first). After the more competitive ions are leached away from the particle surfaces, the ions with lower charge densities dissolve into solution.

Although early leaching behavior has previously been recognized (3), no mention has been made of delayed leaching behavior for similar tests, although similar behavior is seen

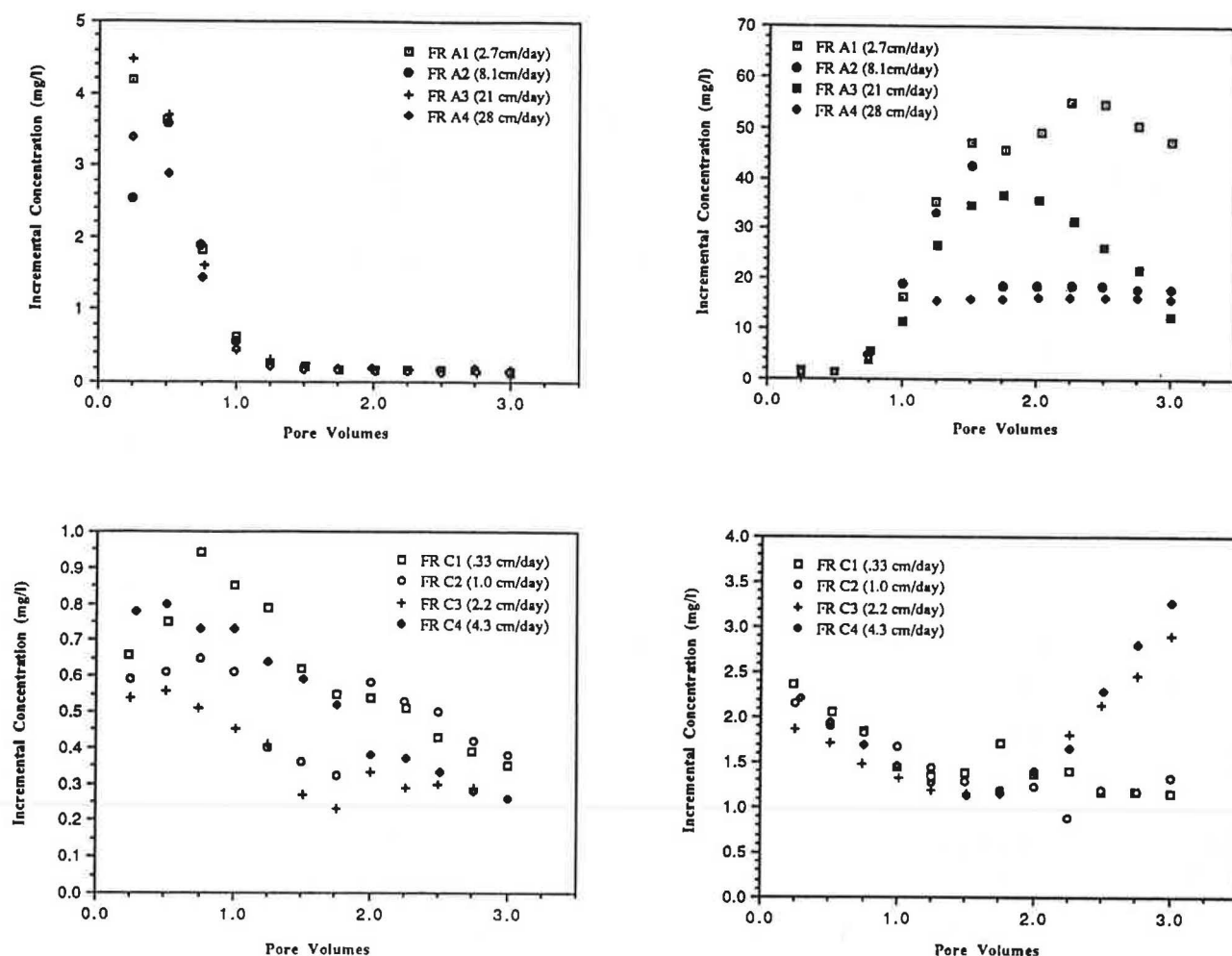


FIGURE 3 Typical early and delayed leaching curves.

for Ca in the leaching curves of Edil et al. (5). Nonetheless, it appears from the results of this study that the equilibrium chemistry of the leaching fluid may play a significant role in the leaching of metals from fly ash particle surfaces. However, additional study is needed to verify this trend.

#### Percent Mass Leached

The percentages of the mass of each metal leached per mass available for five test specimens (one from each of the five different sample mixtures) are provided in Table 5. The test specimens were chosen such that the flow rates for the test specimens are approximately the same (i.e., relative to the range of flow rates in the flow-rate tests). Overall,  $\leq 0.1$  percent of Fe and Mn was leached within three pore volumes of flow regardless of the mixture constituency. The 317 percent reported for Ba in Test FR E2 and the values exceeding 100 percent for Mo in Tests FR C3 and FR D3 are probably due to insufficient extractions procedures.

The addition of bentonite (5 or 10 percent) resulted in significant reductions in the leached masses of Ba, Ca, and Sr relative to the 100 percent fly ash or the 50 percent fly ash/

50 percent sand specimens. For the same comparison, reductions in the leached amounts of Al, Cd, Cu, Pb, and Zn were less significant, and a slightly adverse effect was noticed for the leached masses of B, Cr, and Mo.

The reductions in the leached masses of Ba, Ca, and Sr apparently represent a retardation effect wherein there is a delay in the leaching of these three metals due to a greater adsorption affinity for the exchange sites of the bentonite. A greater adsorption affinity of Ba, Ca, and Sr would indicate that the charge densities of these three metals are greater than the charge densities of the other competing ions, contradicting the hypothesis that the delayed leaching behavior of these three metals in all mixtures is due to smaller charge densities. The contradiction may be related to the difference between the charge densities of unhydrated versus hydrated chemical species (i.e., a smaller charge density for an unhydrated chemical species may result in a greater charge density for the same chemical species when hydrated, and vice versa). Therefore, smaller charge densities for unhydrated  $\text{Ba}^{2+}$ ,  $\text{Ca}^{2+}$ , and  $\text{Sr}^{2+}$  may result in greater charge densities for hydrated  $\text{Ba}^{2+}$ ,  $\text{Ca}^{2+}$ , and  $\text{Sr}^{2+}$ . Once dissolved, greater hydrated charge densities for  $\text{Ba}^{2+}$ ,  $\text{Ca}^{2+}$ , and  $\text{Sr}^{2+}$  would result in relatively more competitive adsorption reactions for these three species rel-

TABLE 5 PERCENTAGE MASS LEACHED AND CENTROIDAL PORE VOLUME VERSUS MIXTURE

STAGE	STAGE 1		STAGE 2		STAGE 3A		STAGE 3B		STAGE 4	
MIXTURE	100% FA		50% FA, 50% S		50% FA, 45% S, 5% B		75% FA, 15% S, 10% B		75% FA, 22.5% S, 2.5% C	
Flow rate (cm/day)	FR A1 (2.73)		FR B2 (3.41)		FR C3 (2.22)		FR D3 (2.73)		FR E2 (2.59)	
	Percentage Mass Leached	Centroidal Pore Volume	Percentage Mass Leached	Centroidal Pore Volume	Percentage Mass Leached	Centroidal Pore Volume	Percentage Mass Leached	Centroidal Pore Volume	Percentage Mass Leached	Centroidal Pore Volume
<b>Metal:</b>										
Al	87.2	1.00	86.7	1.20	76.5	2.11	53.3	1.60	2.1	1.24
B	9.9	1.17	11.9	1.62	21.1	2.19	16.5	1.62	0.7	0.92
Ba	99.2	3.60	61.5	3.76	4.0	3.09	1.6	1.50	317.0	2.16
Ca	12.5	3.41	11.8	3.79	1.7	3.11	0.6	1.44	12.8	2.55
Cd	11.5	0.77	12.2	1.26	10.4	2.13	8.8	1.55	0.7	0.77
Cr	44.4	0.57	33.0	0.85	50.7	1.32	46.7	1.20	1.4	1.43
Cu	0.6	0.91	0.7	1.15	0.3	1.77	0.2	0.97	1.4	1.13
Fe	< 0.1	2.94	< 0.1	3.12	< 0.1	1.92	< 0.1	1.09	< 0.1	2.12
Mn	0.1	1.01	0.1	2.28	< 0.1	0.80	< 0.1	0.72	< 0.1	2.57
Mo	96.7	0.52	99.0	0.69	111.0	0.97	108.3	0.70	7.6	0.78
Pb	32.5	0.93	17.5	1.12	13.0	2.12	13.4	1.57	1.5	0.77
Sr	24.5	3.40	17.1	3.79	1.6	2.77	0.7	1.36	25.9	2.32
Zn	3.2	1.24	4.2	1.19	2.7	1.70	1.9	0.46	0.7	1.10

\* Percentage Mass Leached = (mass leached/mass available) X 100%  
NOTE: FA = Fly Ash; S = Sand; B = Bentonite; C = Cement

ative to the other species in solution. Whereas this explanation may appear feasible, it cannot be substantiated by the data from the present study.

Except for Ba, Ca, Cu, and Sr, the addition of only 2.5 percent cement (Stage 4) to a mixture including 75 percent fly ash and 22.5 percent sand resulted in a drastic reduction in the mass of metal species leached in three pore volumes of flow. These reductions may be due to a pozzolanic reaction between the cement and the migrating metal species.

#### Leaching Rate

To evaluate the effect of flow rate on the leaching rate of the fly ash mixtures, a centroidal coordinate system was used. In this case, leaching rate refers to the rate of migration of the eluted center of mass of the metal species relative to the average flow rate. Since each effluent sample represents one-quarter pore volume of flow, the measured concentrations of the metals in the effluent samples represent only average values of concentration for a one-quarter pore volume of effluent. As a result, the elution curve represents a histogram of concentration, or mass, of metal leached versus pore volumes of flow. For any given histogram, the center of mass can be described by the centroidal coordinates  $\bar{m}$  and  $\bar{PV}$ , where  $\bar{m}$  is given by

$$\bar{m} = \frac{\sum_i A_i \cdot m_{ci}}{\sum_i A_i} \quad (2)$$

and  $\bar{PV}$  is given by

$$\bar{PV} = \frac{\sum_i A_i \cdot PV_{ci}}{\sum_i A_i} \quad (3)$$

where

$$A_i = m_i \cdot PV_i \quad (4)$$

$$m_{ci} = \frac{m_i}{2} \quad (5)$$

and

$$PV_{ci} = \frac{PV_i}{2} \quad (6)$$

where  $m_i$  is the mass collected in a sample volume (i.e., in  $PV_i$ ) of effluent.

Here,  $PV_i = 1/4$ , so Equation 3 reduces to

$$\bar{PV} = \frac{1}{8} \frac{\sum_{i=1}^{12} m_i(2i-1)}{\sum_{i=1}^{12} m_i} \quad (7)$$

Since the total mass leached, or area under the histogram, is a more pertinent parameter than is the centroidal mass coordinate,  $\bar{m}$ , Equation 2 is not used in this study.



The centroidal pore volume coordinate,  $\overline{PV}$ , given by Equation 7 is a measure of the rate of leaching of the center of mass of a given element; the higher the value of  $\overline{PV}$ , the slower the rate of leaching of a given metal. If the metal species is uniformly and homogeneously mixed in the column sample before leaching, the center of mass is represented by the center of volume of the column. If the metal is not constrained from leaching by some time-dependent process and is not degraded, then the center of mass of the metal should appear at one-half a pore volume of leachate. A  $\overline{PV}$  value greater than one-half indicates a delay process (e.g., ion exchange) in the elution of the metal species. A  $\overline{PV}$  value less than one-half could indicate several possibilities, including nonuniform sample mixture, an error in the determination of chemical concentrations, or preferential flow paths in the sample. Since the value of  $\overline{PV}_i$  of  $1/4$  used in Equation 7 is based on a pore volume assuming a completely saturated sample, the centroidal pore volume coordinate for unsaturated samples must be corrected as follows:

$$\overline{PV}(\theta) = \left( \frac{n}{\theta} \right) \overline{PV}(n) = \frac{\overline{PV}(n)}{S} \quad (8)$$

where

$n$  = total porosity of the sample,  
 $\theta$  = volumetric moisture content of the sample,  
 $S$  = degree of saturation (decimal),

$\overline{PV}(n)$  = centroidal pore volume based on  $n$  (i.e., Equation 7), and

$\overline{PV}(\theta)$  = centroidal pore volume based on  $\theta$ .

In this study, all of the samples were unsaturated, so Equation 8 was used in lieu of Equation 7. However, since  $\theta$  was not constant, an average value of  $\theta$ , or  $\theta_{AVE}$ , was used in Equation 8, where

$$\theta_{AVE} = \frac{\theta_i + \theta_f}{2} \quad (9)$$

and  $\theta_i$  and  $\theta_f$  are the initial and final volumetric moisture contents, respectively. The use of  $\theta$  or  $\theta_{AVE}$ , which is less than  $n$ , resulted in effluent volumes that represent more than the three pore volumes of flow based on a saturated flow system.

On the basis of the centroidal pore volume approach described, the centroidal pore volumes for each metal species and each mixture in the flow-rate tests are given in Table 5. Again, the test specimens were chosen such that the mean flow rates for the test specimens are approximately the same (i.e., relative to the range of flow rates in the flow-rate tests).

Except for Zn, the leaching rates of the metals in the 50 percent fly ash/50 percent sand mixture were lower (higher  $\overline{PV}$  values) relative to the 100 percent fly ash specimen. The substitution of 5 percent bentonite for 5 percent sand in the 50 percent fly ash/50 percent sand mixture resulted in the further reduction in the leaching rates of all metals except Ba, Ca, Fe, Mn, and Sr. The further reduction apparently is attributable to the ion exchange capacity of the sand/bentonite admixture. Ba, Ca, and Sr are the metal species that exhibited delayed leaching behavior, and Fe and Mn are the metal

species that essentially did not leach from the mixtures within three pore volumes of flow.

An increase in bentonite content from 5 to 10 percent did not lower the leaching rates as expected. On the contrary, the leaching rates of all 13 metals increased. However, the fly ash contents of the 10 percent mixtures were also greater by 25 percent relative to the 5 percent mixtures. As a result, an increase in fly ash content apparently controls over an increase in bentonite content. Therefore, on the basis of the results of the test specimens from Stages 1, 2, 3A, and 3B, it appears that an increase in fly ash content tends to increase the leaching rate of the center of mass of the metal species.

A comparison of the two test specimens containing 75 percent fly ash (Stages 3B and 4) indicates that only four (Al, B, Cd, and Pb) of the 13 metals were leached faster for the Stage 4 mixture containing 2.5 percent cement. Therefore, on the basis of this limited comparison, it appears that cement is more effective than bentonite at reducing the leaching rate of metals for mixtures containing the same amount of fly ash. However, when the Stage 4 results are compared with the results for the Stage 2 and Stage 3A tests with lower (50 percent) fly ash contents, 9 of the 13 metal species in Stage 4 were leached at faster rates relative to the Stage 2 mixture, and 10 of the 13 metal species in Stage 4 were leached at faster rates relative to the Stage 3A mixtures. This follows the same trend, already mentioned, of a faster leaching rate (lower  $\overline{PV}$  value) with a greater fly ash content, regardless of the admixture constituents.

No simple trend in the leaching rate of a given metal species as a function of the flow rate for any of the mixtures was found. In addition, within a given mixture, the leaching rates of different metal species may increase or decrease with increasing flow rate or show no apparent trend. It is obvious that from this study that either there are no general trends in leaching rate versus flow rate or the test procedures and setup used in this study do not exhibit enough control to detect such trends.

#### pH and Electrical Conductivity

In general, there was a reduction in pH and electrical conductivity (EC) with an increase in pore volumes of flow for all test specimens. Both the pH and the EC of the effluent tended to be greater than that of the influent, especially within the first pore volume of flow. Also, there was no significant effect of flow rate on the pH and EC of the effluent. The pH values of all effluent solutions fall within the relatively narrow range of 11.8 (Stage 2, FR B4) to 13.3 (Stage 4, FR E4). This range is significantly more basic than the pH of 6.3 measured for the tap water.

Typical plots of EC versus pore volumes of flow are shown in Figure 4. The elution of ions from the test specimens is reflected by the initially high EC values for the effluent solutions. The initially high EC values gradually decrease within three pore volumes of flow towards the background EC value of 29.7  $\mu\text{mhos/cm}$  for the tap water. In the case of the fly ash and fly ash/sand test specimens (Stages 1 and 2), the EC of the effluent approaches background more completely and rapidly, within the first one to two pore volumes of flow. The

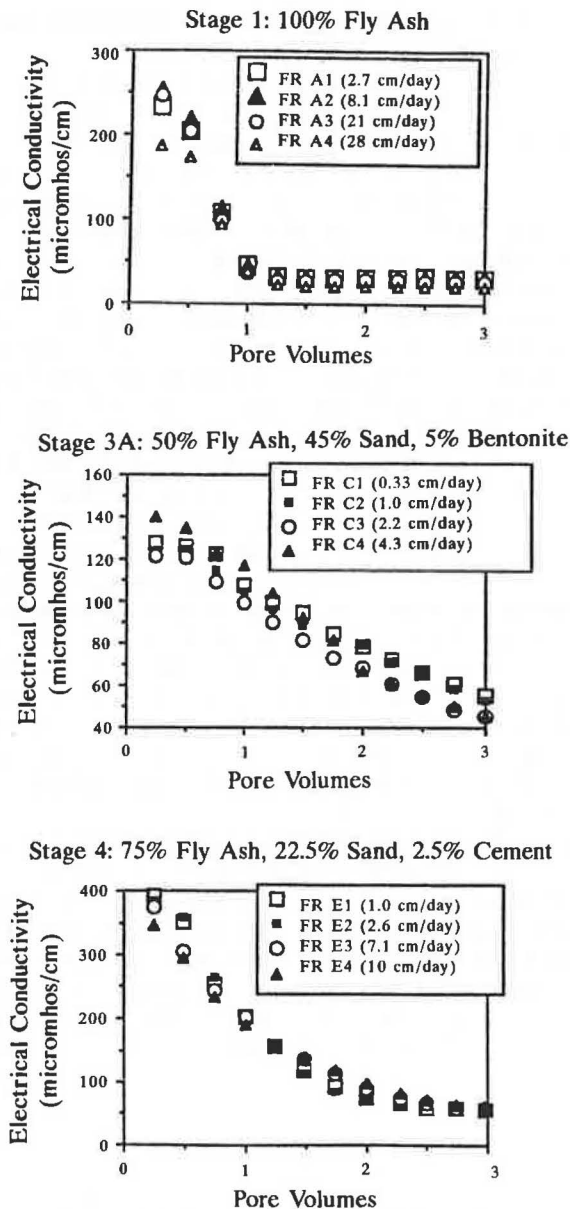


FIGURE 4 Typical curves of electrical conductivity versus pore volumes of flow.

trend is slightly more gradual and less complete for the fly ash/sand/cement specimens (Stage 4), and much more gradual for the specimens containing bentonite (Stages 3A and 3B). The more gradual reduction in EC with increase in pore volumes for the Stage 3A and 3B test specimens may be attributed to the retardation effect of the bentonite on the migrating metal ions.

An increase in fly ash content appears to result in an increase in the maximum EC value measured in the effluent. However, the maximum EC values for the Stage 4 test specimens containing 75 percent fly ash are significantly higher than the test specimen containing 100 percent fly ash (Stage 1), the difference being attributed to the cement content in the Stage 4 mixture.

For mixtures containing the same amount of fly ash, there is a contradiction in the maximum EC values measured in the effluent. In the case of the test specimen containing 50 percent fly ash, there appears to be little effect for the test specimens with bentonite (Stage 3A) relative to the test specimens without bentonite (Stage 2). However, for the test specimens containing 75 percent fly ash, there is a pronounced difference in maximum EC values, with significantly lower EC values for the test specimens containing bentonite (Stage 3B) relative to those containing cement (Stage 4). The differences apparently are due to the bentonite admixture of the Stage 3B test specimens or the incorporation of cement into the Stage 4 test specimens, or both.

In general, the EC data tend to reflect the previously drawn conclusions with respect to leaching rate of specific metal ions with some notable exceptions. First, the measured EC values of the effluent samples are not metal specific; the EC value represents the combined conductivity of all ions in solution. Second, anions (e.g.,  $\text{Cl}^-$ ) as well as cations (metals) contribute to the electrical conductivity of the effluent. Finally, whereas only ions contribute to the electrical conductivity of a solution, the measured metal concentrations probably include neutral metal species (e.g., metal oxides), which do not contribute to the measured EC value.

#### Sidewall Leakage

Although rigid-wall permeability tests are susceptible to sidewall leakage, certain factors suggest that sidewall leakage was not significant in this study. Sample preparation and curing conditions were nearly identical for every sample tested. No disturbance of the samples was noticed during the curing time and the testing period. Though final degrees of saturation varied slightly within the stages, no trend was evident indicating a correlation with leached metal quantities. The percentage of leached metal masses should be an inverse function of the degree of sidewall leakage. Less surface area within the porous media is exposed to the leaching fluid if there are preferential flow paths along the sidewalls. Less exposed surface area allows less dissolution of metals into the leaching fluid, resulting in lower quantities of metals leached. On the basis of an examination of the final values of the degree of saturation and the data for the percentage of metal mass leached within each stage, it is evident that the quantities of metals leached are not a function of the final saturation levels. No trends are evident indicating a particular test to have a low final degree of saturation and to have leached a lower percentage of metal mass relative to the other tests within the same stage. This type of trend would be expected if sidewall leakage were significant. In addition, permeability values measured in this testing program correlate well with those reported in recent literature (3,5,11).

#### SUMMARY AND CONCLUSIONS

##### Metal Leaching Versus Flow Rate

Metal leaching behavior from mixtures containing fly ash showed no dependence on flow rate for the range of flow

rates used in this study. Column tests at hydraulic gradients equal to 25, 50, 100, and 150 resulted in no consistent differences of metal leaching characteristics between tests. For all mixtures, average concentrations of four to six metals exceeded the DWSs (9) for at least one flow rate. However, no trends were apparent that linked magnitude of flow rate to relative metal concentrations in the leachate. On the basis of the centroidal pore volume approach, the leaching rate does not appear to be a function of flow rate. The leaching rate varied from metal to metal in each mixture tested, independent of flow rate. In summary, no evidence was found of a dependence of metal leaching on flow rate.

### Permeability Versus Admixture Constituents

This research indicates that compacted Class F fly ash, when stabilized with bentonite or cement, can have a permeability on the order of  $10^{-7}$  cm/sec, the current regulatory standard for earthen liners. Bentonite appears to be slightly more effective than cement at minimizing the permeability of mixtures containing fly ash and sand. There appears to be an optimum percentage of sand that minimizes the permeability of the fly ash-sand mixtures. For the three types of admixtures considered in this study, the permeability decreased in the following order: fly ash/sand > fly ash/sand/cement > fly ash/sand/bentonite.

### Leaching Behavior

Of the 13 metals traced, Ba, Ca, and Sr showed delayed leaching behavior. Fe and Mn showed no trend, and Al, B, Cd, Cr, Cu, Mo, Pb, and Zn showed early leaching behavior. The delayed leaching behavior contrasts with most other studies, which have indicated only early leaching behavior. These two leaching behaviors suggest that there is competition between metal ions for dissolution into the pore fluid.

### Metal Leaching Versus Admixture Constituents

Addition of sand resulted in little change in the percentage of mass leached of the metals traced. In contrast, the addition of bentonite resulted in a significant reduction in the mass of Ba, Ca, and Sr leached in three pore volumes of flow relative to the 100 percent fly ash mixture, whereas reductions in the amount of Al, Cd, Cu, Pb, and Zn were less significant. However, an adverse effect was noticed for the mass of B, Cr, and Mo leached in three pore volumes of flow. The reduction in the mass of Ba, Ca, and Sr is believed to be due to an increase in the delayed leaching behavior for these three chemical species resulting from the bentonite addition. Except for Ba, Ca, Cu, and Sr, the addition of only 2.5 percent cement to a mixture of 75 percent fly ash and 22.5 percent sand resulted in a drastic reduction in the mass of nine other metal species leached in three pore volumes of flow.

The centroidal pore volume approach was also used to compare the effects of different admixture constituents. For spe-

cific elements between different mixtures at approximately the same flow rate, increasing the fly ash content in fly ash/sand/bentonite mixtures tended to increase the leaching rate of most metals. The 2.5 percent cement content was more effective than the 10 percent bentonite at reducing the leaching rate of metals for mixtures containing the same amount of fly ash.

Both bentonite and cement admixtures reduce the leaching of metals in the first three pore volumes of flow. Overall, cement reduces the leaching of more metals to a greater degree than bentonite.

### ACKNOWLEDGMENT

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# Adsorption of Heavy Metal Pollutants by Landfill Soils

YUNG-HSU HSIEH

The leachate in landfill sites carries out highly concentrated inorganic and organic pollutants. Such pollutants can be intercepted by the adsorption of landfill soil and will proceed with a long-term stabilization-degradation reaction in the soil. Moreover, the possibility of groundwater pollution due to the leachate from landfill can be reduced through the intercepted adsorption by the soil. Simulated adsorption batch experiments were carried out in this study to investigate the adsorption removal of heavy metals by soils. The tested soils included red loam from the Taichung sanitary landfill site and six alluvial soils located over the most widespread area in Taiwan. The results indicated the following trend for the ability of heavy metal adsorption removal by such tested soils: Hsinying soil > Chungsin Village soil > Chung Hsing University soil  $\approx$  Hsuehchia soil > Taichung sanitary landfill soil > Mashan soil > Tunghsiao soil. The adsorption removal of heavy metals followed  $Pb \approx Cu > Ni \approx Cd \approx Zn$ . The adsorption removal by red loam increased with Cd concentration; however, part of the Cd removal in the high-pH region for high Cd concentration was due to precipitation. Thus, consideration of the speciation distribution was necessary to determine the maximum probable precipitated amount and the minimum probable adsorbed amount. In addition, in the presence of organic ligand [ethylenediamine tetraacetic acid (EDTA)], the adsorption effect of Cd by red loam was suppressed because of the complexation with EDTA, and the adsorption decreased with increasing EDTA concentration.

Sanitary landfill is the most common approach for the final disposal of solid wastes in Taiwan. The leachate produced from the landfill site usually carries out highly concentrated inorganic and organic components. Such a potential hazard to the environment has aroused concern and become a chief object of research in solid waste treatment (1). Because food wastes are a large proportion of solid wastes and the weather is humid and rainy in Taiwan, the composition and the quantity of leachate produced from landfill are quite different from those in foreign areas. The infiltration of waterborne pollutants always increases with increasing flow rate of leachate produced, thus increasing the possibility of groundwater pollution. In addition, the constitution and the equipment of many landfill sites in Taiwan are not so well appointed, and the management system of landfill sites exhibits certain defects. Therefore, some inorganic hazardous pollutants, such as heavy metals, from industrial solid wastes can commonly be found in the leachate. Since the soil itself possesses a good adsorption function to the pollutants in leachate (2,3), it is expected that optimal control of soil properties and environmental factors of the landfill site not only can improve the

water quality of leachate, reducing the loading and cost of treatment, but also prevent pollution of the groundwater (4,5). The main thrust of this work is to investigate the adsorbability of inorganic heavy metal pollutants in leachate by the soil in landfill sites. The results may have important implications for the determination of landfill sites according to the properties of soil and the optimum control parameters on the operation of landfill.

## EXPERIMENTAL

### Materials

The tested soils were obtained from seven locations in Taiwan. The samples represent soils with various texture and properties. The locations and notations of the tested soils are specified in Tables 1 and 2. After the drying process under sunlight, the soil samples were crushed and sieved (No. 10, 2 mm pore diameter) to remove impurities in the soil, such as stones and roots, before use. All heavy metal electrolyte solutions were prepared from perchlorate salts, and sodium perchlorate was used as the inert electrolyte for maintaining constant ionic strength (6,7). The organic chemical, ethylenediamine tetraacetic acid (EDTA), was selected as the complexing ligand with heavy metals. The pH was adjusted by the addition of 0.1 N  $HClO_4$  or NaOH. All chemicals were of reagent grade, obtained from Lin-Chun, Japan, and used without further purification. Distilled, deionized water was used for all experimental operations.

### Determination of Soil Properties

The size distribution of soil particles was determined by sieving each given amount of soil sample through a series of pore diameters from 0.002 to 1 mm. The average size of soil particles and the texture of soil samples were thus calculated and specified (8,9). The pH was measured by pH meter (Hanna, Model HI8417, Italy) after the mixture of 25 g of soil sample and 25 mL of distilled, deionized water settled quiescently for 1 hr (10). Organic matter in soil was determined by the Walkley-Black method, in which the consumption of potassium dichromate was determined to convert the amount of organic matter oxidized (10). Cation exchange capacity (CEC) was measured by replacing the unchangeable cations in the soil sample with ammonium ions through the addition of a concentrated solution of ammonium acetate into the soil sample (10).



TABLE 1 SIZE DISTRIBUTION AND TEXTURE OF TESTED SOILS

Soil (Location and Notation)	Composition (%) of the Soil Sample			Texture
	Sand 2-0.05mm	Silt 0.05-0.002mm	Clay <0.002mm	
Taichung Sanitary Landfill Red Loam Soil (Taichung, R)	42.04	33.80	24.16	Loam
Chunghsin Village Soil (Chunghsin Village, C)	34.04	53.80	12.16	Silty Loam
Chung Hsing University Soil (Taichung, U)	38.04	49.80	12.16	Loam
Tunghsiao Soil (Tunghsiao, T)	97.52	1.40	1.08	Sand
Hsinying Soil (Hsinying, E)	6.04	52.80	41.16	Silty Clay
Hsuehchia Soil (Hsuehchia, K)	40.04	51.80	8.16	Silty Loam
Mashan Soil (Mashan, M)	84.02	14.40	1.58	Loamy Sand

TABLE 2 pH, ORGANIC MATTER CONTENT, AND CEC OF TESTED SOILS

Soil (Location and Notation)	pH	Organic Matter Content (%)	CEC (meq/100g)
Taichung Sanitary Landfill Red Loam Soil (Taichung, R)	4.15	1.24	4.17
Chunghsin Village Soil (Chunghsin Village, C)	5.20	2.21	7.91
Chung Hsing University Soil (Taichung, U)	5.90	1.61	7.61
Tunghsiao Soil (Tunghsiao, T)	8.40	0.03	0.88
Hsinying Soil (Hsinying, E)	7.70	1.37	17.94
Hsuehchia Soil (Hsuehchia, K)	7.65	1.31	7.04
Mashan Soil (Mashan, M)	8.10	0.26	2.95

### Adsorption of Heavy Metals

A 1-L solution of 0.5 M ionic strength with a certain amount of heavy metal ions was prepared in an Erlenmeyer flask. One hundred mL of each solution was transferred into a polyethylene plastic bottle, to which 10 g of the tested soil was added into each of the nine bottles, leaving the rest to be a blank, and a certain amount of acid or base was added in incremental amounts to obtain the final pH from 4 to 10 after the saturated adsorption at a period of 24 hr. The suspensions were filtered through 0.45- $\mu$ m pore size, 47-mm-diameter filter paper (Gelman, United States of America). The equilibrium pHs of the filtrates were determined and the residual heavy metals were measured by the atomic adsorption spectrophotometer (ZAA-8000, Hitachi).

## RESULTS AND DISCUSSION

### Analyses of the Tested Soils

The size distribution and the texture of the seven soil samples are indicated in Table 1. The sand contents of Tunghsiao (T) and Mashan (M) soils were 97.52 and 84.02 percent, respectively, and thus were labeled coarse-textured soil. Taichung sanitary landfill red loam soil (R), Chunghsin Village soil (C), Chung Hsing University soil (U), and Hsuehchia soil (K) showed close composition between sand and silt ratios and were classified as medium-textured soil. However, Hsinying soil (E) was thought to be fine-textured soil because of the high contents of both silt and clay.

Table 2 gives the pH, organic matter content, and CEC of the tested soils. Red loam soil (R) and Tunghsiao soil (T) presented the lowest and the highest pH values (4.15 and 8.40, respectively). The pH values of both of the two coarse-textured soils were greater than 8. The organic matter contents of all the soil samples were low, whereas the organic matter in sand was much less than that in loam and clay. Usually, the removal of heavy metal according to adsorption and complexation mechanisms was more significant when the organic matter content in soil was higher (11). CEC values of soils roughly increased with increasing organic matter content. Sands indicated the lower and loams the higher CEC values. Hsinying soil (E), which had the highest clay content, presented the largest CEC value, 17.94 meq/100 g.

### Effect of Type of Soil

To compare the adsorbabilities of heavy metal ions by soil, a little inconsistency was observed among the tested soils. However, a general trend in the ability of heavy metal adsorption removal was still noted, and the order Hsinying soil (E) > Chunghsin Village soil (C) > Chung Hsing University soil (U)  $\approx$  Hsuehchia soil (K) > Taichung sanitary landfill red loam soil (R) > Mashan soil (M) > Tunghsiao soil (T)



was concluded. This order was compatible with the sequences of CEC values and organic matter contents of the tested soils. This implied that the adsorption removal of heavy metals exhibited a close relationship with the number of functional groups in the soil. The removal of heavy metals by the seven tested soils is represented by cadmium and is shown in Figure 1. There was a breakpoint between pH 6 and 8 at which all the removals increased sharply. At pHs greater than 9, the removals were always beyond 95 percent; however, there was a marked difference in the adsorbabilities of Cd among the soil samples at low pHs. The order  $E > C > R > K > U > M > T$  was observed. Hsinying soil (E) was identified as the silty clay of fine-textured soil, which exhibits the largest specific surface area and the highest CEC value, and the best Cd removal situation was then indicated. On the other hand, Tunghsiao soil (T) and Mashan soil (M) presented more sand and less clay composition, higher pH, and the lowest organic matter content and CEC value. Therefore, the adsorbability of heavy metal was relatively weak.

#### Effect of Type of Heavy Metal

Although there was a little inconsistency in the sequences of adsorption removal by soils among the selected heavy metals, a general order of  $Pb \approx Cu > Ni \approx Cd > Zn$  was concluded. The removal of heavy metals by the seven tested soils is represented by Taichung sanitary landfill red loam soil (R) and is shown in Figure 2. Such a removal sequence as a function of pH might have something to do with the solubility product constants of heavy metal hydroxides. At a concentration of  $10^{-4}$  M, the removal of heavy metals from soil suspension partly resulted from the formation of hydroxide precipitates at high pHs. Comparing the solubility products of the selected heavy metals in this study, it was found that the formation

sequence of hydroxide precipitates followed the order  $Cu > Zn > Ni > Pb > Cd$  with increasing pH. However, this order was not completely the same as that from the adsorption experiments, in particular, the sequence of Pb and Zn. The relatively high removal of Pb could be related to its large hydrolysis equilibrium constant. Pb showed the largest first hydrolysis equilibrium constant among the five heavy metals and enhanced adsorbability by soil. Zn might exhibit the specific affinity with red loam soil. Therefore, a significant and consistent removal percentage was observed for the whole pH region studied.

#### Effect of Concentration of Heavy Metal

Figure 3 shows the adsorption removals of Cd by Taichung sanitary landfill red loam soil (R) at various Cd concentrations. Although the results indicated that the removals of Cd were all less than 10 percent in the pH region studied at a Cd concentration of  $10^{-5}$  M, and the removals were obviously enhanced at Cd concentrations of  $10^{-4}$  M and  $10^{-3}$  M, the speciation distribution of Cd should be considered because the mechanisms of Cd removal actually included both adsorption and precipitation at Cd concentrations greater than  $10^{-4}$  M. This was cited by theoretical calculation at the presence of  $10^{-4}$  M Cd in solution. To determine the minimum probable amount of Cd removal by adsorption, the maximum probable amount of removal by precipitation calculated on the basis of the hydrolysis equilibrium constants and the solubility product constant of Cd was subtracted from the actual removal by experiments, and the pH-dependent distribution curve of Cd removal by adsorption was then obtained. The results are shown in Figure 4. It was also found that removal by adsorption increased gradually when the pH of the solution increased. Up to a pH value of 8.43, the adsorption amount

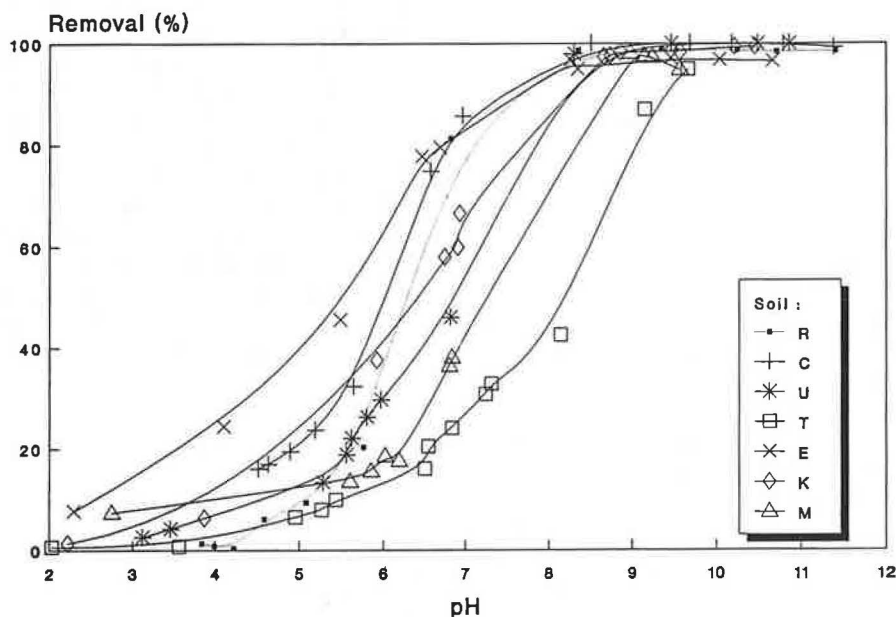


FIGURE 1 pH-dependent adsorption removals of Cd by the seven tested soils (10g/L soil, 0.5 M ionic strength,  $10^{-4}$  M Cd).

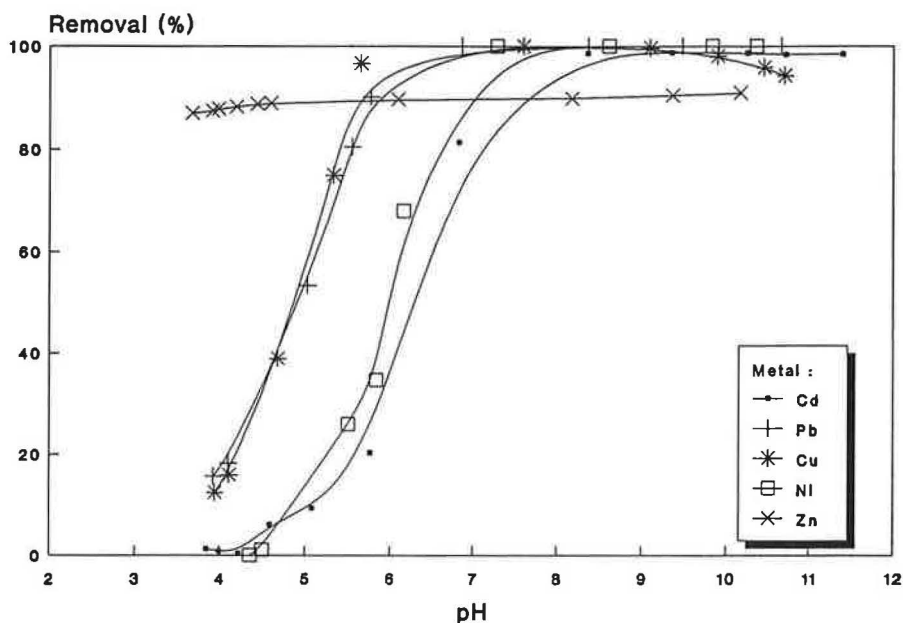


FIGURE 2 pH-dependent adsorption removals of various heavy metals by Taichung sanitary landfill red loam soil (10g/L soil, 0.5 M ionic strength,  $10^{-4}$  M heavy metals).

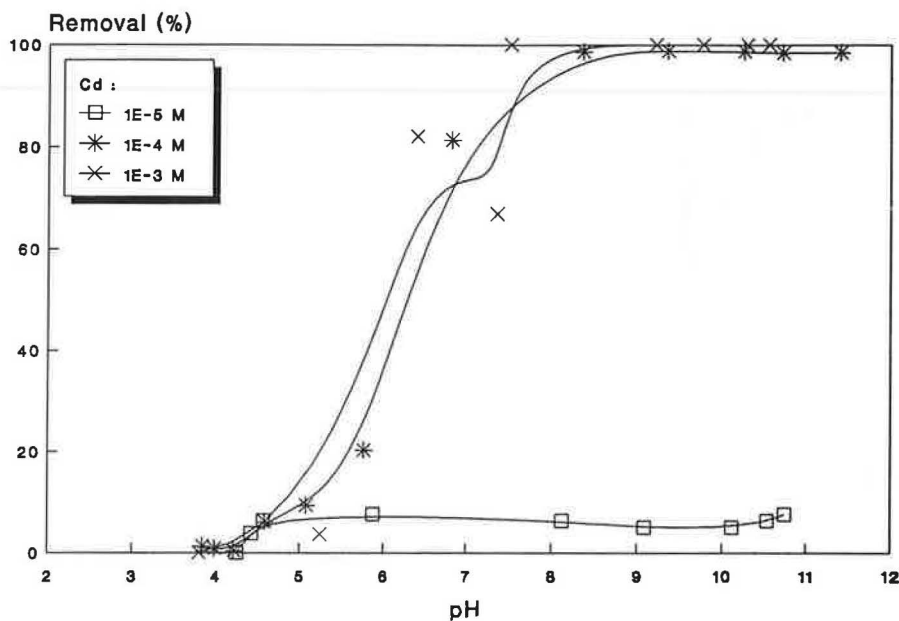


FIGURE 3 pH-dependent adsorption removals of Cd by red loam soil at various Cd concentrations (10g/L soil, 0.5 M ionic strength).

reduced significantly because of the formation of precipitates. That is, the adsorption should be the only mechanism for Cd removal at pHs less than 8.43. Moreover, the diminution of Cd removal by adsorption at pHs greater than 8.43 was considered on the basis of the maximum probable amount of precipitation. The actual Cd removal by adsorption might be more than that indicated in Figure 4 if the consecutive redissolution of  $\text{Cd}(\text{OH})_2$  precipitates proceeded toward an equilibrium situation because of the reduction of Cd in aqueous phase by adsorption.

#### Effect of the Presence of EDTA

To simulate the influence on the adsorbability of heavy metal by soil in the presence of organic ligand in leachate, EDTA is commonly selected (12–16). When Cd and EDTA coexisted in the soil solution, the adsorption removal of Cd was markedly affected because of the strong complexation reaction. Figure 5 shows the pH-dependent Cd removals at various Cd:EDTA ratios (10:1, 1:1, and 1:10) based on a fixed Cd concentration of  $10^{-4}$  M. The removal of Cd significantly

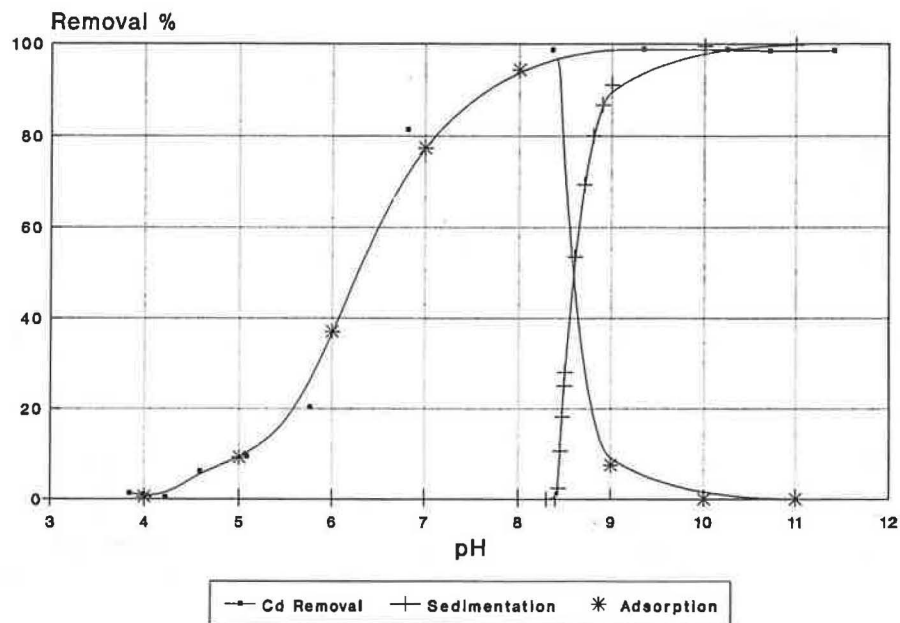


FIGURE 4 pH-dependent maximum probable removal of Cd by precipitation and minimum probable removal of Cd by adsorption on red loam soil (10g/L soil, 0.5 M ionic strength,  $10^{-4}$  M Cd).

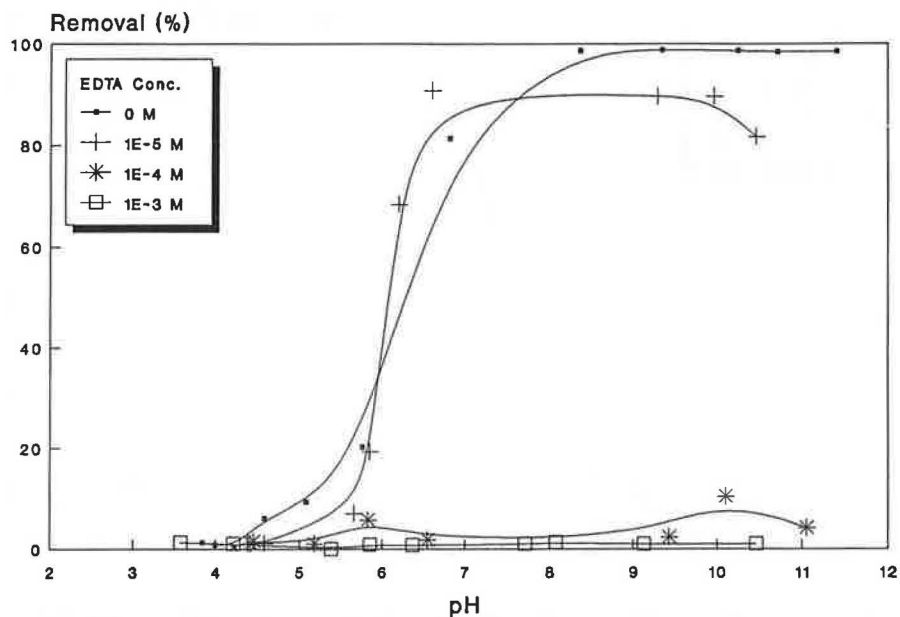


FIGURE 5 pH-dependent adsorption removals of Cd by red loam soil in the presence of EDTA with various concentrations (10g/L soil, 0.5 M ionic strength,  $10^{-4}$  M Cd).

decreased with increasing concentration of EDTA, and the removals of Cd were extremely scarce in the whole pH range at a Cd:EDTA ratio of 1:10.

At a Cd:EDTA ratio of 10:1, the removals of Cd at pHs less than 9 were similar to those without EDTA in solution, and the removals reduced about 10 percent consistently at pHs greater than 9. This could be demonstrated by the spe-

ciation distribution diagram (Figure 6) established through the calculations of Cd-EDTA complex-formation constants. For the dissolved Cd species, 10 percent of Cd existed as the complexed  $\text{CdEDTA}^{2-}$  and 90 percent as the hydroxyl species at all the pHs. However, most were  $\text{Cd}^{2+}$  at pHs less than 9. The removals of Cd decreased by 10 percent in the presence of EDTA when the pH was raised to 8 or higher. This defi-

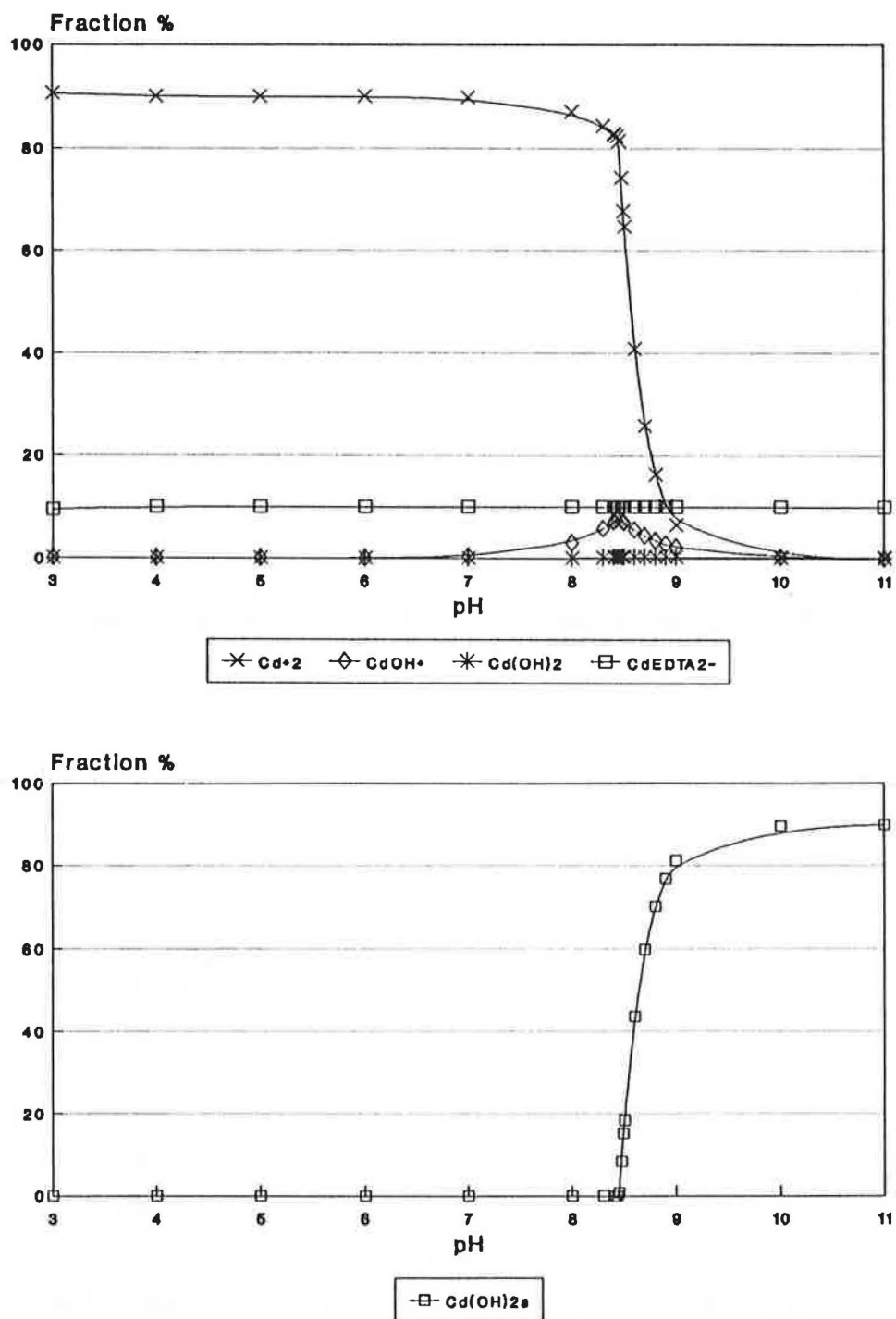


FIGURE 6 Speciation distribution of dissolved Cd (*top*) and Cd(OH)<sub>2</sub> precipitate (*bottom*) in Cd-EDTA aqueous solution (0.5 M ionic strength, 10<sup>-4</sup> M Cd, 10<sup>-5</sup> M EDTA).

nitely implied that the formation of the complexed Cd-EDTA<sup>2-</sup> caused the depression of Cd removal, that is, CdEDTA<sup>2-</sup> could not be adsorbed by the tested soil.

At a Cd:EDTA ratio of 1:1, the removals of Cd were largely diminished compared with those without EDTA in solution, and the removals were all within 10 percent in the whole pH

range studied. Since CdEDTA<sup>2-</sup> was the predominant species (Figure 7), the removals of Cd were significantly reduced. At a Cd:EDTA ratio of 1:10, the removals of Cd were further depressed and were almost zero at all pHs. To look at the speciation distribution of Cd and EDTA in solution, all the Cd presented as the complexed CdEDTA<sup>2-</sup>. Ten percent of

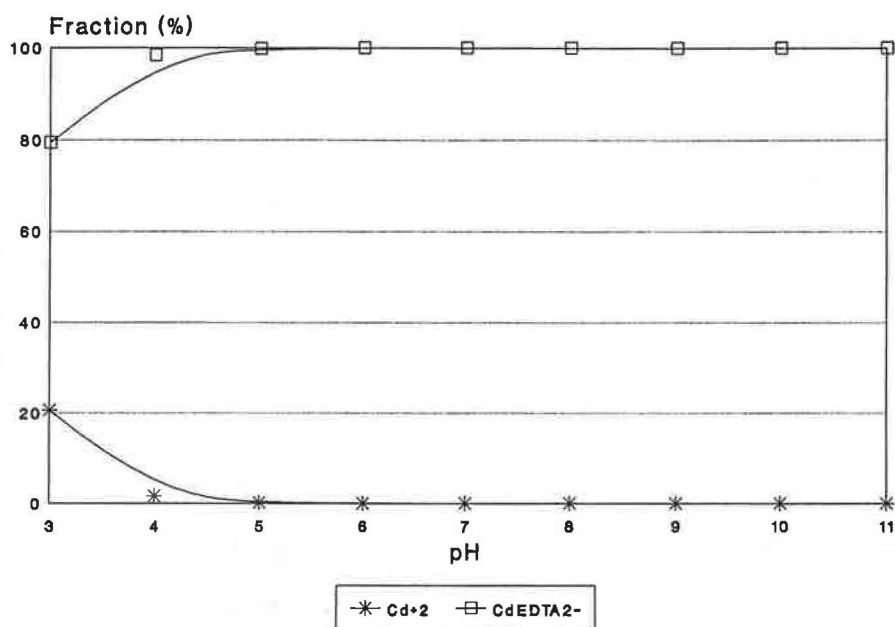


FIGURE 7 Speciation distribution of Cd in Cd-EDTA aqueous solution (0.5 M ionic strength,  $10^{-4}$  M Cd,  $10^{-4}$  M EDTA).

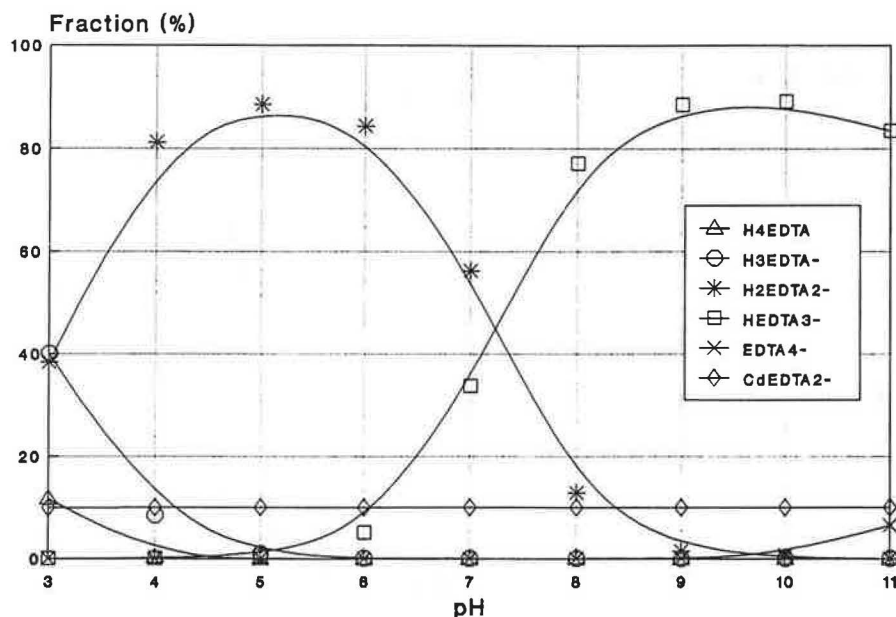


FIGURE 8 Speciation distribution of EDTA in Cd-EDTA aqueous solution (0.5 M ionic strength,  $10^{-4}$  M Cd,  $10^{-3}$  M EDTA).

EDTA existed as  $\text{CdEDTA}^{2-}$ . However, all other EDTA species were mostly  $\text{H}_2\text{EDTA}^{2-}$  (predominated at pH 3 to 7) and  $\text{HEDTA}^{3-}$  (predominated at pH 7 to 11) (Figure 8). Because the 90 percent free EDTA (either  $\text{H}_2\text{EDTA}^{2-}$  or  $\text{HEDTA}^{3-}$ ) might compete with  $\text{CdEDTA}^{2-}$  for surface adsorption sites of the soil, the removals of Cd were much less

than those at a Cd:EDTA ratio of 1:1 in the whole pH range. Moreover, the surfaces of soil particles were negatively charged in the pH region studied, and the repulsion between the  $\text{CdEDTA}^{2-}$  and the surfaces also probably decreased the removals of Cd. At Cd:EDTA ratios of 1:1 and 1:10, no  $\text{Cd}(\text{OH})_2$  precipitate formed in the whole pH range, according



to the calculations based on the hydrolysis equilibrium constants and the solubility product constant. This also contributed to the fact that the removals of Cd were so limited.

## CONCLUSIONS

The adsorbability of heavy metal by various soils under the same operation conditions was affected by the texture and the properties of soil, including the pH, organic matter content, and CEC. Although a little inconsistency in the adsorabilities of heavy metal ions by soils was observed among the seven soils tested in this study, a general trend was still noted. The order Hsinying soil (E) > Chunghsin Village soil (C) > Chung Hsing University soil (U)  $\approx$  Hsuehchia soil (K) > Taichung sanitary landfill red loam soil (R) > Mashan soil (M) > Tunghsiao soil (T) was concluded. Moreover, a general order of  $\text{Pb} \approx \text{Cu} > \text{Ni} \approx \text{Cd} > \text{Zn}$  was found for the sequence of adsorption removal by various soils. This result might have something to do with the hydrolysis equilibrium constants of heavy metal ion and the solubility product constant of heavy metal hydroxide.

When the Cd concentration was  $10^{-5}$  M, the adsorption removals were not greater than 10 percent in the whole pH range, and the removal increased with increasing Cd concentration. However, the  $\text{M}(\text{OH})_2$  precipitates were formed and removed at high pHs and high Cd concentrations. Thus, the speciation distribution of heavy metal ions in aqueous solution should be evaluated when the adsorabilities of heavy metals by soils are compared. The adsorption removal of Cd by red loam soil (R) was suppressed in the presence of organic ligand, EDTA, because of the strong complexation reaction between Cd and EDTA. Such suppression was more obvious when the ratio of EDTA:Cd was increased. At an EDTA:Cd ratio of 10:1, the removals of Cd were almost zero at all the pHs studied.

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# Evaluation of Recycled Concrete, Open-Graded Aggregate, and Large Top-Size Aggregate Bases

RICHARD D. BARKSDALE, SAMIR Y. ITANI, AND TERRY E. SWOR

Eight unstabilized aggregate bases were investigated: three recycled concrete materials, an open-graded crushed dolomite drainage material, large top-size crushed dolomite, dense-graded crushed dolomite, sand and crushed gravel blend, and a dense-graded crushed granite. Tests performed included repeated load triaxial tests to evaluate resilient modulus and rutting potential, Los Angeles degradation tests, CBR tests, Hveem stabilometer tests, and gradation tests to define aggregate breakdown due to compaction and repeated loading. The essentially  $\frac{3}{4}$ -in. top-size open-graded drainage material, which had about 0.9 percent fines, exhibited the best performance and was not sensitive to moisture effects. The large 1.5-in. maximum size crushed dolomite also exhibited outstanding performance and was only slightly sensitive to moisture. When minus  $\frac{3}{8}$ -in.-size crushed dolomite was added to one recycled concrete, better resilient modulus properties were observed than for the sand and crushed gravel blend; the other recycled concrete material exhibited slightly lower resilient modulus. All of the recycled concrete materials performed better than the sand and crushed gravel blend with respect to rutting potential. The recycled concrete materials, however, exhibited the most degradation with maximum observed levels of degradation being modest. Recycled concrete has been found by others to become significantly stiffer in the field with time, probably because of degradation effects or recementing.

Many concrete structures such as buildings, bridges, and pavements are being demolished as they reach the end of their life cycle or become obsolete. The quantity of this type of construction debris available will become greater in the future. Also, recently passed federal legislation requires that the quantity of material deposited in landfills must be significantly reduced by 1996. These and other environmental factors will encourage more emphasis to be placed on recycling of concrete, rubble, and other construction materials.

One important use of appropriately crushed and screened concrete construction debris is as unstabilized, engineered aggregate base. At the present time, little is known about the engineering characteristics of recycled concrete base, although some important work has been reported (*1*). The purpose of this paper is to compare the behavior of recycled concrete base with two conventional crushed stone bases, an open-graded base, a large top-size base, and a sand and gravel base. Tests performed included repeated load triaxial tests, CBR tests, Hveem stabilometer tests, and gradation tests.

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## MATERIALS TESTED AND TEST PROCEDURES

Table 1 summarizes selected physical properties of the eight bases studied. The gradation of the bases tested both before and after repeated load testing are included in this table. Materials were tested in accordance with the referenced AASHTO specifications. The gravel tested was crushed and had 48 percent of the particles with two or more fractured faces; sand was added to the gravel to give the desired base course gradation. All of the materials tested except one were from Minnesota and the gradations used conformed to Minnesota DOT (MnDOT) standards. The crushed granite included in the study was from Colorado.

### Repeated Load Test Specimen Preparation and Test Procedure

Specimen preparation was in general accordance with the recommendations of AASHTO Specification T-274-82 (which has been withdrawn and is being replaced). Cylindrical specimens of base material 6 in. in diameter by 12 in. high were prepared directly on the base of the triaxial cell. The material was compacted using a vibratory compactor. Complete specimen preparation and testing procedures are described in detail elsewhere (*2*). A second rubber membrane was placed around the specimen after preparation. The use of a second rubber membrane was necessary because the first membrane usually became punctured during the compaction process.

Some samples were tested at a high degree of saturation. To achieve a high degree of saturation, a small vacuum was applied to the top of the specimen through the top porous plate, and a water source was connected to the bottom porous plate. Water was circulated through the specimens until no air bubbles were observed coming out with the circulating water. All the specimens were tested in the fully drained condition by opening the drainage valve on the triaxial cell during the testing phase.

### Test Procedure

After sample preparation, specimens were conditioned for 1,200 load repetitions using the stress levels given in Table 2. Then the specimens were tested using the stress states given in Table 3. Both the elastic and permanent axial deformations

TABLE 1 SELECTED PROPERTIES OF AGGREGATE BASES TESTED

Material	1-1/2"	1	3/4	3/8	(1) Percent Passing				(2)		Absorption <sup>(3)</sup> (%)	Y <sup>(4)</sup> max (pcf)	Optimum Moisture Content (%)	
					No.4	No.10	No.40	No.200	L.A. Abr. (%)					
Sample 1 Dolomite Class 5	-	100	97	70	45	33	19	8.5	41.6	3.19	139	7		
	-	100	97.9	71.9	47.3	34.2	21.2	8.7						
Sample 2 Sand & Gravel Class 5	-	100	97	70	45	33	19	8.5	22.9	1.32	145	4.5		
	-	100	97.1	71	47	34	21	8.6						
Sample 3 Recycled Concrete I Class 5	-	100	97	70	45	33	19	8.5	37.1	5.56	123	8		
	-	100	97	74	49.4	36.8	22.7	9.2						
Sample 4 Recycled Concrete with 3/8 in. Minus Dolomite	-	100	97	70	45	33	19	8.5	37.2	5.9	135	8		
	-	100	98.1	72.1	47.9	34.4	21.7	9.3						
Sample 5 2 in. Minus Dolomite	100	89	72	46	29	20	14	7	39.7	2.8	144	6.5		
	100	90.6	76	48.9	31.4	21.7	15.3	7.6						
Sample 6 No. 67 Dolomite - Open-Graded Base	-	100	98.3	50.7	27.7	18.5	6.8	0.9	37.2	2.6	131	6		
	-	100	98.7	54.6	30.3	19.8	7.16	1.1						
Sample 8 Recycled Concrete II	-	100	97	70	45	33	19	8.5	38.3	4.94	128	8.5		
	-	100	97.1	73.6	46.5	34.6	21.2	9.1						
Material	3/4	1/2	3/8	#4	#8	#16	#30	#50	#100	#200				
Sample 7 Crushed Granite	100	94.6	73.8	38	27.3	22.3	18.7	14.6	9.7	6.8	34.6	1.72	140	5.5
	100	95.9	75.7	42.7	30.1	23.2	20.1	16.2	12.3	8.2				

Notes: 1. The first gradation given is before compaction and testing; the second one is after testing.

2. ASTM C-131.

3. AASHTO T-85 Specific Gravity Test.

4. AASHTO T-180 density test method.

5. Gradations were evaluated using dry sieve analysis.

were recorded after 200 load repetitions. After completing the first set of resilient modulus tests (i.e., completing the 20 stress states listed in Table 3), the confining stress was reduced to 6 psi and a deviator stress of 18 psi was then cycled until 8,600 load repetitions were completed. After that, another set of resilient moduli values were evaluated at the same stress levels previously used. Finally, using a confining pressure of 6 psi and the deviator stress of 30 psi, the sample was cycled up to a total of 70,000 load repetitions. At this stage the test was terminated and the total axial permanent deformation was recorded.

## TEST RESULTS

### Resilient Modulus

Figures 1 and 2 show the variation of the resilient modulus ( $M_R$ ) as a function of bulk stress  $\theta$  (i.e., the sum of the principal stress  $\sigma_1 + \sigma_2 + \sigma_3$ ) for the different testing conditions and materials studied. Tables 4 and 5 summarize the variation in resilient moduli due to the effect of material type and test conditions (compaction level, moisture level, and number of repetitions) at a bulk stress ( $\theta$ ) of 50 and 21 psi, respectively. A bulk stress of 21 psi represents approximately the average stress state that exists in the base layer for a moderately thick asphalt concrete surfacing, whereas a bulk stress of 50 psi is representative of thin asphalt concrete surfacing (3).

### Influence of Material Type

The materials tested are ordered from best to worst on the basis of their resilient moduli performance as follows:

Ranking		Sample Number	Material Type
( $\theta =$ 50 psi)	( $\theta =$ 21 psi)		
1	1	6	No. 67 dolomite—open-graded base (100%, 100%)
2	2	5	2 in. minus dolomite (90%, 99%)
3	4	1	Dolomite—Class 5 (80%, 70%)
4	3	7	Crushed granite (77%, 78%)
5	5	2	Sand and gravel—Class 5 (73%, 69%)
6	6	4	Recycled concrete with 3/8 in. minus dolomite (73%, 65%)
7	7	8	Recycled Concrete II—Class 5 (66%, 59%)
8	8	3	Recycled Concrete I—Class 5 (64%, 51%)

The percentages in parentheses for the ranked materials represent the relative resilient modulus performance of all the samples compared with the open-graded base that had the highest value (values for  $\theta = 50$  psi are given first). This relative performance is evaluated at 100 percent AASHTO T-180 density and 1,200 load repetitions. The following discussion is for  $\theta = 50$  psi.

TABLE 2 STRESS SEQUENCE USED IN TRIAXIAL TEST—  
CONDITIONING PHASE

Conditioning Phase	Number of Applications	Confining Stress (psi)	Deviator Stress (psi)
1	200	5	5
2	200	5	10
3	200	10	10
4	200	10	15
5	200	15	15
6	200	15	20

TABLE 3 STRESS SEQUENCE USED IN TRIAXIAL TEST—  
RESILIENT MODULUS STRESS STATES

Stress State	No. Applications (1)	Confining Stress (psi)	Deviator Stress (psi)	Bulk Stress $\sigma$ (psi)
1	200	20	10	70
2			20	80
3			30	90
4			50	110
5		15	10	55
6			20	65
7			30	75
8			50	95
9		10	10	40
10			20	50
11			30	60
12			40	70
13		5	5	20
14			10	25
15			15	30
16			20	35
17		3	3	12
18			6	15
19			9	18
20			12	21

Note: (1) 200 load applications were used for each of the twenty stress states.

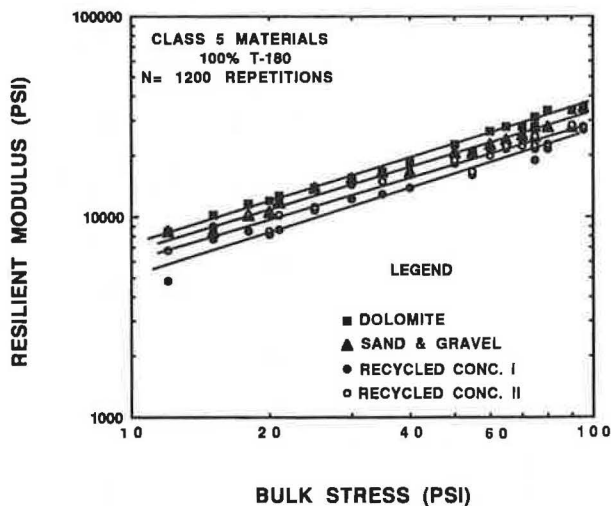


FIGURE 1 Resilient modulus response of Recycled Concrete I and II compared with sand and crushed gravel and dolomite base materials.

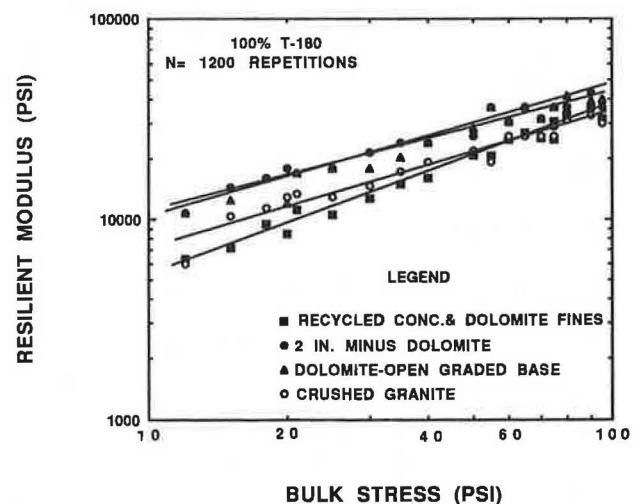


FIGURE 2 Resilient modulus response of recycled concrete with dolomite added compared with large top-size dolomite, open-graded dolomite, and crushed granite base materials.

TABLE 4 SUMMARY OF RESILIENT MODULUS TEST RESULTS AT BULK STRESS OF 50 psi

Material <sup>(3)</sup>	100% T-180 <sup>(2)</sup>		95% T-180 <sup>(2)</sup>		Soaked (100% T-180)	
	N=1200	N=8600	N=1200	N=8600	N=1200	N=8600
Dolomite: No.1 <sup>(1)</sup> (Class 5)	23,000	26,000	21,000	24,000	18,000	18,000
Sand & Gravel (Class 5): No.2	21,000	24,000	17,200	18,500	13,500	15,000
Recycled Concrete (Class 5)						
Source I: No.3	18,500	21,600	13,500	15,000	13,000	13,600
Source II: No.8	19,200	22,400	15,500	16,800	13,400	14,400
Recycled Concrete with Dolomite Fines: No.4	21,000	24,000	19,200	19,000	17,240	20,000
2 in. Dolomite (Modified Class 5): No.5	26,000	28,000	20,000	24,000	24,000	28,000
Dolomite - Open-Graded Base: No.6	28,800	28,800	24,000	28,800	24,000	24,000
Granite: No.7 (Colorado Base)	22,000	23,000	17,000	20,800	19,500	21,000

Note: 1. Sample number given in Table 1.  
2. Tested at optimum water content.  
3. The Class 5 base gradations are from the state of Minnesota specifications.

TABLE 5 SUMMARY OF RESILIENT MODULUS TEST RESULTS AT BULK STRESS OF 21 psi

Material	100% T-180		95% T-180		Soaked 100% T-180	
	N=1200	N=8600	N=1200	N=8600	N=1200	N=8600
Dolomite (Class 5): No.1	12,000	10,500	10,000	10,500	9,100	13,000
Sand & Gravel (Class 5): No.2	11,800	11,200	9,700	11,000	8,000	9,000
Recycled Concrete (Class 5)						
Source I: No.3	8,700	9,800	7,600	8,200	6,800	8,100
Source II: No.8	10,100	10,100	7,500	8,900	7,400	7,400
Recycled Concrete with Dolomite Fines: No.4	11,200	11,200	9,200	9,200	9,300	9,300
2 in. Dolomite (Modified Class 5): No.5	17,000	17,200	10,800	11,400	16,000	16,000
Dolomite - Open-Graded Base: No.6	17,200	19,000	-	-	-	-
Granite: No.7 (Colorado Base)	13,400	13,400	10,400	10,400	10,400	11,400

Note: 1. Sample number given in Table 1.  
2. Tested at optimum water content.  
3. The Class 5 base gradations are from the state of Minnesota specifications.



The open-graded base (Sample 6), which had only 0.9 percent fines (material passing the No. 200 sieve), has the highest values of resilient moduli. The large, 2-in. top-size dolomite (Sample 5) exhibited the next-to-highest resilient moduli. The three recycled concrete materials exhibited the lowest resilient moduli and hence the poorest resilient moduli behavior. The recycled concrete, which had minus  $\frac{3}{8}$ -in.-size particles of crushed dolomite added, performed the best of the recycled concrete materials with respect to resilient modulus.

The recycled concrete with minus  $\frac{3}{8}$  in. dolomite particles performed the same as the sand and gravel blend. The sand and gravel blend was crushed, with about 48 percent of the gravel particles having two or more fractured faces. Recycled Concrete II, which performed slightly better than Recycled Concrete I base material, had a resilient modulus about 10 percent less than the sand and crushed gravel blend. The recycled concrete bases, therefore, would be expected to have resilient moduli similar to those of uncrushed sand and gravel blends. In Minnesota an uncrushed sand and gravel blend would serve as the basis of comparison for a conventional base.

The response for  $\theta = 21$  psi (Table 5), for practical purposes, is similar to that for  $\theta = 50$  psi except that less difference is present in  $M_R$  values for the open-graded and large top-size bases. These two bases clearly did quite well. An important difference existed between these two excellent materials and the poorer performing ones.

#### *Influence of Compaction Level*

The materials were tested at two levels of density: 95 and 100 percent of AASHTO T-180 (refer to Tables 4 and 5). The effect of density on the resilient modulus can be relatively large, with a decrease in the resilient modulus occurring as the compaction level is reduced from 100 to 95 percent of AASHTO T-180. For the material tested, the results (Table 5) indicate that at a low bulk stress of 21 psi, the decrease in the modulus varied from 13 to 36 percent when the density was decreased from 100 to 95 percent of T-180. At a high bulk stress of 110 psi (2), the reduction in the resilient modulus was less pronounced, but still important, varying from 10 to 25 percent for the same decrease in the original density.

#### *Influence of Degree of Saturation*

The resilient modulus is known to decrease with increasing degree of saturation (3,4). In these tests (2) at a low bulk stress of 12 psi, the decrease in the resilient modulus varied from 2 to 50 percent when the water content of the base increased from the optimum value to a high degree of saturation. At high bulk stress, the reduction in the resilient modulus varied from 3 to 35 percent for the same increase in water content. The degree of saturation that was reached, as determined at the end of the test, was material and gradation dependent and varied from 87 to 100 percent. The large top-size and open-graded base materials were least moisture sensitive and hence performed best when wet.

#### *Influence of Number of Load Repetitions*

The resilient modulus ( $M_R$ ) was evaluated for the eight materials tested at both 1,200 and 8,600 load repetitions. The general trend observed for  $\theta = 50$  psi was generally a slight increase (0 to 20 percent maximum) in the resilient modulus due to the increase in the number of load repetitions (Table 4). The increase in  $M_R$  with load repetitions is hypothesized to be partly due to a slight increase in density or decrease in water content, or both. The  $M_R$  for the open-graded dolomite apparently increased with increasing repetitions entirely because of the effects of density. When initially compacted to 95 percent of T-180 density, the  $M_R$  of this material increased by about 20 percent with increasing load repetitions. When placed initially at 100 percent of T-180 density, a change in  $M_R$  was not observed.

For  $\theta = 21$  psi (Table 5), the increase in resilient modulus of the open-graded and large top-size bases was generally less than at  $\theta = 50$  psi, and for the poorer-performing materials frequently no increase at all was observed for  $M_R$ .

#### **Permanent Deformation Test Results**

The approximate ranking of the base materials with respect to permanent deformation behavior and observed rut indices are summarized in Table 6, and the behavior for selected bases is compared graphically in Figure 3. The rut index is used for comparing the permanent strain behavior, with the rut index being approximately proportional to rutting in the material (3,4). For this study the rut index was taken as the permanent strain observed in the tests, which were carried out until 70,000 repetitions, multiplied by 10,000.

The open-graded dolomite (which had only 0.9 percent fines) and the oversize minus 2-in. dolomite performed best with respect to permanent strain. The Class 5 sand and gravel performed poorly compared with all the other materials. The Class 5 recycled concrete without dolomite fines, both Sources I and II, performed about the same as the comparable Class 5 dolomite base. In fact, Source II recycled concrete specimens appeared to perform slightly better than the Class 5 dolomite. The Class 5 recycled concrete with minus  $\frac{3}{8}$ -in.-size dolomite appeared to perform slightly poorer with respect to permanent strain than the other Class 5 recycled concrete bases and the Class 5 dolomite base. This difference in performance, however, was small and probably not statistically significant. Average permanent strain is proportional to permanent deformation.

For the materials tested, the increase in the permanent strain varied from 47 to about 75 percent when the compacted density was reduced from 100 to 95 percent of AASHTO T-180 density. The degree of saturation also has a significant effect on the rutting potential, with an increase in the permanent deformation occurring as the water content increases. For the materials tested the increase in the rutting potential, as defined by measured permanent strain and expressed by the rut index, varied from 10 to about 45 percent as the water content was increased from optimum to near saturation.

Sources I and II of the Class 5 recycled concrete both appeared to be very slightly more susceptible to permanent strain

TABLE 6 VARIATION OF RUT INDEX WITH DENSITY AND MOISTURE FOR AGGREGATE BASE MATERIALS TESTED

Rank	Material	Rut Index <sup>(1)</sup>		
		100% T-180	95% T-180	Soaked 100% T-180
1	Dolomite (open-graded base): No.6 <sup>(2)</sup>	29	48.6	32
2	2 in. Dolomite (Modified Class 5): No.5	38.5	56	42
3	Granite (Colorado Base): No.7	41	62	54
4	Recycled Concrete Source II: No.8	51	86	81
5	Recycled Concrete Source I: No.3	55	95	80
6	Dolomite - Class 5: No.1	55	92	76
7	Recycled Concrete with Dolomite Fines: No.4	58.6	100	82
8	Sand and Gravel - Class 5: No.2	62	102	91

Notes: 1. Rut Index is evaluated as the permanent strain observed after 70,000 load repetitions. Rut Index is approximately proportional to the rutting that would be expected in the base course [3,4].

2. These numbers indicate the sample number given in Table 1.

than the comparable Class 5 dolomite, particularly when wet. The sand and crushed gravel blend studied in this investigation was more susceptible to rutting than the other materials when tested at a high degree of saturation. The open-graded material had the least rutting susceptibility among all the materials tested at a high degree of saturation.

### CBR and R-Value

Table 7 compares the results of soaked CBR tests, R-values determined from Hveem stabilometer tests, and resilient moduli test results. Sources I and II of the Class 5 recycled concrete performed as well as or better than the comparable Class 5 dolomite base as indicated by the CBR and R-value results, and better than the Class 5 sand and crushed gravel blend.

### Degradation

Degradation of base materials due to the combined effects of compaction and repeated loading to 70,000 repetitions was evaluated by comparing the grain size distribution obtained after testing with the original distribution using dry sieve analysis. The results are presented in Table 1 with the gradation before compaction given first and the gradation after compaction and testing given on the line below it. Only modest changes occurred in specimen gradation during specimen preparation and testing. The Class 5 sand and gravel performed best in terms of degradation, and the materials containing dolomite also performed well. The Class 5 recycled concrete (both sources) exhibited the highest degradation of particles compared with the other materials. All specimens

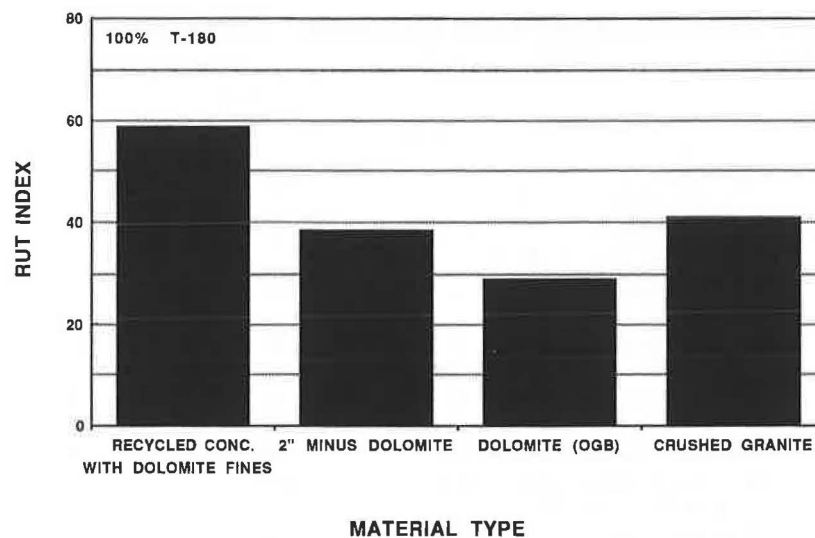
were prepared by vibratory compaction, which causes the least particle breakage compared with impact methods of laboratory compaction.

### DISCUSSION OF RESULTS

The MnDOT Class 5 recycled concrete base (all three types), open-graded dolomite drainage layer base, and large top-size dolomite base are all materials that are not widely used in pavement construction at this time. Recycled concrete will undoubtedly be used more for base and in other applications in the future as aggregate and disposal costs both rise. Proper drainage of water from beneath a pavement significantly increases the life of a pavement, which accounts for the considerable interest at the present time for using open-graded drainage layers. Rapid removal of water, as shown elsewhere (5) and in this study, increases the stiffness of the pavement and decreases susceptibility to rutting. Considerable interest also exists in using large top-size stabilized and unstabilized base mixes to reduce rutting. Therefore, an examination of the engineering characteristics of these unstabilized bases is timely.

### Recycled Concrete

The recycled MnDOT Class 5 concrete base, both Sources I and II, performed comparably with the standard MnDOT Class 5 dolomite base with respect to rutting. Although the recycled concrete with dolomite fines exhibited a slightly greater tendency to rut than did the Class 5 recycled concrete and the Class 5 dolomite, the resilient moduli characteristics of the recycled concrete/dolomite blend were superior to the



**FIGURE 3** Comparison of rut index of recycled concrete (dolomite added) with granite base and large top-size and open-graded dolomite base materials.

**TABLE 7** COMPARISON OF RESILIENT MODULUS, SOAKED CBR, AND R-VALUES FOR AGGREGATE BASE MATERIALS TESTED<sup>1</sup>

Material	Soaked CBR	$M_R^{(2)}$ (N = 1200)	R-Value
Dolomite (Class 5) <sup>(3)</sup>	162	23,000	85
Sand & Gravel (Class 5)	85	21,000	79
Recycled Concrete (Class 5)			
Source I	170	18,500	84
Source II	270	19,200	85
Recycled Concrete with Dolomite Fines	95	21,000	84
2 in. Dolomite (Modified Class 5)	350	26,000	84
Dolomite (Open-Graded Base)	137	28,800	79
Granite (Colorado Base)	126	22,000	71

Note: 1. Specimens prepared at 100% of AASHTO T-180 density.  
 2. Resilient Modulus determined at a bulk stress of 50 psi.  
 3. These numbers indicate the sample number given in Table 1.

straight recycled concrete base and only slightly less than for the MnDOT Class 5 dolomite base. Therefore, from the standpoint of overall performance, the use of minus  $\frac{3}{8}$ -in.-size dolomite with the recycled concrete is probably desirable. Replacement of the finer portion of the recycled concrete with dolomite would certainly be desirable if the 1986 AASHTO design guide (6) is used as the sole basis for design, since it is based entirely on resilient moduli without any direct consideration being given to rutting. Therefore, an important practical limitation of the 1986 AASHTO design guide is that rutting of not only the base but also other layers is not adequately considered.

The increase in fines (material passing the No. 200 sieve) due to compaction and cyclic loading was between 0.6 and 0.8 percent of the three recycled concrete materials studied; the amount of fines before compaction in the specimen was 8.5 percent. Although the level of degradation observed is not considered serious, it was the highest measured and justifies further investigation, including the effects of degradation on frost action.

Field test results indicate that the stiffness of full-scale pavements constructed with crushed concrete, crushed rubble with slag, and crushed rubble substantially increases with time, whereas crushed masonry has been found to increase only

slightly with time (*I*). The increase in stiffness may be due to rehydration or the increase in fines, which occurs as the aggregate degrades with increasing number of wheel loads. The importance of rehydration with time is unclear. However, after about 2 years from the time of pouring, usually less than about 5 percent unhydrated cement particles remains. The general lack of unhydrated cement particles suggests that rehydration may not be the only cause of the observed increase in stiffness with time. Because of the stiffening of this type base with time, short-term laboratory resilient modulus test results, such as those described in this paper, are considered to be on the conservative side.

Reflective properties and color uniformity of recycled concrete are poor. Non-air-entrained concrete may have reduced freeze/thaw durability compared with air-entrained concrete or natural aggregate. Fines generated from the recycled concrete have been found in some instances to clog up filter-fabric wrapped subsurface drains. Recycled concrete has a limited history of use, and observed field performance has been variable. This material can contain extraneous debris such as metal, wood, brick, and organics.

Because the leachate from recycled concrete is highly alkaline (concrete has a pH of 11.5 to 12.5), caution should be exercised to satisfy applicable state and federal regulations concerning pollution.

If aluminum contaminants, such as conduit pipe, are present within the recycled concrete, the high pH of the concrete can cause accelerated corrosion and formation of expansive products and hydrogen gas. At a pH between 4.66 and 6.32, aluminum forms an oxide film of aluminum, which protects the aluminum from further oxidation. Corrosion of aluminum occurs outside of these pH limits.

### Open-Graded Drainage Material

The open-graded dolomite drainage layer material exhibited the best resistance to rutting of all the materials investigated (Table 6) and the highest (best) resilient moduli values (refer, for example, to Tables 4 and 5). When properly placed and subjected to confinement, this drainage layer material should exhibit excellent structural performance and good drainage characteristics. The open-graded drainage material used in this study had a 1-in. top size, about 28 percent passing the No. 4 sieve, and about 1 percent fines (unwashed). This gradation has a smaller top size and is finer than the extremely open-graded materials that have sometimes been used for drainage layers. The finer gradation and smaller top size appear to account for the very good structural performance of this material. Even though a reasonably fine gradation was used, the permeability should be about 200 to 400 ft/day, which is approximately 400 times greater than the conventional Class 5 base as determined by MnDOT permeability tests.

### Large Top-Size Base

The large top-size dolomite base material, with the top size increased from 1 in. to 1.5 in., reduced the rutting potential

(Table 6) when compared with the conventional Class 5 gradation, which for practical purposes has a 0.75-in. top size. When well-compacted to 100 percent AASHTO T-180 density, the larger top-size dolomite base also exhibited higher resilient moduli than the conventional Class 5 base (Tables 4 and 5). Therefore, use of the larger top-size gradation should give improved pavement performance, which has also been found true for other aggregates and gradations (7).

### Granular Equivalency

The granular equivalency is defined for this discussion as the ratio of the AASHTO layer coefficient ( $a_2$ ) of a selected base aggregate to the layer coefficient ( $a_2$ ) of the sand and crushed gravel blend used in this study. For example, a base with a granular equivalency of 1.25 means that 1.25 in. of sand and gravel base can be replaced by 1 in. of the selected base. The granular equivalency can be estimated by comparing the base layer coefficient ( $a_2$ ) of a selected base material with the base layer coefficient of the MnDOT Class 5 standard sand and gravel. For this comparison the  $a_2$  layer coefficients were determined using the AASHTO pavement design guide (6) relationship between the laboratory-determined resilient modulus and  $a_2$ . An alternative granular equivalency factor was also evaluated on the basis of the rutting potential of a selected material compared with the MnDOT Class 5 sand and gravel. Table 8 gives the granular equivalencies based on both the resilient modulus ( $\theta = 50$  psi) and rutting potential. On the basis of resilient moduli, the Class 5 recycled concrete (Source I) has the lowest granular equivalency of 0.92. The open-graded base has the best (highest) granular equivalency on the basis of both the resilient modulus (1.39) and the rutting potential (2.13).

### Sensitivity of $M_R$ on Base Thickness

To illustrate the potential influence of variations in the resilient modulus ( $M_R$ ) between the different base materials tested, a 1986 AASHTO pavement design (6) was developed for each different base studied (Table 8). For this illustrative example the structural base coefficient ( $a_2$ ) of the Class 5 sand-gravel base used in this study was assumed to be 0.14, and the relative structural base coefficient was determined for the other materials. A fair subgrade was assumed having a resilient modulus of 5,000 psi. Other factors used in the analysis are summarized in Footnote 1 at the bottom of Table 8. An asphalt concrete surface layer thickness of 3 in. was used for each design. The base thickness, which reflects the influence of the variation in resilient modulus of the base with material type, was determined. Required base thicknesses varied from 14 to 24 in. for a reliability of 80 percent and from 12 to 20 in. for a reliability of 50 percent. These designs do not consider the influence of difference in rutting potential, which should also be taken into account. Of significance is the fact that the gravel tested was crushed and had 48 percent of the particles with two or more fractured faces. Hence, the sand-gravel tested was better than the uncrushed gravel usually used in Minnesota.



TABLE 8 GRANULAR EQUIVALENCIES AND REQUIRED LAYER THICKNESS BASED ON RESILIENT MODULUS AND RUTTING DETERMINED FROM REPEATED LOAD TESTING

Material Type	Granular Equivalency		Base Layer Thickness (in.) <sup>(1)</sup>	
	Based on $M_R$	Based on Rutting	80% Reliability	50% Reliability
Dolomite (Class 5): No.1 <sup>(2)</sup>	1.11	1.14	20	16
Sand & Gravel (Class 5 - Crushed): No.2	1.00	1.00	22	19
Recycled Concrete (Class 5)				
Source I: No.3	0.92	1.14	24	20
Source II: No.8	0.98	1.21	22	19
Recycled Concrete with Dolomite Fines: No.4	1.00	1.09	22	19
2 in. Dolomite (Modified) (Class 5): No.5	1.27	1.61	17	15
Dolomite - Open Graded Base: No.6	1.39	2.13	14	12
Granite (Colorado Base): No.7	1.06	1.52	21	17

Note: 1. Assumptions made for illustrative example: 3 in. AC Surfacing; Terminal Serviceability of 2.5;  $w_{18} = 2.5 \times 10^6$ ; reliability = 50 and 80%; Overall standard deviation = 0.45; initial serviceability = 4.5; final serviceability index = 2.5; serviceability loss due to frost heave and swelling = 0.64.

2. These numbers indicate the sample number given in Table 1.

## SUMMARY

The following conclusions can be drawn from the findings of this study:

1. A wide range in performance was observed between the eight different aggregate base materials tested. Hence, care must be exercised in selecting an aggregate base that will be suitable for an intended application. Whether the materials are compacted to 95 or 100 percent of AASHTO T-180 density is also important for most materials studied as well as the level of moisture present.

2. The stiffness characteristics of an aggregate base, as determined by the resilient modulus, are not necessarily related to rutting behavior as defined by permanent strain measured in the repeated load triaxial test. For example, the recycled concrete exhibited lower resilient moduli compared with the other type bases after 8,600 load repetitions. The recycled concrete, however, performed better in permanent strain (rutting) than the sand and gravel (crushed) and about the same as dolomite aggregate base.

3. Both the laboratory test results and previous field measurements indicate that a base constructed with recycled concrete should become significantly stiffer with time, perhaps because of rehydration or a small increase of fines. The recycled concrete (Sources I and II) should perform on the average as well as an uncrushed sand and gravel base, and probably about as well as a sand and gravel base that has been crushed. Experience indicates that recycled concrete has a more varied performance record than for other base materials.

4. The essentially 0.75-in. top-size open-graded dolomite having 0.9 percent fines performed, overall, the best with respect to both resilient modulus and permanent strain characteristics of the eight bases studied. The resilient modulus of the open-graded drainage material was not sensitive to the effects of water; rutting showed only a slight moisture sen-

sitivity. The other materials were sensitive to water but to varying degrees. The large 1.5-in. top-size crushed dolomite performed next to best, showing only a slight sensitivity to water.

## ACKNOWLEDGMENT

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# Recycling Project: Concrete Grinding Residue

SCOTT GOODWIN AND MICHAEL W. ROSHEK

In spring 1990 the Utah Department of Transportation (UDOT) was responsible for disposing of 3,200 yd<sup>3</sup> of alkali solid waste and 891,000 gal of wastewater resulting from a large scale pavement grinding project (12 lane-mi on I-15). Numerous disposal methods were considered and reviewed for both cost and potential for success. Detailed laboratory testing was conducted as part of the preliminary investigation. UDOT worked closely with the Utah Division of Environmental Health in arriving at a solution. Ultimately, a decision was reached to recycle the solid waste into a road project, where it would serve as the mineral filler in a portland cement-treated base course. Disposal project phases included initial grinding, characterization of the waste material, heuristic evaluation of possible stabilization methodologies, wastewater treatment and solids interim storage, and final project design and placement. Care was taken to ensure that the material and wastewater were handled in an environmentally safe manner. Water was decanted from the grinding slurry, filtered to remove the suspended solids, and treated with sulfuric acid to lower the pH before disposal. A major emphasis in the pavement structural section design was to eliminate the possibility of future ground-water contamination on-site.

In August 1989, the Utah Department of Transportation (UDOT) had contracted to apply a proprietary polymer-modified concrete as a thin bonded overlay on 12 lane-mi of one of the busiest highways in Utah (128,000 AADT) (I), northbound Interstate 15 through Salt Lake Valley. The overlay's purpose was to eliminate ruts and rehabilitate a portland cement concrete (PCC) pavement that was almost 25 years old. The overlay varied from  $\frac{3}{4}$  in. in the wear ruts to feather edge thickness at the lane lines.

The new overlay had been completed for less than a month when portions began to delaminate. After 4 months 15 percent had come off, and more was in the process of doing so. This project had been very high profile and controversial from inception. Consequently, media coverage had also been profuse, and anything related to the I-15 overlay was politically sensitive. When the material failed, the media's coverage became more extensive, and the political atmosphere became even more charged.

By April 1990, 6 months after initial placement, UDOT was involved in a full-scale grinding project to remove any remaining overlay as well as to correct the still-existent rutting and faulting of the original pavement. The grinding project generated 891,000 gal of alkali wastewater and 3,200 yd<sup>3</sup> of alkali solid waste, all of which required treatment and disposal.

## GRINDING PROCESS

UDOT has, for many years, been involved in small-scale bump grinding projects of PCC pavement. But a grinding project of this magnitude, removing the overlay and the original concrete to an average depth of  $\frac{3}{4}$  in. on more than 750,000 ft<sup>2</sup>, was a unique experience.

With bump grinding, UDOT practice had been to allow the contractor to let the grinding slurry flow onto the shoulders or into the median swales. The project started out with the same concept; however, four grinding machines were mobilized for the project, removing considerably more material than the localized bump grinding. Because of the volume of material, the project was to be a turning point for this practice.

The overlay removal began on April 6, 1990. A few days later a representative of the Salt Lake County Health Department came on-site, took a grab sample from the end of the hose coming off the grinder, had the slurry water analyzed, discovered the total suspended solids (TSS) and pH of 12.0 to be substantially above acceptable limits, and had a restraining order in effect on April 11. The water was evaluated by Ford Analytical Laboratories. The results are given in Table 1. The high levels of heavy metals originated from the aggregate used in the overlay, which was slag from the Kennecott copper smelting process. It was also noted that a local hazardous site of cement kiln dust had similar characteristics. The project halted, and UDOT was instructed to "cease and desist" until it could devise and integrate a means for containing the slurry. A plan for final disposal of the slurry had to be developed, subject to final approval by the Utah Division of Environmental Health (DEH). In addition, instructions were given to clean up the slurry previously placed in the median and shoulders. Once containment of the grinding slurry was addressed, UDOT was allowed to continue grinding, with the understanding that completion of the disposal plan was pending.

## GRIND WATER TREATMENT

Work was restricted for 10 days while agreements were being reached between UDOT and DEH. When grinding operations recommenced on April 21, the slurry was pumped directly from the grinder into on-site tanker trucks. The trucks then hauled the slurry to one of two sites for water treatment and temporary storage of the grinding sediment. (Initially, two storage sites were used. Circumstances, however, dictated the eventual movement of everything to a single site.) Both

TABLE 1 WATER QUALITY ANALYSIS COMPARISON\*

Parameter	Concentration (ppm, except pH)	
	Off Grinder:	Values Acceptable to WWTP Before Discharge :
pH	12.6	7 to 9
TSS	> 50	45
Total Copper	1558	9.1
Total Arsenic	84.63	0.82
Total Barium	433	8.1
Total Chromium	132.37	4.0
Total Lead	318.99	5.0
Total Silver	1.09	3.1
Total Mercury	0.1787	0.03

\*Tested by Ford Analytical Laboratories, Inc.

sites were decommissioned wastewater treatment plants (WWTPs).

On arrival at the storage-treatment sites, the tanker trucks discharged their loads into empty sludge drying beds. A hydraulic gradient was created, and the grinding water flowed across the adjacent beds, over or through the sidewalls, to the endmost drying bed. At this location the water was filtered through filter fabric twice, then pumped through a 200- $\mu$ m screen into UDOT-manufactured bins, where sulfuric acid was added to reduce the pH. The sediment that remained was to be handled later. At this time a method for sediment disposal had not been agreed on by UDOT and DEH. A number of possibilities were still being considered.

As previously stated, the pH value of the slurry as it came off the grinder was high: 12.6. After sedimentation, filtration, and titration the pH of the slurry water was lowered to less than 8.5, well within the range (7.0 to 9.0) needed for delivery to the WWTP. Once the pH was acceptable, the water was pumped from the acid treatment bins into empty, recently washed tanker trucks and hauled to an operating WWTP for final discharge.

After UDOT treatment, the water was of such good quality that the highway agency would have had no difficulty meeting state water quality requirements for discharge. However, the constraint of a National Pollution Discharge Elimination System (NPDES) permit to allow us to do so arose. UDOT did not have the waiting time (60 to 120 days) necessary to apply for and receive one. Therefore, arrangements with an operating wastewater facility having a current NPDES permit were made.

After arrival at the WWTP, the grinding wastewater was discharged into the inflow and then through each of their unit operations and processes before ending up as effluent in the Jordan River, which flows into the Great Salt Lake. The water delivered was of better quality than the limits allowed for discharge from the treatment plant. Almost 891,000 gal were treated by this WWTP. The bill for treating the water was approximately \$10,000. Costs will be discussed later.

#### MEDIAN AND SHOULDER CLEANUP

At the same time that grinding and water treatment were under way, a cleanup operation was under way in the median of I-15. Front-end loaders were removing the previously placed

TABLE 2 HYDROMETER ANALYSIS OF GRINDING RESIDUE

Size (MM)	Sieve	Percent Passing
4.750-----	#4	100.00
2.000-----	#10	92.38
0.425-----	#40	86.61
0.075-----	#200	70.39
0.020		51.14
0.010		41.04
0.002		17.59
0.001		7.49
Specific Gravity:		2.59

grinding residue, along with several inches of soil and gravel, vegetation, automobile parts, and miscellaneous debris; loading them into dump trucks; and hauling them away to one of the two storage sites. Approximately 1,600 yd<sup>3</sup> of material was removed from the median in this manner. A like quantity of grinding sediment remained in the sludge drying beds after water treatment and decanting.

More than 3,200 yd<sup>3</sup> of material had to be disposed of, half of which was a sludgelike material unlike anything UDOT had ever dealt with before. The pH was high, in the range of 12.0 to 12.6. The material contained large volumes of water, depending on the time elapsed since placement and subsequent evaporation. Moisture contents in the range of 140 percent were common. Size of the individual particles was similar to a silt or clay and is given in Table 2.

The material also displayed thixotropic behavior. During preliminary testing activities, buckets of the material were retrieved from the drying beds. While being shoveled, the grindings were semisolid, sticky, and plastic. Once back in the laboratory, however, subjecting the grindings to small amounts of mechanical agitation caused the material to return to the liquid phase in a matter of minutes.

Numerous suggestions were offered on how to deal with the solids. A few were seriously considered and examined. UDOT's major objective was either to stabilize the waste product solids, rendering them essentially inert, or to dispose of them in such a manner as to minimize the possibility of ever having them resurface as a future problem.

The options reviewed included the following:

1. Lower the pH of the grinding solids in situ and leave them in the sludge drying beds, where they would eventually

be covered over, becoming fill material for a future parking lot. This option required the drilling of observation wells to monitor groundwater for an indefinite period of time.

2. Lower the pH of the solids and haul them to a licensed landfill for disposal. Landfill personnel would monitor groundwater conditions indefinitely.

3. Leave the residue in the sludge drying beds and stabilize the particles into either a soil-cement mixture or a lean concrete mix, once again to serve as fill for a future parking lot. No groundwater monitoring would be required with this option.

4. Move the waste material from the present storage site as is and recycle it into a construction project.

Option 2, taking the waste to a landfill, was almost immediately rejected. The landfill fee was prohibitive, around \$130,000. This left Options 1, 3, and 4. Options 1 and 3 were seriously considered and investigated: mix designs were determined; cylinders were made; 7-, 14-, and 28-day compressive strength tests were conducted; logistical and economical analyses were carried out; equipment was hired; test runs were performed; and so forth. Results of the various mix designs are given in Tables 3 and 4. At the same time, Option 4 was being investigated. A number of upcoming construction projects were being scrutinized for possible methods of grinding solids inclusion.

After numerous investigations and many meetings between UDOT and DEH engineers, the disposal method selected was Option 4, incorporation of the waste into a construction project.

ect. A project was on the drawing board that was just right: small enough to delay temporarily while the design was modified, yet large enough to use all of our grinding solids.

## WASTE RECYCLING CONSTRUCTION PROJECT

All the solids were finally moved to one storage-treatment site. The 3,200 yd<sup>3</sup> of material were to remain at this location until being recycled into more than 1/2 mi of portland cement-treated base course (CTB).

The project selected for solids recycling incorporation was a 0.6-mi-long strength of frontage road west of Salt Lake City designed to accommodate heavy commercial truck traffic. Stations 182 + 47 to 206 + 50 (2,403 ft) exclusively were to contain the recycled solids. The solids were to go in the proposed pavement structure as mineral filler for a 12-in. CTB, permitting a reduction in the total thickness required. On top of this would follow 6 in. of granular borrow, 12 in. of untreated base course, 7.5 in. of bituminous surface mix, and 1 in. of plant mix seal.

The construction site was an alkali flat (a remnant of ancient Lake Bonneville): relatively level, high groundwater, high alkalinity, ponds of standing water, and salt-tolerant riparian vegetation. Drilling logs taken profiled the soil as ranging from sandy silt with gravel to clayey silt with sand, or silty clay. During drilling operations, groundwater would begin to enter the auger hole near 4 ft deep, and the zone of saturation would usually be encountered around 8 ft deep. Values of pH

TABLE 3 SOIL-CEMENT MIX DESIGN COMPRESSIVE STRENGTHS

### Soil-Cement Mixes

Cement: Type II low alkali  
Coarse Aggregate: grinding sludge

Compressive Strengths: Cement content/cu. yd.	12 bag	11 bag	9 bag	6 bag	4 bag
Cure Time					
1 day	376	396	255	41	20
2 day	2250	837	238		
5 day				219	103
7 day	3979	1870	561		131
14 day	4416	2407	676	462	239
28 day	5411	3183	940	629	318

TABLE 4 CONCRETE MIX DESIGN COMPRESSIVE STRENGTHS

	SAMPLE NUMBER					
	NO. 1	NO. 2	NO. 3	NO. 4	NO. 5	NO. 6
Cement-Type	III	III	III	II	II	III
Bags/Cu. Yd.	10	4	3	3	2	3
Grinding sludge %	30	40	28	50	50	58
Coarse Aggregate %	70	30	36	25	25	
Fine Aggregate %		30	36	25	25	42*
CURE TIME	COMPRESSIVE STRENGTHS					
1 DAY		239		32		68
3 DAY	540					
4 DAY			776			
5 DAY	754	1014				203
7 DAY			861	155	40	356
14 DAY	1164	1353	1154	204	60	
28 DAY	1468	1671	1440	314	103	

\*Fine aggregate consisted of grindings and surface median material

for the groundwater ranged from 7.6 to 8.6. The pH of the soil was found to be close to 9; values are given in Table 5.

The prevalence of free water at the site and the possibility of groundwater contamination were major concerns. UDOT's roadway section design reflects the attention given to this matter. The solid particles were stabilized in a CTB; 6 in. of free-draining granular material was placed under and adjacent to the CTB as a capillary break; perforated drain pipe was placed in and a filter fabric was placed below the granular material to assist in stabilizing the subgrade; the CTB was prime coated top and sides; and finally a 15-mil-thick polyethylene sheet was placed over the top of the prime coat.

Construction of the treated base course proceeded in the following manner. After clearing, grubbing, and excavation to grade, a layer of filter fabric was put down followed by 6 in. of free-draining granular borrow. The CTB containing the grinding residue was laid down next. It consisted of, first, a 6-in. lift of the solids removed from the I-15 median and shoulders followed by a 6-in. lift of the sludge material from the drying beds. Portland cement was then placed over the grinding residue at a rate of 1 bag/yd<sup>2</sup>. This equates approximately to a three-bag mix per cubic yard of soil-cement.

After application of the portland cement, a piece of heavy equipment, a recycler, was run through the lifts to mix the ingredients together. Water was added as needed. Most of the time, sufficient moisture was present in the grinding waste to meet the conditions of the special provisions, even after 4 months of drying in Utah's summer sun.

A road grader then proceeded to level the mix. Mechanical compaction to 96 lb/ft<sup>3</sup> dry density followed. After completion of the CTB, the remainder of the project (prime coat, polyethylene sheeting, granular borrow, untreated base course, bituminous surface course, and plant mix seal) was routine. Asphalt was laid down November 1990. (UDOT's specifications do not ordinarily permit asphalt placement after October 15 of any year, but because of the unusual nature of this project, allowances were made.) Traffic was on the finished roadway by December 1990.

TABLE 5 COMPARATIVE pH VALUES

1.) Soil sample taken 6/25/90 from frontage road disposal project:	<u>8.9</u>
2.) Water from drill hole #1, above defined site:	<u>7.8</u>
3.) Water from drill hole #2, above defined site:	<u>8.2</u>
4.) Water from drill hole #3, above defined site:	<u>8.4</u>
5.) Water from drill hole #4, above defined site:	<u>7.6</u>
6.) Water from drill hole #5, above defined site:	<u>8.6</u>
7.) Polymer Modified Concrete (PMC) grindings taken from I-15:	<u>12.1</u>
8.) PMC grindings + median material mixed 50/50:	<u>11.3</u>
9.) Class F fly ash from the Navajo Power Plant near Page, Az.:	<u>11.5</u>
10.) Ideal type III, High Early strength portland cement:	<u>13.0</u>

TABLE 6 DIRECT COSTS OF WASTE AND WASTEWATER DISPOSAL

<b>Structural Section Modification Costs:</b>	
Portland Cement Treated Base Course:	\$70,000
6" Underdrain:	9,000
Filter Fabric	13,000
Polyethylene Sheeting	28,000
Prime Coat	2,000
Free Draining Granular Borrow	33,500
Sub-total	\$155,500
<b>Miscellaneous Costs:</b>	
Hauling	162,000
UDOT Acid Treatment	60,000
Pumping Into Trucks	205,000
Standby Time During Shutdown	215,000
Bill From WWTP	9,500
Sub-total	\$651,500
TOTAL	\$806,500

TABLE 7 STRUCTURAL SECTION COST COMPARISON

<b>Modified Section Costs:</b>	
Section Modifications	\$155,000
6" Granular Borrow (A-1-a)	18,000
12" Untreated Base Course	27,000
7.5" Bituminous Surface Course	81,000
1" Plant Mix Seal Coat	14,000
	\$295,000
<b>Typical Section Without Modifications:</b>	
27" Granular Borrow (A-2-4)	\$83,000
12" Granular Borrow (A-1-a)	36,000
9" Untreated Base Course	20,250
7.5" Bituminous Surface Course	81,000
1" Plant Mix Seal Coat	14,000
	\$234,250
Cost Difference:	\$60,750

## PROJECT COSTS

Table 6 gives the costs directly related to dealing with the waste and water disposal. Table 7 gives the additional cost generated by incorporating the solid waste into the roadway structural section.

## CONCLUSIONS

To say that PCC is a major construction material in any urban environment is an understatement. The list of items constructed from PCC is extensive. When broken up or crushed, PCC becomes an alkali waste. Consequently, it has the potential to alter the environment; in our case, the pH of adjacent waters. This was UDOT's first experience of this nature. Activities conducted on a learning curve can be expensive, as was this project. Since then, ideas on how UDOT might handle a similar problem more effectively and less expensively have emerged.

One perceived scenario is as follows: During the grinding operation ready mix trucks will be on site and receive a specific amount of slurry discharge. They would then proceed to a batch plant and add calculated amounts of sand, rock, and cement to produce concrete meeting UDOT specifications for structures such as noise wall sections, Jersey concrete barriers, and so forth. This material would then be discharged into the appropriate forms, and the truck would drive back to the grinding operation to receive additional slurry. When finished, the UDOT will have a concrete product ready for use, rather than hundreds of thousands of gallons of wastewater and thousands of yards of solid waste to contend with.

Because of the high fines content of such waste, more than the usual amounts of portland cement are required to produce an acceptable concrete mix. Test batches have indicated around 12 bags/yd<sup>3</sup>. This is expensive, but compared with treatment and disposal costs of an industrial waste, costs would be approximately one-half of those required to treat and dispose of the same quantity of waste material.

If soil and water conditions in Utah were more acid, as they are in other parts of the country, an alkali discharge

might even be desirable. But most of Utah is desert, and millennia of leaching under hot, dry conditions are not conducive to acid soil or water conditions. Inevitably UDOT will encounter this problem again. Recent experience should provide a sound base from which to proceed.

#### ACKNOWLEDGMENTS

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#### REFERENCE

1. UDOT AADT Publication. 1989.

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# Engineering Properties of Phosphogypsum-Based Slag Aggregate

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Between January 5, 1989, and November 15, 1989, Freeport-McMoran, Inc. (FMI) in cooperation with the Davy McKee Corporation operated an experimental pilot plant for the recovery of sulfur dioxide ( $\text{SO}_2$ ) and slag aggregate from the thermal decomposition of phosphogypsum and other materials. The pertinent results collected from FMI's testing program on the slag aggregate are summarized. The research program included physical and chemical characterization; mineralogical, leachate, and radiological analysis; and durability tests. The results indicate that the slag aggregate may find uses in road base/subbase construction, hot mix asphalt paving mixtures, and portland cement concrete mixtures.

Since 1982, the Davy McKee Corporation (DMC) and the Florida Institute of Phosphate Research (FIPR) have been involved in a cooperative research and development program to develop a practical process for the thermal decomposition of phosphogypsum for the production of sulfuric acid and a saleable, solid by-product (1). The process developed to date incorporates the use of a circular grate system that has been successfully applied to the iron and steel industry. The conversion process uses a mixture of phosphogypsum, a solid carbon source (currently petroleum coke), waste phosphatic clays, and other additives depending on certain process parameters. A typical feed composition is given in Table 1. The mixture is fed onto the rotating, circular grate and is processed in a series of sealed zones on the grate. The raw gas, high in  $\text{SO}_2$ , is collected for use as the feed gas for a conventional metallurgical-type sulfuric acid plant. The sintered, solid by-product, which remains on the grate after reaction, is discharged in a dry form from the grate by a tilting pan mechanism. The DMC/FIPR conversion process is shown in Figure 1 (12).

Freeport-McMoRan, Inc. (FMI), a major materials company located in south Louisiana, constructed a demonstration pilot plant at its Agrico Uncle Sam facility in Donaldsonville, in a joint project with DMC (3). Different experimental burns (Campaigns 1 through 5) were conducted between January 5, 1989, and November 15, 1989. The various campaigns were needed to achieve a better moisture control in the feed mix. Also, inconsistent nodule particle size and poor mixing in the pug mill were dominant in the initial campaigns. However, Campaign 5 was considered a success in terms of quality of aggregate produced. Slag aggregate samples were collected from the five campaigns for future testing.

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Although a definite market has not yet been identified for the slag aggregate, the following applications appear to be promising:

1. Phosphatic slimes consolidation (1),
2. Neutralization of phosphate industry waste materials (1),
3. Replacement or supplement to high-grade limestone or lime used in  $\text{SO}_2$  scrubber systems (1),
4. Roadbed construction,
5. Portland cement concrete mixtures,
6. Hot mix asphaltic concrete mixtures,
7. Asphalt shingles granules, and
8. Railroad ballast.

Technical, environmental, economic, and regulatory factors will influence the actual use of the slag aggregate in these applications.

## OBJECTIVE

The main objective of this paper is to summarize the results of physical, chemical, mineralogical, and engineering properties tests conducted on the slag aggregate. Particular attention is given to the aggregate performance in relationship to the Louisiana Department of Transportation and Development (LDOTD) specifications.

## MATERIAL

The feed mix used in the production of  $\text{SO}_2$  and the slag aggregate consists of phosphogypsum, petroleum coke, pyrite, bentonite, sand, and returns. The returns include  $\frac{1}{4}$ -in. output material from the sintering grate. Currently, there are about 100 tons of slag aggregate from Burn Campaign 5 stockpiled at the Uncle Sam plant (3). Also, smaller quantities of slag aggregate from the other campaigns were collected in 5-gal buckets and are currently stored at the FMI Research and Development Laboratory in Belle Chasse, Louisiana. Characterization of the slag aggregate from these campaigns is discussed later.

## EXPERIMENTAL RESULTS

### Chemical (Elemental) Composition

Chemical characterization of the slag aggregate was carried out as part of the research program (3). The chemical break-

TABLE 1 TYPICAL COMPOSITION OF FEED MATERIALS USED IN THE CIRCULAR GRATE SYSTEM PROCESS (1)

Raw Materials	Percentage Weight Composition (Dry Basis)
Phosphogypsum (w/ hydration water)	53.1
Petroleum Coke	5.0
Pyrite	18.8
Bentonite	1.1
Sand	5.0
Returns <sup>a</sup>	17.0

<sup>a</sup>Returns include minus 1/4 inch output material from the sintering grate.

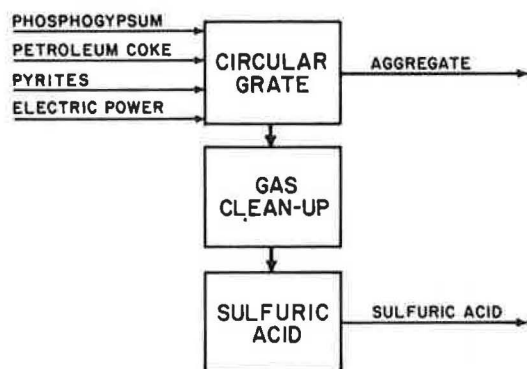


FIGURE 1 DMC/FIPR conversion process of phosphogypsum to sulfuric acid and aggregate (2).

down of the material is given in Table 2. Also, a number of trace elements such as As, Ba, Cd, Cr, Pb, Ag, Hg, and Se are present in the slag aggregate.

### Mineralogical and Morphological Analysis

Examination of the slag aggregate by X-ray diffraction and microscopy indicates the following original phases of crystallized materials in the approximate order of decreasing abundance: Magnetite ( $\text{Fe}_3\text{O}_4$ ) plus hematite ( $\text{Fe}_2\text{O}_3$ ), amorphous/glassy constituents, belite ( $\beta\text{-}2\text{CaO}\cdot\text{SiO}_2$ ), and quartz ( $\text{SiO}_2$ ). Table 3 presents a compositional analysis of selected samples from the Campaign 2 slag aggregate. Gamma  $\text{C}_2\text{S}$  was not detected by X-ray diffraction (4). Microscopy studies of a polished section of reference sample indicated magnetite, dicalcium ferrite (angular crystals in matrix), and belite (round to dendritic crystals with multidirectional lamellae). Initial comments by Campbell and Tang (5) indicated that "the application of the slag aggregate in road base and embankments does not appear to have problems as suggested by its phase composition and volume stability."

### Physical Analysis

Physical tests performed on the slag aggregate include particle size distribution, unit weight, specific gravity, percent absorption, ignition loss, Los Angeles abrasion, accelerated polish, aggregate freeze-thaw, and others.

TABLE 2 RESULTS OF THE ELEMENTAL ANALYSIS OF THE SLAG AGGREGATE (3)

Constituent	Content (%)
$\text{Na}_2\text{O}$	0.16
$\text{CaO}$	39.3
$\text{SiO}_2$	22.0
$\text{Al}_2\text{O}_3$	0.86
$\text{TiO}_2$	0.10
$\text{MgO}$	0.14
$\text{C}_{\text{tot}}$	0.21
$\text{C}_{\text{org}}$	0.07
F	0.40
Pb	0.03
As	0.094
Zn	0.74
$\text{Fe}_2\text{O}_3$	29.2
Total S	3.04
$\text{SO}_4$	1.10
L.O.I. (weight gain)	-3.06

TABLE 3 MINERALOGICAL COMPOSITION ANALYSIS OF CAMPAIGN 2 SLAG AGGREGATE (4)

Crystal Type	Composition (%)
Magnetite ( $\text{Fe}_3\text{O}_4$ ) + Hematite ( $\text{Fe}_2\text{O}_3$ )	35
Amorphous/Glassy Constituents	30
Belite ( $\beta\text{-}2\text{CaO}\cdot\text{SiO}_2$ )	25
Quartz ( $\text{SiO}_2$ )	8
Calcium Ferrite ( $\text{Ca}_2\text{Fe}_2\text{O}_5$ )	minor
Pyrrhotite ( $\text{Fe}_7\text{S}_8$ )	minor
Alkali Sulfate ( $\text{Ca, K, Na-SO}_4$ )	minor

### Specific Gravity, Unit Weight, and Percent Absorption

The slag aggregate can be described as a nonplastic material. The aggregate particles have an irregular shape. Their surface texture can be generally described as rough and honeycombed (visible pores and cavities). Table 4 presents typical specific gravity, unit weight, and percent absorption data on the material from Campaigns 2, 3, and 5 (Houston, unpublished data). The specific gravity and percent absorption values reported in this table were determined in accordance with ASTM C127. The unit weight values were determined using ASTM C29. Currently, there are no specification limits set on specific gravity or percent absorption by LDOTD. Material specifications place limits on unit weight only. ASTM and AASHTO specify minimum dry rodded unit weights from 60 to 70 lb/ft<sup>3</sup> depending on the type of application. The data of Table 4 indicate compliance with these requirements.

### Gradation Analysis

Particle size distribution curves for the slag aggregate produced in Campaigns 2 through 5 are shown in Figure 2 (Houston, unpublished data). The gradation analysis was performed in accordance with ASTM C136 or LDOTD TR113. The

TABLE 4 SPECIFIC GRAVITY, UNIT WEIGHT, AND PERCENT ABSORPTION TESTS DATA FOR SELECTED SAMPLES FROM CAMPAIGNS 2, 3, AND 5 (Houston, unpublished data)

Campaign No.	K <sup>a</sup>	Sample	Unit <sup>b</sup> Weight (DL lb/ft <sup>3</sup> )	Unit <sup>c</sup> Weight (DR lb/ft <sup>3</sup> )	SG <sup>d</sup> (SSD)	SG (dry bulk)	Absorption <sup>e</sup> (%)
2	0.77	Bucket (ASTM #56)	67.1	74.2	3.07	2.99	2.49
2	0.77	1/4" x 3/8"	N/A <sup>f</sup>	N/A	2.76	2.61	5.97
3	0.77	1/4" x 3/8"	N/A	N/A	2.83	2.71	4.33
3	0.77	Bucket (ASTM #56)	69.7	73.7	N/A	N/A	N/A
5	0.77	N/A	N/A	N/A	3.05	2.97	N/A

<sup>a</sup>K = (CaO)/(Fe<sub>2</sub>O<sub>3</sub>+SiO<sub>2</sub>)

<sup>b</sup>DL = dry loose

<sup>c</sup>DR = dry rodded

<sup>d</sup>SSD = saturated surface dry

<sup>e</sup>Absorption = % of dry weight

<sup>f</sup>N/A = not available

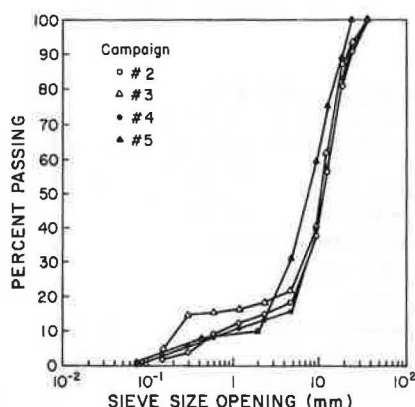


FIGURE 2 Gradation curves for slag aggregate samples selected from Campaigns 2, 3, 4, and 5 (Houston, unpublished data).

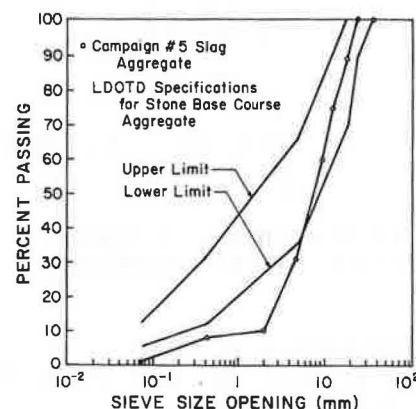


FIGURE 3 Comparison of the Campaign 5 slag aggregate gradation with LDOTD requirements for base course material.

LDOTD gradation requirements for stone used in base course applications are plotted in Figure 3. They indicate that the Campaign 5 slag aggregate will not conform to the gradation requirements for stone unless sufficient fines (< #4 sieve) are added to the slag aggregate. This can be accomplished by adding sand, silt, fly ash, or phosphogypsum size fractions. Gradation can also be improved by crushing some of the slag aggregate itself. The addition of a fine fraction will also be needed when preparing portland cement concrete or hot mix asphaltic concrete mixtures.

#### Ignition Loss Test

The ignition loss test is generally used as a means of assessing some measure of burn efficiency. The LDOTD TR114 test method covers the procedure for testing coarse aggregate for resistance to loss of weight by ignition. Table 5 presents the ignition loss test results for selected samples from Campaigns 2 and 3 (Houston, unpublished data). All the data indicate

weight gains upon firing the slag aggregate samples. The weight gains may be due to thermally induced reactions with oxidation being a prime mechanism (Houston, unpublished data). Hydration and carbonation reactions are also possible sources of the weight gain.

#### Durability Studies

The durability tests performed on the slag aggregate encompassed such tests as Los Angeles (LA) abrasion, aggregate freeze-thaw, expansion, hot water immersion, soundness, and accelerated polish.

#### LA Abrasion Test

LDOTD TR111 and AASHTO T96 test methods cover the procedure for testing different sizes of coarse aggregate for resistance to abrasion using the Los Angeles testing machine.

TABLE 5 IGNITION LOSS TEST RESULTS FOR SELECTED SAMPLES FROM CAMPAIGNS 2 AND 3 (Houston, unpublished data)

Campaign No.	K <sup>a</sup>	Sample	Ignition Weight Change (%) <sup>b</sup>
2	0.77	1/4" x 3/8"	+3.2
2	0.77	1/4" x 3/8"	+4.1
2	0.77	+5/8"	+3.9
2	0.77	+5/8"	+5.6
3	0.77	3/8" x 5/8"	+3.9
3	0.77	1/4" x 3/8"	+5.4
3	0.77	+#4	+5.0

<sup>a</sup>K = (CaO)/(Fe<sub>2</sub>O<sub>3</sub>+SiO<sub>2</sub>)

<sup>b</sup>Heating Conditions: 900°C for 30 minutes in a muffle furnace.

Data summarizing the results of LA abrasion testing of selected aggregate samples from Campaigns 1 through 5 are given in Table 6 (Houston, unpublished data). The LDOTD specifications require an LA abrasion loss not to exceed 40 percent (6). The differences in the abrasion loss data obtained for samples for Campaigns 1 through 5 and for Gradings A and B were relatively insignificant. However, the results were high compared with the 40.0 percent specification limit. Grading C test data exceed the 40 percent limit in all burn campaigns except Campaign 5. Such data indicate the presence of poorly burned nodules in the Grading C test fraction, which is composed of #4 × 3/8-in.-sized material (Houston, unpublished data).

For Campaign 3, with an increase in the K value, there seems to be an increase in the abrasion loss values for A grading tests. However, such a conclusion is not validated for Grades B and C.

Selected slag aggregate samples from Campaigns 2 and 5 were also subjected to LA abrasion testing after 4 weeks of immersion in 5-gal buckets containing approximately 12,500

cc of 150°F tap water. Table 7 gives the LA abrasion losses before and after the hot water treatment of the slag aggregate (Houston, unpublished data). The results indicate that the 4-week hot water immersion test had no significant adverse effect on the abrasion resistance of these samples. The slight improvement indicated for the soaked samples is considered insignificant (Houston, unpublished data).

#### Five-Cycle Sodium Sulfate Soundness Test

The sulfate soundness test (AASHTO T104), using the sodium sulfate procedure, was conducted on slag aggregate samples from Campaigns 2 and 3 (Houston, unpublished data). Table 8 presents a summary of the soundness test results. As noted in the table, the LDOTD specifications specify a maximum degradation loss of 15 percent. It is therefore concluded that the slag aggregate samples will meet the requirements of typical sulfate soundness tests for use as a construction aggregate.

#### Aggregate Freeze-Thaw Test

The aggregate freeze-thaw test is not currently required by LDOTD. The test was conducted to provide another measure of long-term durability. Failure criteria for rating aggregate resistance to disintegration by freezing and thawing are generally related to the failure criteria established for judging the sulfate soundness for aggregates (AASHTO T104). LDOTD sets a 15 percent loss limit for all aggregates when tested by the five-cycle magnesium sulfate soundness test (6). Indiana uses a 50-cycle aggregate freeze-thaw test. Its specification limits on degradation losses are 12 to 16 percent for the top four classes of construction aggregates and 20 to 25 percent for the two lowest classes of aggregate allowed for use in the state.

Both 25- and 50-cycle aggregate freeze-thaw tests were performed on selected slag aggregate samples from Campaign 2. The results are presented in Table 9 (Houston, unpublished data). The data indicate that the smaller aggregate fractions (#4 × 3/8 in. and 3/8 × 3/4 in.) were much more resistant to freeze-thaw cycles than was the 3/4 × 1 1/2 in. sized fraction. The latter failed to meet the Indiana specification limits or the LDOTD magnesium sulfate soundness test requirements. Furthermore, with an increase in the number of freeze-thaw cycles, there is an increase in the percentage of degradation loss.

During the freeze-thaw testing of the slag aggregate, a white precipitate formed in the water during the thaw cycles. A sample of the white precipitate was collected and analyzed by X-ray diffraction (Houston, unpublished data). The analysis indicated that the precipitate was more than 90 percent calcium carbonate (CaCO<sub>3</sub>). It is probable that readily soluble calcium compounds within the slag are leached to the surface when the aggregate is exposed to moisture during the thaw cycles (Houston, unpublished data). The development of such reaction product upon wetting is undesirable since it may prevent contact or adhesion between the slag aggregate and liquid asphalt or cement paste, respectively, in hot mix asphaltic concrete mixtures or portland cement concrete. Fur-

TABLE 6 AVERAGE LA ABRASION TEST RESULTS FOR SELECTED SAMPLES FROM CAMPAIGNS 1 THROUGH 5 (Houston, unpublished data)

Campaign No.	K <sup>a</sup>	Abrasion Loss (%)			LDOTD Specification Limit (%)
		Grade A	Grade B	Grade C	
1	0.66	37.7			40
2	0.77	38.1			
2	0.77		36.2		40
2	0.77			53.8	
3	0.77	30.8			
3	0.82	37.4			40
3	0.86	39.8			
3	0.77		36.2		
3	0.82		34.2		
3	0.86		35.1		
3	0.77			45.5	
3	0.82			41.0	
3	0.86			44.6	
4	0.77	39.9	38.8	48.5	40
5	0.77	36.4	34.0	38.6	40

<sup>a</sup>K = (CaO)/(Fe<sub>2</sub>O<sub>3</sub>+SiO<sub>2</sub>)

TABLE 7 EFFECT OF HOT WATER IMMERSION ON LA ABRASION FOR SELECTED SAMPLES FROM CAMPAIGNS 2 AND 5 (Houston, unpublished data)

Campaign No.	K <sup>a</sup>	Abrasion Loss (%)						LDOTD Specification Limit (%)
		Pretreatment Sample			Hot Water Immersion Sample			
		Grade A	Grade B	Grade C	Grade A	Grade B	Grade C	
2	0.77	37.3	--	49.6	36.7	--	46.6	40
5	0.77	36.4	--	38.6	33.5	--	40.7	40

$$^aK = (\text{CaO})/(\text{Fe}_2\text{O}_3 + \text{SiO}_2)$$

TABLE 8 FIVE-CYCLE SODIUM SULFATE SOUNDNESS TEST RESULTS FOR SELECTED AGGREGATE SAMPLES FROM CAMPAIGNS 2 AND 3 (Houston, unpublished data)

Campaign No.	K <sup>a</sup>	Particle Size	% Loss	LDOTD <sup>b</sup> Specification Limit (%)
2	0.77	#4 x 3/8"	0.87	15
2	0.77	3/8" x 3/4"	0.16	15
2	0.77	3/4" x 1-1/2"	1.01	15
3	0.77	#4 x 3/8"	0.16	15
3	0.77	3/8" x 3/4"	0.29	15
3	0.77	3/4" x 1-1/2"	0.67	15

$$^aK = (\text{CaO})/(\text{Fe}_2\text{O}_3 + \text{SiO}_2)$$

<sup>b</sup>The 15% loss limit is set by the LDOTD for all aggregates when tested by the 5 cycle magnesium sulfate soundness test.

ther testing and evaluation are necessary to investigate this phenomenon.

#### *Pennsylvania DOT Test for Expansion of Slag Aggregate*

Since slag-type aggregates generally exhibit some expansion due to hydration of free oxides, the Pennsylvania DOT test for expansion was used to evaluate slag samples from Campaigns 2, 3, and 5. Slag aggregate samples were presized and then compacted in accordance with Pennsylvania DOT specifications to determine optimum moisture and maximum dry density values. Compacted specimens were then submerged in a 160°F water bath to accelerate the potential expansion reactions. Following a 7-day immersion period, the specimens were removed from the water but kept saturated for an additional 7 days curing in air at 160°F.

The results of the expansion tests are given in Table 10. The measured expansion is reported as a percentage of the original length of the compacted specimen (Houston, unpublished data). The 14-day expansion values varied from a low of 0.04 percent to a high of 0.11 percent. These values are within the Pennsylvania DOT specification limit of 0.5 percent.

#### *Accelerated Polish Test (Skid Resistance)*

The accelerated polish test was conducted on slag aggregate samples from Campaign 5. The aggregate tested should pass the 1/2-in. sieve and be retained on the No. 4 sieve. LDOTD TR116 covers the procedure for determining a relative mea-

sure of the extent to which aggregate in the wearing surface of the roadway will polish under traffic. The accelerated polish test is a relative one, and it is an indicator of aggregate performance rather than a required criterion. LDOTD has four classes for skid resistance rating. The scheme is given in Table 11 (6). A skid resistance rating of I corresponds to the best aggregate type for use in a wearing course asphaltic mixture. Generally, good quality sandstone will have a skid resistance rating of I. A material with a skid resistance rating of IV will still be used in designing wearing course mixtures. However, the quantity of coarse aggregate used will be considerably less than a material with a skid resistance rating of I. The initial and final polish values for the slag aggregate were 37 and 32, respectively, as determined by LDOTD laboratory personnel (LDOTD, unpublished data). Therefore, a skid resistance rating of III was assigned to the slag aggregate in accordance with Table 11.

#### **Environmental Analysis**

The environmental testing program consisted of leachate and radiological testing.

#### *Leachate Testing*

The leachate analysis of the trace elements present in the slag aggregate from Campaign 5 was conducted using the EP Toxicity and the TCLP tests. The results are given in Table 12 (3). The results indicate that the leachate quality is well within the EPA leachate standards.

#### *Radiological Testing*

Radiation assays were also performed on slag aggregate samples obtained from Campaign 3 (Scott, unpublished data). This material should not be significantly different from the products from other campaigns in its radiological characteristics. Table 13 gives the results of the radiation assay measurements. As noted, the radium-226 content increased in the slag aggregate compared with the raw phosphogypsum. This is a significant finding and certainly represents a cause of concern, particularly in light of the existing Environmental Protection Agency ban on the use of raw phosphogypsum. This ban, which encompasses research studies as well as practical uses, is currently under reconsideration by EPA. Previously, exemptions have been made to allow the phospho-



TABLE 9 AGGREGATE FREEZE-THAW TEST DATA FOR CAMPAIGN 2 (Houston, unpublished data)

Aggregate Size	K <sup>a</sup>	Freeze-Thaw Degradation %			Average (%)	State of Indiana Specification Limits (%)	LDOTD Specification Limit (%)
		Test Cycle	Test #1	Test #2			
#4 x 3/8"	0.77	25	1.3	1.7	1.5	12-25	15
		50	3.8	5.2	4.5		
3/8" x 3/4"	0.77	25	2.1	5.0	3.5	12-25	15
		50	5.0	5.7	5.4		
3/4" x 1-1/2"	0.77	25	20.3	28.8	24.6	12-25	15
		50	55.5	42.4	29.0		

<sup>a</sup>K = (CaO)/(Fe<sub>2</sub>O<sub>3</sub>+SiO<sub>2</sub>)<sup>b</sup>The 15% loss limit is set by the LDOTD for all aggregates when tested by the 5 cycle magnesium sulfate soundness test.

TABLE 10 PENNSYLVANIA DOT TEST DATA FOR EXPANSION OF SELECTED SAMPLES FROM CAMPAIGNS 2 AND 3 (Houston, unpublished data)

Campaign No.	Test No.	K <sup>c</sup>	Optimum Moisture (%)	Maximum Dry Density (lb/ft <sup>3</sup> )	Molded Moisture (%)	Molded Dry Density (lb/ft <sup>3</sup> )	Expansion after 14 Days (%)	Penn. DOT, Specification Limits (%)
2 <sup>a</sup>	1	0.77	12.2	118.6	11.5	118.2	0.11	0.5
	2		--	--	11.5	118.1	0.04	
3 <sup>a</sup>	1	0.77	11.8	118.2	12.3	117.8	0.04	0.5
	2		--	--	11.4	118.0	0.04	
5 <sup>b</sup>	1	0.77	8.0	135.1	8.5	126.4	0.09	0.5
	2				8.0	130.4	0.07	

<sup>a</sup>Compaction: ASTM D698 (standard Proctor)<sup>b</sup>Compaction: ASTM D1557 (modified Proctor)<sup>c</sup>K = (CaO)/(Fe<sub>2</sub>O<sub>3</sub>+SiO<sub>2</sub>)

TABLE 11 LDOTD SKID RESISTANCE RATING FOR COARSE AGGREGATES (6)

Polish Value	Skid Resistance Rating
> 37	I
35 to 37	II
30 to 34	III
< 30	IV

TABLE 13 RADIATION ASSAY MEASUREMENTS ON SLAG AGGREGATE SAMPLES FROM CAMPAIGN 3 AND RAW PHOSPHOGYPSUM (Scott, unpublished data)

Material	Radium-226 (pCi/g)	Radon 222 Emanation Power (%)	Gamma (uRem/h)
Slag Aggregate	51	0.25	20
Raw Phosphogypsum	29 <sup>a</sup>	10	30 <sup>b</sup>

<sup>a</sup>Louisiana phosphogypsum (sample was tested in the laboratory).<sup>b</sup>Florida phosphogypsum (measurement was done over an actual stack).

TABLE 12 RESULTS OF THE LEACHATE ANALYSIS ON THE SLAG AGGREGATE (3)

Element	EP Toxicity <sup>a</sup> (mg/l)	TCLP <sup>b</sup> (mg/l)	EPA Leachate Standards (mg/l)
As	1.6	0.882	5.0
Ba	0.52	0.17	100.0
Cd	< 0.01	< 0.01	1.0
Cr	< 0.01	< 0.05	5.0
Pb	< 0.05	< 0.1	5.0
Ag	< 0.01	< 0.05	5.0
Hg	< 0.001	< 0.005	0.2
Se	< 0.01	< 0.1	1.0

<sup>a</sup>Method 40CFR261<sup>b</sup>Methods 40CFR268 and 6010/7470

gypsum to be used for agricultural purposes. Future studies must be directed at ascertaining the health-related risks associated with the use of slag aggregate for practical applications such as an unbound base material or as an aggregate in asphaltic concrete or portland cement concrete. The studies must consider both the short time exposure to construction workers and the long-term exposure to users of such facilities. On a more positive note, the radon-222 emanation power in the slag aggregate is about 0.25 percent compared with 10 percent in the raw phosphogypsum. Thus, the potentially negative consequences resulting from radon-222 emanation have been reduced significantly.

## CONCLUSIONS

The research studies on the slag aggregate included physical and chemical characterization tests, radiological and leachate analysis, and durability studies. Important findings of the testing program relevant to the slag aggregate properties and performance include the following:

1. The raw mix design, the blending and burning processes, and the methods of handling the grate product will significantly influence the quality of the slag aggregate produced. Therefore, quality control is a necessity. Otherwise, failures in applications may result because of material variability.

2. The results of the EP toxicity and the TCLP leachate tests indicate that the slag aggregate will meet the EPA leachate standards.

3. There is an increase in Ra-226 content in the slag aggregate compared with raw phosphogypsum. However, radon emanation is significantly decreased.

4. There are weight gains rather than weight losses upon firing the slag aggregate during the ignition loss test. Oxidation, hydration, and carbonation reactions are possible sources of the weight gain.

5. Generally, the LA abrasion losses (Grades A and B) for the slag aggregate will meet the LDOTD specification limit of 40 percent. However, these values are relatively close to the 40 percent limit.

6. Hot water immersion had no adverse effect on LA abrasion values for the slag aggregate.

7. The larger slag aggregate fraction ( $\frac{3}{4} \times 1\frac{1}{2}$  in.) will not meet the aggregate freeze-thaw specification requirements of Indiana. The smaller slag aggregate fractions ( $\#4 \times \frac{3}{8}$  in. and  $\frac{3}{8} \times \frac{3}{4}$  in.) were much more resistant to freeze-thaw cycles and will meet the specification requirements.

8. The expansion test results indicate that the slag aggregate will meet the Pennsylvania DOT maximum specification limit of 0.5 percent for expansion. No such test is required by LDOTD.

9. The slag aggregate will meet the 15 percent loss limit set by LDOTD for aggregates tested by the five-cycle sodium sulfate soundness test.

10. A skid resistance of III, based on LDOTD criteria, was assigned to the slag aggregate on the basis of initial and final polish values.

## RECOMMENDATIONS FOR FURTHER STUDIES

The data obtained by the FMI studies provide an excellent starting point for research related to the use of slag aggregate

in roadbed applications, in portland cement concrete mixtures, or in hot mix asphalt paving mixtures. A follow-up comprehensive testing program is currently under development at the Institute for Recyclable Materials at Louisiana State University. This program will explore the potential use of slag aggregate in the applications noted.

The scope of work will include both environmental and mechanical testing of the slag aggregate and of the mixtures developed for the unbound base, asphaltic concrete, and portland cement concrete applications. Environmental analysis will include toxicity characteristic leaching procedure, EP toxicity, and radiation assay measurements. The mechanical studies will include dynamic modulus testing, flexural strength relations, fatigue analysis, and creep testing.

## ACKNOWLEDGMENT

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# Corrosivity of Indiana Bottom Ash

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Current studies of the engineering properties of bottom ash have justified its use in many kinds of highway construction, such as backfill, embankment, subbase, and even base courses. However, the electrochemical characteristics of bottom ash are still not well known, and neither is its corrosivity to the metal structures that are commonly included in highway construction. A simple corrosivity test and corrosivity evaluation results are presented for Indiana bottom ashes to clarify the extent of this problem. Four corrosivity parameters were used to estimate the corrosivity of bottom ash: minimum electrical resistivity ( $r$ ), pH, soluble chloride content ( $\text{Cl}^-$ ), and soluble sulfate content ( $\text{SO}_4^{2-}$ ). The corresponding noncorrosive limits are proposed to be a minimum resistivity of 1,500 ohm-cm, a minimum pH of 5.5, a maximum soluble chloride of 200 ppm, and a maximum soluble sulfate of 1,000 ppm. Eleven bottom ashes collected from 10 power plants in Indiana were tested. The  $r$  varied from 200 to 7,000 ohm-cm, the pH from 3 to 10, the  $\text{Cl}^-$  from 0.4 to 16 ppm, and the  $\text{SO}_4^{2-}$  from 50 to 1,100 ppm. At least 7 of the 11 bottom ashes tested were therefore classified as corrosive. Most Indiana bottom ashes are potentially corrosive. If the same holds true for bottom ashes produced in other states, the following actions are recommended before use in the vicinity of the metal structure in highway construction: (a) thoroughly examine the corrosion potential of the bottom ashes proposed for use; (b) pretreat the potentially corrosive bottom ashes, either at the power plant source or on the site of use; and (c) use corrosion-resistant metal members or plastic substitutes.

The corrosivity of bottom ash may limit its extensive use in highway construction. Simple corrosivity tests and corrosivity evaluation results of bottom ash samples collected in Indiana are presented.

Every year a huge quantity of solid waste is produced by industry. The disposal of this waste is costly and may cause environmental problems. If this material were to be reused, not only could a solid waste disposal problem be solved but an alternative material could be provided for the construction market.

Recently, bottom ash has become one of the solid wastes most actively considered for reuse. Bottom ash is one kind of coal ash, a slag material that builds up on the heat-absorbing surface of a coal furnace and subsequently falls through the furnace bottom to the ash hopper below (see Figure 1). Either wet or dry bottom ash may be produced (1). Wet bottom ash particles are generally black, hard, and look much like crushed glass, since they derive from the quenching of the molten ash from the furnace into water. In contrast, during its sedimentation through the furnace bottom, dry bottom ash directly solidifies and agglomerates into coarse particles, which are gray and more porous ("popcorn"

like). Recent studies on the engineering properties of bottom ash (1,2) have recommended the use of bottom ash in many kinds of highway construction, such as backfill, embankment, subbase, and even base courses.

In highway construction, metal structures such as steel culverts, rebars in concrete, steel piles as retaining walls, and reinforcing steel strips in reinforced earth are often involved. Since the electrochemical characteristics of bottom ash are not well defined, potential interactions between the bottom ash and such metal structures are of concern. Accordingly, the corrosivity of bottom ash needs to be examined to clarify the extent of this problem. That was the objective of the study reported herein.

Corrosivity tests selected for routine highway use must be simple and inexpensive. In this study, a set of electrochemical characteristics was chosen to estimate the corrosivity of bottom ash. They are minimum resistivity ( $r$ ), pH, soluble chloride content ( $\text{Cl}^-$ ), and soluble sulfate content ( $\text{SO}_4^{2-}$ ). Accordingly, the testing program in this study was limited to the determination of these four characteristics, which mostly followed the test methods used by the California Department of Transportation. Thereafter, the corrosivity of bottom ash samples was evaluated by comparing the testing results with the proposed evaluation criteria. The reliability of the testing methods used in this study and the validity of the proposed evaluation criteria need to be justified by future field burial tests.

## LITERATURE REVIEW

Studies on the corrosion tendency of metals in a bottom ash environment are limited. Most investigators [Headon and Chan (3), Jablonski and Aliff (4), and Reinforced Earth Company (5)] have transferred knowledge of metal corrosion in soils to the case of bottom ashes. This section briefly covers definition, significance, mechanism, and test methods for corrosivity. Most of the contents of this section are extracted from Ke (6).

## Definition

Corrosion is the deterioration of a metal, usually caused by chemical or electrochemical interaction with the surrounding environment or media. Corrosivity is the characteristics of a material (or an environment) that indicate the likelihood of its causing the corrosion of a contacted metal. In a metal/medium system where corrosion is severe, either the metal is denoted as easily corroded or the medium is denoted as very corrosive.

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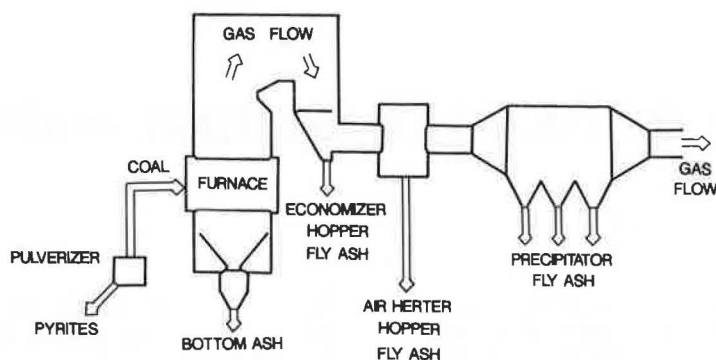


FIGURE 1 Ash collection system.

### Significance

Corrosion may result in mechanical failure of metal members or leakage of metal pipes. This leads to high maintenance costs or required replacement of corroded metal members. The annual direct losses due to corrosion in the United States were reported to be \$70 billion (7). The combination of direct and indirect losses is forcing engineers and scientists to further study and understand complicated corrosion mechanisms and to find effective solutions. In highway construction, attention is often directed to the selection of noncorrosive engineering materials to be used in the vicinity of metal structures.

### Underground Corrosion

Corrosion environments include atmospheric, aqueous, and underground environments. Because of the proposed use of bottom ash (like soils), only underground corrosion is of interest. Understanding of the underground corrosion mechanism has been developed through experience with soils. Escalante (8) characterized four types of underground corrosion: corrosion by stray current, bacteria corrosion, corrosion in undisturbed soils, and corrosion in disturbed soils. Corrosion in disturbed soils is the most complex, relating to a variety of soil characteristics and site conditions. Ke (6) presented the possible correlations between them, as given in Table 1. Furthermore, he considered the nature of bottom ash and the site conditions after construction and concluded that the parameters best related to the corrosivity of bottom ash are as follows: minimum resistivity, pH, soluble chloride content, and soluble sulfate (in order of decreasing significance), along with site variations in water content, alien inclusions, and so forth. A material with lower values of minimum resistivity and pH and higher values of soluble chloride and soluble sulfate is, generally, more corrosive.

### Test Methods

Three methods have been used to investigate the corrosion resistance of a metal or the corrosivity of a surrounding medium. They consist of burial methods (a direct method by measuring metal losses), electrochemical techniques (mainly

measuring the corrosion potential or current followed by converting it to metal loss), and correlation between medium parameters and corrosivity (multivariate regression analyses). Burial methods are the most reliable but are costly (time-consuming) and destructive, whereas electrochemical techniques are reliable and nondestructive. The cost and reliability of the correlation methods depend on the number of parameters to be used.

In this study, because of the limitation of funding and time, the correlation method with the aforementioned four electrochemical characteristics was selected to estimate the corrosivity of bottom ash. On the basis of several available evaluation criteria for soil corrosivity (with respect to these four parameters) (9–13), the approximate noncorrosive limits for bottom ash to general metal structures are proposed as follows [see Ke (6) for details]:  $r > 1,500$  ohm-cm,  $\text{pH} > 5.5$ ,  $\text{Cl}^- < 200$  ppm, and  $\text{SO}_4^{2-} < 1,000$  ppm.

The more sophisticated criteria of this kind should also depend on metal type, functional use, structure size, and so forth. Ke (6) also collected information from previous studies and provided the approximate service life of several metal members corresponding to the preceding criteria and that for other values of the four parameters.

### EXPERIMENTAL PROGRAM

In the sources from which the preceding criteria were extracted (9–13), California test methods were specified in determining the four parameters. Therefore, the testing program (laboratory) of this study basically followed these methods to readily apply the criteria to testing results. Field tests of pH and resistivity were also performed on the ash deposits of several power plants. Data are available in Ke (6).

### Selection of Samples

A total of 11 bottom ashes from 10 power plants in Indiana were sampled for testing. Consideration was given to furnace type, coal source, geographic distribution, and ash storage method. Figure 2 shows the approximate locations of the power plants. All ash samples except the Schahfer 14 ash were dry bottom ashes.

TABLE 1 CORRELATIONS BETWEEN UNDERGROUND CORROSION TYPES AND ELEMENTS OF CORROSION IN DISTURBED SOILS (6)

+: positively-related -: negatively-related	Disturbed Soils (DS)													
	SC	SRB	US	R	W	RE	pH	SS	Cl	SO <sub>3</sub>	SO	Fe	AE	ID
Stray Current (SC)	+			-	+			+						+
Sulfate-Reducing Bacteria (SRB)		+	+			-				+	+	+	-	-
Undisturbed Soils (US)			+	+									-	-
Resistivity (R)	-			+	-			-	-	+	+	+	-	-
Moisture (W)	+			+	+		+	+	+	+	+	+	-	-
Redox Pot. (RE)		-				+							+	
pH						+	+							±
Soluble Salts (SS)	+			-	+			+					+	+
Cl <sup>-</sup> (Cl)				-	+			+					+	+
SO <sub>3</sub> <sup>2-</sup> (SO)		+		+					+				+	+
Fe <sub>2</sub> <sup>+</sup> (Fe)		+		+						+			+	+
Aeration (AE)		-	-	-	+							+	+	+
Internal Drainage (ID)								+	+	+	+	+	+	+
Temperature (T)	+			-	-			+	+	+	+	+	+	+
Change of Water Table (WT)			-										+	+
Water Movement (WM)				+				-	-	-	-	-		+
Inhomogeneity (IN)				±									+	+
Alien Inclusion (AI)				±				±	±	±	±	±	±	+



FIGURE 2 Approximate locations of 10 power plants in Indiana.

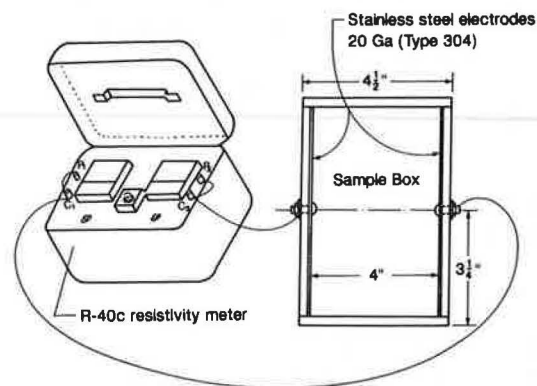


FIGURE 3 Connection of R-40C meter with sample box.

### Determination of Minimum Resistivity

The configuration of the sample box (in a plan view) and its connection with a SOILTEST R-40C resistivity meter is shown in Figure 3. The determination of minimum resistivity was mainly conducted as specified in California Test 532 (9), except for one testing procedure modified by the authors. Since this test method specifies the maximum size of particles used for testing to be the opening of the No. 8 sieve (related to the dimension of the sample box), it does not take into account the contribution of the coarser particles to the measured resistivity. A crushing method was hence used to crush the ash

sample into particles smaller than the aforementioned maximum size, followed by the other operational procedures described in the method. "Crushing" would increase the total surface area of ash particles that water can access and the intensity of ion diffusion from ash to water, probably leading to a lower resistivity value (a more critical point). Indeed, lower resistivity values were observed in some dry bottom ash samples after crushing than in the same samples with larger particles sieved out (6). The minimum resistivity is usually obtained when the ash sample is fully saturated.

### Determination of pH Values

The laboratory determination of pH was also made in accordance with California Test 532 (9). First, 2 teaspoons of bottom ash were mixed with 2 teaspoons of deionized water in a cup (with an ash:water ratio near unity), and the pH values were



then measured using a Corning meter 125, with a reading precision of two decimal places. The suggested crushing procedure was also performed on the samples before measuring to obtain a pH value more representative of all the ash particles. The effects of particle size, ash:water ratio, and stirring time on pH value can be found in Ke (6).

#### Determination of Soluble Chloride Content

The determination of the soluble chloride content present in the ash sample was basically accomplished following the California Test 422 (14). However, instead of a potassium chromate indicator, an Orion combination chloride electrode (Cat. No. 961700) was used to measure the endpoint of titration of soluble chloride (in the ash-water suspension) and silver nitrate (added), from which the soluble chloride content was calculated. The effect of stirring/suspending time on  $\text{Cl}^-$  can be found in Ke (6).

#### Determination of Soluble Sulfate Content

The determination of soluble sulfate content was performed in accordance with the California Test 417 (15), except that a filtration method was used to determine the amount of  $\text{BaSO}_4$  instead of a turbidimeter (not available in the laboratory). The effect of stirring/suspending time on  $\text{SO}_4^{2-}$  can be found in Ke (6).

### TEST RESULTS AND CORROSIVITY EVALUATION

#### Test Results

Table 2 summarizes the test results of the 11 ashes examined. Because of the time constraint, determinations of soluble chloride and sulfate contents were only conducted on the first four bottom ashes in the table. The  $r$  varied from 200 to 7,000 ohm-cm, the pH from 3 to 10, the  $\text{Cl}^-$  from 0.4 to 16 ppm ( $\mu\text{g}$  of  $\text{Cl}^-$  per gram of dried ash), and the  $\text{SO}_4^{2-}$  from 50 to 1,110 ppm ( $\mu\text{g}$  of  $\text{SO}_4^{2-}$  per gram of dried ash).

#### Corrosivity Evaluation

The results of corrosivity evaluations of Indiana bottom ashes are summarized in Table 3. A bottom ash is classified as noncorrosive if all four parameters meet the corresponding noncorrosive limits; otherwise, it is corrosive. For the seven ash samples lacking data on  $\text{Cl}^-$  and  $\text{SO}_4^{2-}$ , meeting the limits of both  $r$  and pH led to a tentative classification of noncorrosive. (This is not conservative.) Therefore, at least 7 of the 11 bottom ashes (about 64 percent) were classified as corrosive. The comparison of the results of the Schahfer 14 ash and those of the others in Table 3 indicates that wet bottom ash seems to be much less corrosive than dry bottom ash. Moreover, comparing the data in Table 3 with those of natural soils (16) and those of backfills used by the Reinforced Earth

Company (17), Indiana bottom ashes appeared to be more corrosive.

### CONCLUSIONS AND RECOMMENDATIONS

Most Indiana bottom ashes are potentially corrosive, on the basis of the four ash characteristics and the proposed evaluation criteria. The reliability of the test methods used and the validity of the proposed evaluation criteria need to be verified by future field tests. If the same conclusion holds true for bottom ashes produced in other states, note of this should be taken by all state highway agencies before selecting bottom ash as an alternative highway construction material in the vicinity of metal structures.

To avoid possible corrosion failure of the adjacent metal structures in highway construction, the following actions are recommended:

1. Thoroughly examine the corrosivity of the bottom ashes proposed for use;
2. Pretreat the potentially corrosive bottom ashes before their use, either at the power plant or on the site of use; and
3. Use corrosion-resistant steel structures or plastic substitutes.

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TABLE 2 CORROSIVITY PARAMETERS OF INDIANA BOTTOM ASHES

Ash Name	$r$ (ohm-cm)	pH	$\text{Cl}^-$ (ppm)	$\text{SO}_4^{2-}$ (ppm)
Perry K.	980	4.8	15.5	598
Gibson	2201	7.6	7.3	1127
Schahfer 14 <sup>a</sup>	>6663	9.6	0.4	50
Schahfer 17	3082	8.6	6.1	383
Gallegher	335	9.1	— <sup>b</sup>	—
Mitchell	1771	8.0	—	—
Wabash	1051	5.7	—	—
Richmond	247	8.2	—	—
Stout	4249	6.6	—	—
Culley	486	8.5	—	—
Brown	213	3.2	—	—

<sup>a</sup> wet bottom ash

<sup>b</sup> not determined

TABLE 3 CORROSIVITY EVALUATION OF INDIANA BOTTOM ASHES

Ash Name	Acceptance				Overall Corrosivity
	r	pH	Cl <sup>-</sup>	SO <sub>4</sub> <sup>2-</sup>	
Perry K.	NA <sup>a</sup>	NA	A <sup>b</sup>	A	C <sup>c</sup>
Gibson	A	A	A	NA	C
Schahfer 14 <sup>e</sup>	A	A	A	A	NC <sup>d</sup>
Schahfer 17	A	A	A	A	NC
Gallegher	NA	A	- <sup>f</sup>	-	C
Mitchell	A	A	-	-	NC
Wabash	NA	A	-	-	C
Richmond	NA	A	-	-	C
Stout	A	A	-	-	NC
Culley	NA	A	-	-	C
Brown	NA	NA	-	-	C

<sup>a</sup> not meeting the limit of the proposed criteria<sup>b</sup> meeting the limit of the proposed criteria<sup>c</sup> denoting "corrosive", if at one "NA" exists<sup>d</sup> denoting "noncorrosive", if no "NA" exists<sup>e</sup> wet bottom ash<sup>f</sup> not determined

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