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Utilization of Waste Materials and Upgrading of Low-Quality Aggregates

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Subject Areas

25 Pavement Design
33 Construction
35 Mineral Aggregates

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CONTENTS

FOREWORD

Aggregates comprise much of the material required for highway construction and maintenance. Most base courses and subbases are composed of 95 to 100 percent aggregate; bituminous base, binder, and surface courses usually contain more than 90 percent aggregate; and portland cement concrete averages about 75 percent aggregate.

The annual aggregate requirements for highway construction currently exceed 600 million tons. Maintenance of existing facilities requires an additional 200 million tons annually. Approximately 40 percent of the total aggregate production in the United States is utilized in highway construction. Aggregate requirements increase at a rate of about 5 percent per year.

Extensive use of conventional aggregates following World War II for all types of construction activities seriously depleted the supply of conventional aggregates suitable for construction in some sections of the United States. There are geographical areas in the United States today in which the conventional aggregate shortage is approaching a critical stage. Aggregates must be transported into these areas, often over great distances, at high costs. The problem is compounded by the increasing number of existing conventional aggregate sources that are now or will shortly become unavailable because of economic reasons, zoning restrictions, pollution control, and appreciating land values.

The demand for aggregates will continue to increase in all areas of the United States in the years ahead. Under this stress, more and more geographic areas may fall into the category of "conventional aggregate-deficient areas." In these areas, selective use of conventional aggregates, beneficiated low-quality aggregates, and some waste materials may offer economical solutions to the problem of a diminishing aggregate supply.

Throughout the country there are substantial quantities of low-quality aggregates and some waste materials that do not meet existing construction specifications and consequently have been rejected for use in construction activities. Many agencies are beginning to take a second look at these materials for use in construction.

Some natural aggregates have serious deficiencies, such as poor abrasion resistance or reactive constituents, and thus are of low quality. Poor gradation is another common problem of some natural aggregates. It may be entirely possible and practical to utilize such low-quality aggregates, which have been beneficiated prior to use, in highway construction in areas that are experiencing shortages of quality conventional aggregates.

There are a number of solid wastes that may be suitable for use in highway construction. Included in this category are rubble from buildings and replaced highways, battery cases, rubber tires, scrap iron and steel, glass, and mine tailings. Some waste materials have already been utilized on a limited basis. Unfortunately, with regard to aggregate production, the supply of many waste materials is small and, except in unusual circumstances, cannot justify the facilities needed to convert the wastes into suitable aggregates. Other wastes occur in such quantities and in a sufficiently continuous supply that they do warrant the research effort and facilities required to transform them into suitable aggregates.

In this RECORD, four papers are presented. Three of them are directed to the utilization of specific waste materials in highway construction. The paper by Buck reports the results of an investigation made to evaluate the use of crushed waste concrete as concrete aggregate. Hughes and Haliburton evaluate four types of zinc smelter waste for suitability as aggregate in Oklahoma. The paper by Moulton, Seals, and Anderson is concerned with the utilization of ash from coal-burning power plants in highway construction.

The paper by Ledbetter relates criteria for performance of synthetic aggregates. Physical, chemical, mechanical, and volume change performance are considered.

Although nonconventional aggregates cannot and indeed should not be used indiscriminately in a pavement system, the authors of the papers contained herein conclude that each material studied can be successfully incorporated into a pavement. Some materials are suitable only for use in the base courses, whereas others can be used in asphalt concrete or portland cement concrete surface layers or both. Sound, durable, high-quality highways can be constructed with nonconventional aggregates when proper engineering judgment is exercised.

—Charles R. Marek

RECYCLED CONCRETE

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A discarded concrete driveway that contained siliceous aggregates and a laboratory concrete beam that contained limestone as coarse aggregate and natural siliceous sand as fine aggregate were selected. Portions of each kind of concrete were processed into aggregate sizes. Three test mixtures and two control mixtures were made. Specimens from each round of each mixture were tested for compressive strength at different ages up to 6 months, for resistance to accelerated freezing and thawing, and for volume changes due to temperature changes or to moisture effects at a constant temperature. The aggregate particles produced by crushing concrete had good particle shape, high absorption, and low specific gravity by comparison with conventional natural mineral aggregates. It is concluded that the present results are promising for the use of recycled pavements or similar concretes as concrete coarse aggregate and perhaps as fine aggregate. If additional work tends to support this tentative conclusion, then existing specifications should be revised to permit and encourage the use of this material as concrete aggregate so that existing supplies of natural aggregates are conserved and the amounts of solid wastes are reduced. The results in this work pertain only to waste concrete that is free of contamination by other materials such as sulfates.

•EXISTING supplies of natural aggregates are being depleted even as the demand for aggregates continues to rise. Because the remaining aggregate supplies are less and less accessible for convenient and economical use, the supply problem is compounded. There is a need now to develop replacements for conventional aggregates. If any of the materials that are now treated as solid wastes can be effectively utilized as aggregates, then the amount of waste that must be disposed of will be reduced, and aggregate resources will be conserved at the same time.

This report covers tests and evaluation of waste concrete of two types for use as concrete aggregate. Waste concretes from pavements and from buildings should be considered separately as raw material for concrete aggregate because concrete from buildings is likely to contain calcium sulfates from plastering or gypsum wallboard, which could raise the problem of sulfate attack if the recycled concrete is used in concrete accessible to moisture. Enough concrete of both kinds is demolished and wasted each year to make the reuse of either kind as aggregate of real benefit. The two concretes evaluated as aggregates in this investigation did not contain contaminating sulfates. One came from a driveway and the other from a test beam containing 3-in. maximum-sized aggregate.

LITERATURE REVIEW

A search of literature on the use of solid wastes as aggregate of any kind is continuing. Because the present interest was in the use of waste concrete as aggregate, special efforts were made to include work done in European countries during the late 1940s and early 1950s. This selection was made because it is known that considerable amounts of debris produced by bombing and shelling were used in rebuilding in urban areas in European countries after World War II.

The majority of the foreign work that was found described the use of bricks and of material identified as rubble for aggregates during the rebuilding process. Because rubble is a general term, references made to it were of no direct value, nor are those made to bricks valuable at this time. The results of some Russian work (1) with waste concrete will be discussed later.

Some references to the use of waste concrete as aggregate for asphaltic mixtures and as base course material in this country (2, 3) were found. However, no references to the use in the United States of waste concrete as concrete aggregates were found.

TEST PROCEDURE

Materials

Several tons of large pieces were obtained from a 6-in. thick concrete driveway that was being removed. This air-entrained concrete was about 8 years old when it was removed; it had been made by a local ready-mix concrete plant, using natural chert gravel and natural sand as the coarse and fine aggregates, to a specified strength of 3,000 psi at 28 days of age. Some of this material was processed into $\frac{3}{4}$-in. maximum-sized aggregate. All of the fines produced by this crushing and sizing operation were caught, combined, and saved.

A large unreinforced concrete beam that had been tested in flexure in the laboratory was processed into the same sizes by the same methods that were used for the waste driveway concrete. The concrete in the beam contained aggregate of 3-in. maximum size and had been wet-screened from a mixture containing aggregate of 6-in. maximum size. The beam was $9\frac{1}{2}$ months old when it was made into aggregate.

Because many concrete aggregates are predominantly siliceous or calcareous, the use of one waste concrete containing the siliceous aggregate (chert gravel and natural sand) and of another containing the calcareous coarse aggregate (limestone) and a siliceous natural sand represented two aggregates that are frequently used.

The chert gravel and the natural sand that were used as coarse and as fine aggregate were intended to be similar to the aggregates that had been used in the driveway concrete; sand from the same lot was used in making the concrete beam.

The limestone coarse aggregate used was from the same lot that was used in the concrete beam.

Portland cement meeting the requirements of Federal Specification SS-C-192g for low-alkali Type II was used. Cement from the same lot was used in the beam.

After selected physical tests of the aggregates, the materials that have been described were used in different combinations to make five concrete mixtures.

Mixtures

Three rounds of three concrete mixtures were made to evaluate the recycled concrete from the driveway as aggregate. The designations of the mixtures and aggregate combinations are as follows:

Mixture Number	Coarse Aggregate	Fine Aggregate
1	Chert gravel	Natural sand
2	Crushed concrete	Natural sand
3	Crushed concrete	Crushed concrete fines

Mixture 1 was the control mixture for this series. All mixtures were proportioned as directed in CRD-C 114 (4), which specifies the aggregate gradings, a water-cement ratio of 0.49, an air content of $6 \pm \frac{1}{2}$ percent, and a slump of $2\frac{1}{2} \pm \frac{1}{2}$ in. Although neither fine aggregate completely met the grading requirements of the test method, they were used without modification of grading for reasons described later.

One round of two other concrete mixtures was made to evalute the recycled concrete that contained limestone coarse aggregate. The designations of the mixtures and aggregate combinations are as follows:

Mixture Number	Coarse Aggregate	Fine Aggregate
4	Limestone	Natural sand
5	Crushed concrete	Natural sand

Mixture 4 was the control mixture for this pair. These mixtures were also proportioned to conform with CRD-C 114(4) except for the sand grading as already mentioned.

The specimens made from each round of each mixture were as follows:

Specimen Number	Size and Type
20	3- by 6-in. cylinders
3	$3\frac{1}{2}$- by $4\frac{1}{2}$- by 16-in. beams
4	3- by 3- by 11-in. prisms with gauge studs

Tests

The compressive strength of three 3- by 6-in. cores, which had been drilled from representative portions of the old driveway concrete, was determined according to ASTM Designation C 42. The approximate compressive strength of the concrete beam was already known.

Specimens from each mixture were tested for compressive strength, frost resistance, linear coefficient of thermal expansion, and length changes due to changes in moisture content.

The compressive strength of three cylinders from each round of each mixture was determined at ages of 7, 28, 56, 90, and 180 days according to ASTM Designation C 39.

Three beams from each round of each mixture were tested in accelerated freezing and thawing in conformance with CRD-C 114(4).

Three prisms from each mixture were tested to determine their linear coefficient of thermal expansion at 28 days according to CRD-C 39 (4). The test plan required testing only one round of specimens from each mixture, but the test was repeated for the third round of mixture 3 because of difficulties with loose inserts in the specimens from the first round.

One prism from each round of each mixture was stored in the moist room at relative humidity above 90 percent and temperature of 73 ± 2 F. The lengths were measured at 1, 28, and 90 days.

TEST RESULTS

Both of the coarse aggregates and the sand made from waste concrete had good particle shape as judged by visual inspection.

Most of the particles in the coarse aggregate sizes of crushed driveway concrete were individual chert particles or crushed portions of them with partial coatings of mortar adhering to the chert. Small proportions of chert particles and of mortar particles were also present. The same types of particles were present in the fine aggregate sizes, with the amounts of mortar and rock particles increasing at the expense of mortar-coated rock with decreasing particle size.

About 75 percent of the $\frac{1}{2}$- and $\frac{3}{8}$-in. sizes of the crushed-beam concrete are particles composed of rock with partial coatings of mortar, with the other 25 percent consisting of individual particles of limestone.

The compressive strength of the beam was about 8,000 psi before it was crushed. The compressive strengths of three 3- by 6-in. cores drilled from different portions of the driveway concrete and broken in the laboratory at about 9 years of age are as follows:

Core Number	Compressive Strength (psi)
1	6,510
2	5,500
3	5,960
Average	5,990

The absorption and specific gravity of the aggregates that were used are given in Table 1. The gradings of the natural sand and of the fines from the crushed driveway concrete are given in Table 2 with the fine aggregate grading prescribed in CRD-C 114(4). The absorptions and specific gravities of the natural sand, the chert gravel, and the crushed carbonate rock are within the usual range for these materials. The two crushed concrete coarse aggregates had high absorptions and rather low specific gravities. The crushed concrete fines used as fine aggregate had absorptions of 7.6 and 9.0 (8.3 ± 0.7) in repeat determinations and low specific gravity. The relatively high absorptions and low specific gravities are to be expected in aggregates produced by recycling concrete.

The data given in Table 2 show that neither of the fine aggregates meets the grading requirements of CRD-C 114 (4) and that the concrete fines depart more widely from the limits than the natural sand. The concrete fines were used in the grading in which they were produced to see what effect this might have on the test results; it was not considered worthwhile to alter the grading of the natural sand.

Properties of the freshly mixed concrete are given in Table 3. Mixture 3, which contained only crushed concrete as aggregate, had lower slump and higher cement content than the other siliceous mixtures. This mixture appeared wet even though it was stiffer than its companion mixtures 1 and 2. When natural sand was used as fine aggregate, there was little difference in slump, air content, or cement content between the control mixtures and their companions, mixtures 2 and 5 respectively.

Compressive strengths of all mixtures are given in Table 4, through 180-day tests. Mixtures 2 and 5, containing waste concrete as coarse aggregate, ranged from about 300 to 1,300 psi lower than the control mixtures at corresponding ages. Mixture 3, with crushed concrete coarse and fine aggregates, is intermediate in strength between mixtures 1 and 2. Mixture 3 may have had higher strength than mixture 2 because the water-cement ratio of mixture 3 was actually lower than that of mixture 2. The lower strengths of mixtures 2, 3, and 5 will be discussed later.

The results of the freezing-thawing tests are given in Table 5. Although the average DFE_{300} values (durability factor based on modulus of elasticity after 300 cycles of freezing and thawing) of 3, 23, and 28 for mixtures 1, 2, and 3 are low, the increased resistance to freezing and thawing indicated by the mixtures containing crushed chert-gravel concrete (2, 3) as aggregate is striking. Probable reasons for this will be discussed later. A reversed trend is shown by the average DFE_{300} values for mixtures 4 and 5, with the control mixture showing slightly higher DFE.

The linear coefficients of thermal expansion are given in Table 6. The value for control mixture 1 is as expected for concrete containing chert gravel and siliceous sand, and the coefficients of mixtures 2 and 3 are similar. The coefficient of mixture 4 is as expected for a limestone coarse aggregate with siliceous natural sand. The coefficient of mixture 5 is higher; the difference is probably significant, but the value is still lower than the coefficients of the first three mixtures.

Length change of prisms stored in the moist room at high humidity and constant temperature is given in Table 7. The test mixtures have about the same amount of change as the corresponding control mixtures.

DISCUSSION

The intent in this work was to evaluate crushed waste concretes similar to concrete used in pavements for use as concrete aggregates. Pavement and building concrete should be considered separately because sulfate from plaster or wallboard may be associated with building concrete and may create the problem of sulfate attack. The chert-gravel concrete from the driveway and the crushed limestone concrete from the laboratory beam used in this work are believed to be fairly similar to pavement concrete, except that the beam contained aggregate of 3-in. maximum size. One concrete contained chert gravel and natural sand, and the other contained limestone and natural sand; both had compressive strengths of about 6,000 and 8,000 psi. Because the beam was not reinforced and the concrete from the driveway contained wire mesh, there are no results from this work on possible problems in processing concrete that contains

Table 1. Specific gravities and absorptions of aggregates.

Aggregate	Bulk Specific Gravity Saturated Surface-Dry[a]	Absorption[a] (percent)
Crushed siliceous concrete		
Coarse	2.43	4.0
	2.44	4.3
Fine	2.34	7.6
	—	9.0
Crushed calcareous concrete (coarse)	2.52	3.9
Chert gravel	2.52	2.6
Limestone	2.67	0.8
Natural sand	2.63	0.4

[a]Tested in accordance with ASTM Designations C 127 and C 128.

Table 2. Gradings of fine aggregates.

Aggregate	Percent Passing U. S. Standard Sieve						
	No. 4 (100)[a]	No. 8 (85 ± 3)	No. 16 (65 ± 5)	No. 30 (45 ± 5)	No. 50 (21 ± 5)	No. 100 (7 ± 2)	No. 200
Crushed siliceous concrete (fine)	100.0	77.1	58.7	42.6	23.5	12.4	6.6
Natural sand	98.1	87.6	74.4	52.6	25.6	7.0	1.2

[a]Figures in parentheses are fine aggregate gradings prescribed in CRD-C 114.

Table 3. Selected physical properties of the five concrete mixtures tested.

Mixture Number[a]	Round	Slump[b] (in.)	Air[c] (percent)	Cement Content (lb/yd³)
1	1	2¼	6.0	461
	2	2½	6.3	461
	3	2½	6.3	461
2	1	2½	5.7	461
	2	2½	5.8	461
	3	2½	6.0	461
3	1	2	6.3	498
	2	2	6.0	508
	3	2	5.9	508
4	—	2¾	6.0	508
5	—	2½	6.1	489

[a]Tested in accordance with CRD-C 114 using the slump and air content as controls. All mixtures had 0.49 water-cement ratio.
[b]The specified slump is 2½ ± ½ in.
[c]The specified air content is 6 ± ½ percent.

Table 4. Average compressive strengths of the five concrete mixtures tested.

Mixture Number	Round[a]	Compressive Strength (psi)				
		7 Days	28 Days	56 Days	90 Days	180 Days
1	1	2,880	4,420	5,160	5,230	5,660
	2	2,360	3,840	4,400	4,890	5,120
	3	2,520	4,160	4,530	5,070	5,050
	Combined	2,590	4,140	4,700	5,060	5,280
2	1	1,910	2,880	3,480	3,900	3,850
	2	1,990	3,210	3,620	3,840	4,090
	3	2,030	3,050	3,650	3,900	4,140
	Combined	1,980	3,050	3,580	3,880	4,030
3	1	2,440	3,210	3,790	4,270	4,570
	2	2,210	3,570	3,930	4,440	4,640
	3	2,240	3,430	3,700	4,120	4,340
	Combined	2,300	3,400	3,810	4,280	4,520
4	—	3,180	4,510	4,790	5,320	5,530
5	—	2,580	4,150	4,000	4,660	4,840

Note: Tested in accordance with CRD-C 114.
[a]The individual round values are for three 3- by 6-in. cylinders; the combined values are for nine 3- by 6-in. cyclinders.

steel bars. However, two literature sources (2, 3) and a personal communication on the use of recycled highway concrete as aggregate for asphaltic mixtures and as a base course material indicate that processing of waste concrete that contains steel reinforcing bars is practical.

Strength, durability, and volume-change tests were made to see if there were substantial differences between test mixtures that contained crushed concrete as aggregates and control mixtures. A comparison between some 1946 test results (1) on waste concrete from the U.S.S.R. and results of tests made at the waterways experiment station is given in Table 8. Where comparisons are possible, the agreement between the Russian results (1) and the present work is excellent.

The mixtures containing crushed concrete as fine aggregate required more cement and were slightly stiffer; however, the increased cost for additional cement should be partly or wholly compensated by the advantage to be gained by using the crushed concrete fine aggregate instead of having to dispose of it. Blending with natural sand, modification of mixture proportions, or use of water-reducing admixtures might permit lowering the cement content and improve the workability when using crushed concrete as sand. None of the test information in this work rules out its use. Its use in an unusual grading did not seem to have any appreciable effect on the test results.

The reasons for the lower compressive strengths of mixtures containing crushed concrete as coarse aggregate (as compared to mixtures containing only natural aggregates) are not known at this time. Several explanations have been considered and rejected or cannot be proved at present. It should be recalled that, although the strengths were lower, they were satisfactory for many uses. It is hoped that slight adjustments of such mixtures will improve their strengths.

The improved frost resistance of mixtures 2 and 3, containing concrete as aggregate, compared to control mixture 1 from a DFE_{300} of 3 to 23 and 28 was substantial. It is thought that this improvement may have occurred because the old mortar, which coats many of the crushed concrete particles, effectively seals off the voids in the frost-susceptible porous chert particles and prevents them from taking up enough moisture to be damaged by freezing.

Comparison of data for test mixtures containing recycled waste concrete as coarse aggregate with data for control mixtures shows the following:

1. There were no unusual problems in mixing or working with the test mixtures,
2. The test mixtures have compressive strengths that are about 300 to 1,300 psi lower than the corresponding control mixtures at all ages tested through 180 days,
3. The resistance to accelerated freezing and thawing is greatly improved when the waste concrete originally contained chert-gravel coarse aggregate,
4. The resistance to freezing and thawing is a little lower but essentially comparable when the waste concrete originally contained limestone coarse aggregate, and
5. Volume changes in response to temperature changes or to continued exposure to moisture at a constant temperature were similar and normal.

The findings for mixture 3, which contained waste chert-gravel concrete as coarse and fine aggregate, were generally like the control mixtures except that cement demand was somewhat higher and workability slightly lower than with the control mixture.

CONCLUSIONS AND RECOMMENDATIONS

The results indicate many reasons in favor of the use of crushed discarded concrete pavements as concrete aggregates. If additional work indicates that the lower concrete strengths obtained with waste concrete as coarse aggregate are not a serious problem, then all existing specifications should be revised to permit and encourage the use of crushed pavement or similar concrete as concrete coarse aggregate.

If, in addition, the mild undesirable effects of waste concrete fine aggregate on workability and cement content of concrete mixtures can be eliminated or reduced or tolerated, then the use of this material should also be encouraged by specification revisions.

Table 5. Durability factor for concrete beams in accelerated freezing and thawing.

Mixture	Round (DFE300)[a] 1	2	3	Combined
1	4	4	2	3
2	28	22	19	23
3	30	28	25	28
4	62	—	—	—
5	45	—	—	—

Note: Tested in accordance with CRD-C 114.

[a]The values by round are for three beams; the combined values are the average of nine values.

Table 6. Linear coefficient of thermal expansion of the five concrete mixtures tested.

Mixture Number[a]	Round (coefficient value)[b] 1	2[c]	Combined
1	6.3	—	—
2	6.1	—	—
3	5.6	5.7	5.6
4	3.6[d]	—	—
5	4.7[d]	—	—

[a]Tested in accordance with CRD-C 39.

[b]Round value is for 3 specimens; combined value is for 6 specimens.

[c]Test repeated with different specimens to verify low value; gauge studs were recemented.

[d]A few obviously faulty values were discarded. This lowered the average coefficient values by only 0.2 and 0.3.

Table 7. Length changes of concrete specimens stored at constant moisture and temperature.

Mixture Number	Round	Specimen[a]	Length Increase[b] (percent) 28 Days	90 Days
1	1	1	0.013	0.019
	2	5	0.016	0.018
	3	9	0.010	0.008
		Average	0.013	0.015
2	1	1	0.014	0.023[c]
	2	5	0.010	0.011
	3	9	0.012	0.014
		Average	0.012	0.016
3	1	1	0.017[c]	0.036[c]
	2	5	0.007	0.009
	3	9	0.007	0.011
		Average	0.010	0.019
4	1	1	0.003	0.001
	2	5	—	—
	3	9	—	—
		Average	0.003	0.001
5	1	1	0.003	0.002
	2	5	—	—
	3	9	—	—
		Average	0.003	0.002

[a]Specimens were stored in a moist room at more than 90 percent relative humidity and a temperature of 73 ± 2 F.

[b]The initial reference length was measured at 1 day for mixtures 1 through 3 and at 2 days for mixtures 4 and 5.

[c]Loose inserts.

Table 8. Comparison of test results.

U. S. S. R.	Waterways Experiment Station
New concrete will be no better than the waste concrete that is used as aggregate.	No comparison possible. Waste concrete used was of good quality.
The use of concrete fines as sand requires an undue increase in the cement content of a mixture.	Mixture 3 was the only one that contained concrete fines as sand. It required 47 lb/yd^3 more cement than mixtures 1 or 2 with natural sand. This is not regarded as excessive.
Compressive strengths are lower when concrete is used as aggregate.	Concretes containing waste concrete as coarse aggregate range from 300 to 1,100 psi lower in compressive strength than corresponding control mixtures.
The specific gravity of crushed concrete aggregate tends to be lower than that of natural aggregates.	Work at waterways experiment station confirms this; in addition, the absorption tends to be high.
The cement factor can be lowered if the crushed concrete aggregate is moistened, not saturated, before use.	The coarse aggregates were inundated; the fine aggregates had moisture added 24 hours before mixing the concrete to satisfy their absorption.
For equal compressive strengths, the flexural strength of mixtures with crushed concrete aggregate is higher than for control mixtures.	No flexural tests were made.
Mixtures with crushed concrete aggregate stiffen rapidly but consolidate well with vibration.	No such difference noted with crushed concrete as coarse aggregate only, but mixture 3 with all aggregate crushed concrete was stiffer than the control even though it appeared wet.

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SYNTHETIC AGGREGATES FROM CLAY AND SHALE: RECOMMENDED CRITERIA FOR EVALUATION

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The transformation of a clay mineral into an inert, stable, amorphous silicate under the combined action of heat and time is an extremely complex phenomenon that occurs under nonequilibrium conditions. Essentially what occurs is, first, the loss of free and absorbed water and, then, the loss of hydroxylated water coupled with the decomposition of minerals such as calcite, pyrite, and dolomite. These chemical transformations take time; therefore, depending on the amount of heat and size of the particle, different degrees of transformation can occur. An interesting special phenomenon is the process of "bloating" where some clays "puff up" slightly. Examination under the microscope of a properly bloated clay reveals a sponge-like structure where the holes, or "blebs" as they are termed, are numerous but small. The dry unit weight of this aggregate (coarse sizes) is often in the range of 35 to 45 lb/ft^3. With such a small "bleb" structure, the resultant aggregate can be strong and durable. To fully develop the criteria for performance, or durability, the research was considered from four aspects: physical performance, chemical performance, mechanical performance, and volume change performance. The following findings relate to the limitations imposed on this study, and further generalizations may not be warranted: Sound, durable, high-quality highways and bridges can be constructed with synthetic aggregates, provided the aggregates meet certain requirements; not all functional uses of synthetic aggregates require aggregate of the same quality; and laboratory evaluations can, with reasonable assurance, predict field performance of synthetic aggregates.

•SYNTHETIC aggregates made from clay or shale are not new. Brick or ceramic tile is a "synthetic" rock in that it is man-made, and broken bricks, made from clay, were used by the Romans many centuries ago as an aggregate in concrete made with pozzolanic lime mortar (1). For the purposes of this report, synthetic aggregates are defined as those aggregates made by the thermal transformation of agglomerations of clays or shales into essentially amorphous silicates.

With the development of the rotary kiln process in 1917 by Hayde (2), the economical production of lightweight synthetic aggregates became a reality, and today a review of the technical literature reveals the almost universal use of lightweight synthetic aggregates in concrete. Bridges and buildings have been built in the United States (3), England, Germany, France, Russia, Japan, and Australia.

Concurrent with the rapid rise in the use of lightweight synthetic aggregates has been the depletion of available natural aggregates in many portions of the United States (4). For these and other reasons, the use of synthetic aggregates has continued to increase, and demand is expected to grow.

In 1964 the Texas Highway Department, realizing the increasing need for synthetic aggregates in all types of highway construction, in cooperation with the Federal Highway Administration, initiated a 6-year study on synthetic aggregate research. The objective of that study was to develop a recommended synthetic aggregate classification system that would identify quality synthetic aggregates for use in highway construction. In other words, the study was aimed at developing accelerated laboratory performance criteria that, if met, would ensure that any qualifying synthetic aggregate made from clay or shale by a thermal process would perform satisfactorily throughout the service life of the highway system. This paper summarizes the results of that study.

TRANSFORMATION PROCESS

The transformation of a clay mineral into an inert, stable, amorphous silicate under the combined action of heat and time is an extremely complex phenomenon that occurs under nonequilibrium conditions. Essentially what occurs is, first, the loss of free and absorbed water and, then, the loss of hydroxylated water coupled with the decomposition of minerals such as calcite, pyrite, and dolomite. These chemical transformations take time; therefore, depending on the amount of heat and size of the particle, different degrees of transformation can occur.

An interesting special phenomenon is the process of "bloating" where some clays "puff up" slightly. Examination under the microscope of a properly bloated clay reveals a sponge-like structure (Fig. 1) where the holes, or "blebs" as they are termed, are numerous, but small. The dry unit weight of this aggregate (coarse sizes) is often in the range of 35 to 45 lb/ft^3. With such a small "bleb" structure the resultant aggregate can be strong and durable. This bloating action is not fully understood, but it is surmised that two conditions must occur almost simultaneously. First the clay must reach the proper pyroplastic condition to be able to expand without rupturing (not too brittle) or subsequently collapsing (not too fluid). Second, the clay must generate the bloating force in the form of gas evolution at just the right time and in just the right amount. Needless to say, these two conditions are restrictive, and thus there are many clays that either do not bloat at all or do not bloat properly.

The state of the art of the production of these bloated aggregates, termed structural lightweight aggregates, is well advanced, and standard specifications have been developed (such as ASTM Designation C 330). There are two methods of burning in wide use in the United States. One is the rotary kiln method where the clay particles are fed into the upper end of a slowly rotating, inclined tube (kiln) and travel slowly to the lower burner end where they reach temperatures as high as 2200 F. The clays are in the kiln for 30 min to an hour, during which time the transformation process is essentially completed. The other is the sintering method in which raw clays are carried on a moving grate through an ignition hood where the clays are heated rapidly to the desired temperature and then allowed to cool slowly as the grate continues to move. Total firing time may be as short as 10 min.

In 1966 there were 65 rotary kiln plants and 18 sintering plants in the United States and Canada. This number has increased significantly since that time.

DEVELOPMENT OF EVALUATION CRITERIA

Strength and Performance

Based on the wealth of research information, it became evident that synthetic aggregates could be made sufficiently strong for almost any anticipated use (3). Also it has been proved that some synthetic aggregates could satisfactorily perform throughout their service life (3). Furthermore the end-product evaluations for strength are generally well established [for example, compressive strength for portland cement concrete (PCC) and stability values for asphaltic concrete]. Thus the main thrust of this research was the development of performance criteria (as distinct from strength) to evaluate new aggregates with no performance records.

To fully develop the criteria for performance, or durability, the research was considered from four aspects: physical performance, chemical performance, mechanical performance, and volume change performance. These are defined in Table 1. It should be emphasized that performance involves aspects other than these four and further that these four are not independent of each other. But each aspect was treated somewhat independently in the conduct of this research.

Physical Performance

The physical performance of lightweight aggregates for PCC was tied to the generally accepted rapid freeze-thaw test for concrete immersed in water (ASTM Designation C 666-Procedure A). The rationale behind this test was the contention that a high-quality, air-entrained concrete containing a synthetic aggregate would only show significant distress in 300 cycles of freeze-thaw if the synthetic aggregates were not physically durable. Results of this phase of the investigation have been reported (5, 6). Essentially it was found that, due to the relatively high saturation capabilities of many synthetic lightweight aggregates (Fig. 2), almost any aggregate can be rendered nondurable if it enters the PCC mix in a relatively saturated condition. Conversely almost any aggregate can be made durable if it is mixed in a relatively dry condition. The volume term "saturation" is used here rather than the more commonly used weight term "absorption." Saturation is the volume of voids filled with water divided by the total volume of voids available. Because many lightweight aggregates contain up to 50 percent voids, a considerable quantity of water can be absorbed into the aggregate.

Laboratory tests have shown that, when the degree of saturation is kept below about 25 percent (by volume of voids), the concrete exhibits good resistance to deterioration from freezing and thawing (Fig. 3) (6). It was also found that some synthetic aggregates reached this critical saturation after less than 30 min of immersion in water, while others had to be immersed several days before becoming critically saturated (Fig. 2). The latter aggregates are often termed "coated" aggregates because they possess a denser surface coating, or rind, that inhibits absorption. Because the degree of saturation is very difficult and time-consuming to determine in the field, it was recommended that a maximum rate of saturation be imposed on the highest quality aggregate to be used in concrete subjected to a freeze-thaw environment and a limit of 15 percent saturation after 100 min of immersion in water was selected[1].

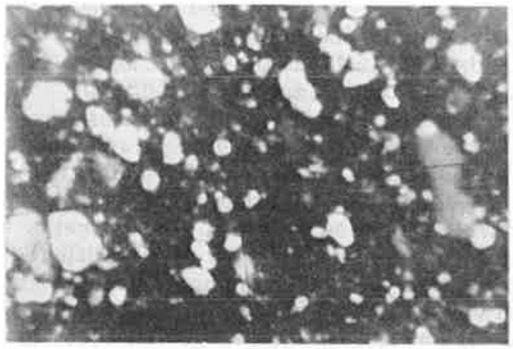
With the increasing emphasis on highway surfaces with higher and longer lasting skid resistance, the use of lightweight synthetic aggregate in asphaltic concrete and surface treatments has been shown to be an economical way to provide excellent, long-lasting, skid-resistant surfaces (7, 8). As with any new material, the physical performance of these aggregates must be demonstrated. It is reasonable to expect that a material that readily absorbs water might not perform satisfactorily, if sufficiently saturated and subjected to freezing and thawing. Gallaway and Harper developed an aggregate freeze-thaw test that approximates the nature of such exposure in the field (9). Based on these results, maximum permissible weight losses have been established for synthetic aggregates.

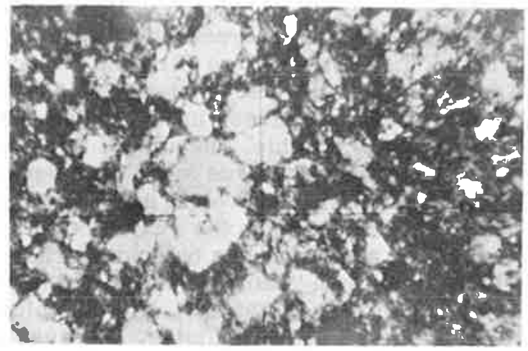
Chemical Performance

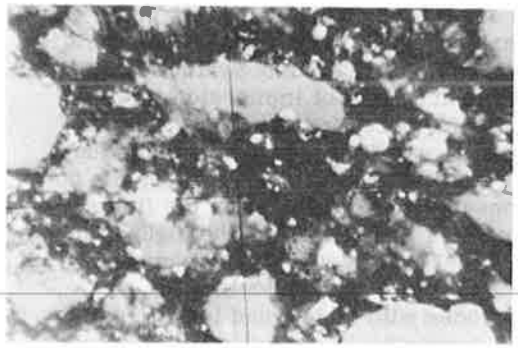
The excellent record of many structural lightweight aggregates in concrete (3) attests to their chemical stability. Further, bricks made from clay have lasted for centuries. However, it is a fact that synthetic aggregates have been transformed from raw clay or shale. What if this transformation is incomplete? Some accelerated laboratory testing was needed to indicate the completeness of this thermal transformation. Such a test, the pressure slaking test, was developed and reported (10). The test involves the cooking of the aggregates underwater in a common pressure cooker and then subjecting them to severe agitation in water. The agitation in water disperses any rehydrated material, and it also produces some abrasion loss. The total loss in weight through a specified sieve is a measure of the degree of transformation. Pressure-slaking losses were shown to be a function of firing temperature, and the results were compared with losses of commercially produced aggregates (both from plants and from in-service locations) (Fig. 4).

[1] The original manuscript of this paper included Appendix A, Texas Highway Department Test Method 433-A; Appendix B, Texas Highway Department Test Method 109-E(Part I); Appendix C, Texas Highway Department Test Method 432-A; and Appendix D, Texas Highway Department Test Method 431-A. The appendixes are available in Xerox form at cost of reproduction and handling from the Highway Research Board. When ordering, refer to XS-44, Highway Research Record 430.

Figure 1. Photomicrographs of sample RGR.


1800 °F


2000 °F


2200 °F

Nichol prisms partly crossed black areas indicate amorphous glass matrix; gray areas indicate pores, and white areas are crystalline minerals (52X).

Table 1. Performance concepts.

Type of Performance	Definition
Physical	The resistance of aggregate to repeated stressing, either from internal sources such as freezing and thawing or from external sources such as fatiguing from traffic loading.
Chemical	The resistance of aggregate to the various chemical reactions occurring on, or in, the aggregate during its service life. An example would be any rehydration of synthetic aggregate back to a clay or shale.
Mechanical	The resistance of aggregate to the abrasive wear of traffic on its exposed surface.
Volume change	The resistance of aggregate to detrimental volume changes of PCC from shrinkage and creep.

Figure 2. Degree of aggregate saturation versus immersion time in water.

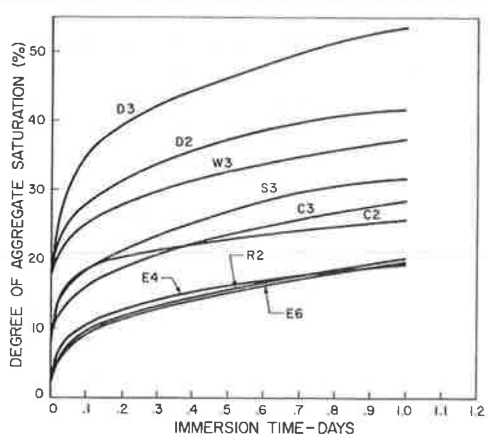
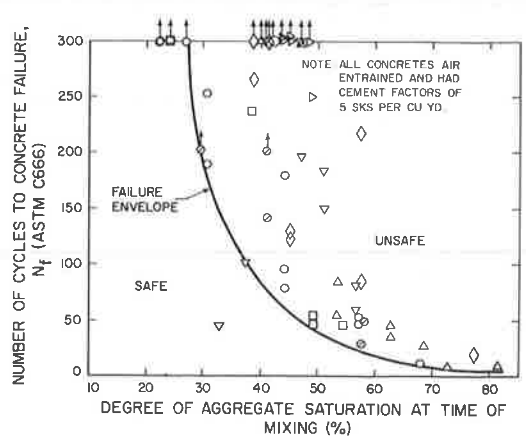


Figure 3. Concrete freeze-thaw failure envelope (6).



The other major adverse chemical reaction that was expected was the alkali-aggregate reactivity in PCC. Because the transformed clay mineral is composed of silicates, some of these might react with PCC. Laboratory investigations of many aggregates did reveal a very few adverse reactions. However, no suitable aggregate test was found that would isolate reactive synthetic aggregates. Thus, a more time-consuming concrete autoclave expansion test, patterned after ASTM Designation C 151, was recommended as a source qualification test. This phase of the study is covered elsewhere (11).

Mechanical Performance

The ability of any highway surface to satisfactorily resist abrasion and wear from traffic has long been of concern to highway engineers and even more so in recent years as prolonged high skid resistance must now be provided. In this study several abrasion tests were evaluated, including the standard Los Angeles abrasion test (ASTM Designation C 131) and the concrete abrasion resistance test (ASTM Designation C 418). In addition an aggregate sandblast test was developed to measure a synthetic aggregate's resistance to abrasion (12). The overall result was that, although the standard Los Angeles abrasion test was open to considerable doubt as to its applicability, no suitable alternative test was found. Therefore, the Los Angeles abrasion test (ASTM Designation C 131) was recommended as a mechanical performance criterion.

Volume Change Performance

Volume changes in PCC from shrinkage and creep can be extremely detrimental. The role of aggregate, as an inert filler, has been to reduce the volume change from that experienced with cement paste alone (1). Lightweight synthetic aggregates, because of their water-absorbing characteristics, can contribute to the concrete volume change phenomena, both constructively and destructively (3). In this phase of the investigation the shrinkage-cracking characteristics of lightweight aggregate concrete were investigated and reported (13). Although the aggregates used were found to influence the amount of shrinkage experienced, no new tests were found to be needed to identify or control shrinkage of lightweight concrete. ASTM Designation C 330 specifies a concrete shrinkage that, in most cases, has been found to be satisfactory.

Evaluation Criteria

Based on the four performance concepts discussed, a classification system for synthetic coarse aggregates was developed (Table 2). The table divides the material into two classes: Class I is subdivided into four groups (A, B, C, and D) of descending physical requirements, and class II is subdivided into three groups of descending physical requirements (A, B, and C).

This classification system is not intended to replace existing requirements for high-quality aggregates in highway construction; rather it is offered as a supplement to existing aggregate requirements. For example, the requirements for clean, sound, durable aggregates of specified gradations (depending on their use) are not mentioned here because they are adequately covered in current highway department specifications.

A functional grouping of coarse synthetic aggregates is given in Table 3. In this table the recommended permissible coarse-aggregate group defined in Table 2 is shown for each highway function, from surface treatments to base materials.

CONCLUSIONS

It should be emphasized that the findings of this paper relate to the limitations imposed on this study, and further generalizations may not be warranted. Conclusions are summarized as follows:

1. Sound, durable, high-quality highways and bridges can be constructed utilizing synthetic aggregates, provided the aggregates meet certain requirements.
2. Not all functional uses of synthetic aggregates require the same quality aggregate. Thus, any classification system must recognize, and allow, synthetic aggregates of differing qualities for different uses.

Figure 4. Pressure slaking loss versus maximum kiln temperature (10).

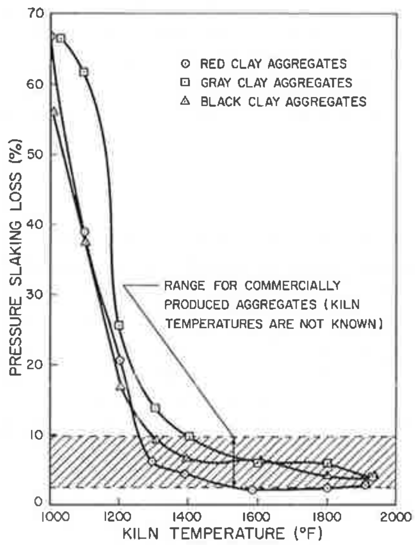


Table 2. Classification system for synthetic coarse aggregates.

Class	Group	Dry Loose Unit Weight (lb/ft³)[a] Maximum	Dry Loose Unit Weight (lb/ft³)[a] Minimum[b]	Maximum 100-Min Saturation (percent)[c]	Maximum Aggregate Freeze-Thaw Loss (percent)[d]	Maximum Pressure Slaking Value (percent)[e]	Maximum Los Angeles Abrasion Loss (percent)[f]
I (bloated)	A	55	40	15	7	6	30
	B	55	35	15	7	6	35
	C	55	35	20	15	6	40
	D	55	35	—	—	10	45
II (nonbloated)	A	—	55	—	7	6	35
	B	—	55	—	15	6	40
	C	—	55	—	—	10	45

[a]Tested in accordance with ASTM Designation C 330.
[b]This minimum should not preclude the experimental use of a lighter weight aggregate from any new source or upgraded existing source.
[c]Tested in accordance with Texas Test Method 433-A.
[d]Tested in accordance with Texas Test Method 432-A.
[e]Tested in accordance with Texas Test Method 431-A.
[f]Tested in accordance with ASTM Designation C 131.

Table 3. Functional grouping of synthetic coarse agggregates.

Function	Permissible Aggregate Group
Surface treatments[a]	IA
Asphaltic concrete surfaces[b]	IB, IA, IIA
Asphaltic concrete bases	IA, B, C, D, IIA, B, C
Exposed lightweight PCC structures[c]	IA, B
PCC pavements[c]	IA, B
Unexposed PCC bases[c]	IA, B, C, IIA, B
Flexible base materials	IA, B, C, D, IIA, B, C

[a]The aggregate should be kept dry during construction.
[b]The 100-min saturation requirement can be waived.
[c]A maximum concrete autoclave expansion (ASTM Designation C 151 modified) of 1,500 μin./in. should be required.

3. Laboratory evaluations can, with reasonable assurance, predict field performance of synthetic aggregates. Thus new synthetic aggregate sources, if they satisfactorily pass certain laboratory requirements, can reasonably be expected to perform satisfactorily in the field.

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USE OF ZINC SMELTER WASTE AS HIGHWAY CONSTRUCTION MATERIAL

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Oklahoma has large quantities of smelter waste, resulting from zinc and lead mining operations, located principally in the north central, northeast, and eastern portions of the state. In their current condition and location, these waste piles are extremely unsightly, and they kill or damage adjacent vegetation and contribute significantly to both surface and groundwater pollution. Before any solid waste material may be accepted for use as construction aggregate, it must possess acceptable physical properties. These properties, determined from standard tests, include mechanical strength, surface texture, particle shape, resistance to polishing or skid resistance, specific gravity, water absorption, chemical stability, particle size distribution, resistance to abrasion, and resistance to frost action (1). The most economical aggregate is often the one closest to the construction site. However, even with modern aerial survey techniques, nearby materials cannot always be located, thus expenses incurred by aggregate transportation increase overall construction costs. An often overlooked method of lowering aggregate material cost is the use of synthetic or artificial aggregates, which are generally considered to be aggregates produced by some chemical and/or physical process. Highway engineers are always hesitant to use artificial aggregates unless they are of proven field quality. However, several types of artificial aggregates have been utilized with reasonable success under limiting conditions.

•TO satisfy a rapidly growing population, the highway engineer is building more and more roads, using naturally occurring aggregates that are being both depleted and covered by expanding cities. Also, one of the hardest environmental pollution problems to solve is the disposal of solid mineral wastes. However, if our economy demands that these mineral wastes be produced, then disposal methods must be developed in order for man to live with them. A common solution for the two problems might be utilization of mineral wastes as aggregates for road construction, especially in many areas of the United States where natural aggregates are being depleted and therefore expensively transported from other regions.

Extensive use has been made of ground or pulverized reef shell as a flexible pavement aggregate for road and airfield construction in the Gulf-states region of the United States. Problems in evaluating laboratory mix design results stem from the reef shell particle size and shape and the inadequacy of the Marshall method of design to predict field behavior (2, 3, 4).

Another widely used source of artificial aggregate is the waste product in the production of iron, called slag. Asphaltic concrete mix designs utilizing "blast furnace slag" are characterized by optimum asphalt contents, which are not indicative of their field performance. Use of unslaked "open hearth slag" can result in volumetric expansion of the aggregate, producing failures in portland cement concrete (PCC) and heaving of slabs overlying the waste if used as a base material (5, 6, 7, 8).

Expanded clay and shale aggregates manufactured from shale aggregates commonly not considered suitable for highway pavements are currently being used in portland cement and asphaltic concretes. The aggregates exhibit desirable features such as low weight and high strength, but the raw material must be located in sufficient quantities and quality to warrant the establishment of a multimillion dollar thermal treatment plant (9, 10).

Research work is currently being conducted at the University of Missouri at Rolla concerning the use of ground waste glass as an aggregate in asphalt paving mixtures. Although at this time no published information concerning field test results is available, problems arising from stripping and public response to driving on broken glass will undoubtedly occur (11).

With added attention toward solid waste disposal, the use of compacted sewage ash as a fine aggregate is being studied. The compacted ash does not swell, slake, or lose its strength on soaking, but it is quite corrosive to metals and should not be compacted by a sheepsfoot roller (12).

Additional information regarding the use of these and other aggregates is summarized elsewhere (13).

WASTE MATERIALS TESTED IN THIS STUDY

Four types of Oklahoma zinc smelter waste and a very fine "blow sand" were studied during the investigation.

Two types of smelter waste were obtained from the Eagle Pitcher Co. smelter located at Henryetta, Oklahoma. One of the samples from the Henryetta smelter was reddish in color and was given the name Henryetta Red Tailings (HRT). Visual observation of individual grains revealed a very porous and cohesionless material, cubical in shape and having a sharp angular texture, as shown in Figure 1. The larger particles of the material were brittle in nature, but grain sizes passing the No. 10 sieve were very durable. Specific gravity, percentage of water absorption, and grain size distribution for HRT are given in Table 1. The other cohesionless smelter waste from the Eagle Pitcher plant was similar in surface texture, particle shape, and porosity but was black in color. This material was called Henryetta Black Tailings (HBT) and is shown in Figure 2. HBT also exhibited the same brittleness of large particles and durability in smaller grain sizes as was found for HRT. Specific gravity, percentage of water absorption, and grain size distribution for HBT are also given in Table 1.

Two additional smelter wastes were obtained from the Blackwell Zinc Co., located at Blackwell, Oklahoma. One cohesionless material, shown in Figure 3 and called Blackwell Tailings (BT), was black in color. The material oxidized in the presence of water and turned a reddish-yellow color. BT was also porous in nature and sharp and angular in texture, and particles retained on the No. 10 sieve appeared shiny or glassy. Smaller particles of BT were not as durable as those found in HBT or HRT. Percentage of water absorption, specific gravity, and grain size distribution data for BT are given in Table 1.

BT is currently sold to other commercial smelting firms for removal of trace metals other than zinc. The economic feasibility of using BT as a road construction aggregate as opposed to its utilization for additional smelting was not considered in this study.

The remaining Blackwell Zinc Co. waste material to be studied was called Blackwell Condenser Tailings (BCT). BCT, as shown in Figure 4, was also cohesionless, white in color, and relatively fine-grained (Table 1). BCT, like the other waste materials studied, has a cubical shape and sharp angular texture; however, it is nonporous. The smelting processes undergone by the waste materials are not described because they were considered privileged information by both companies providing smelter waste for analysis.

Another cohesionless aggregate utilized during the investigation was a naturally occurring very fine, poorly graded sand, locally called "blow sand" but here called Sapulpa sand (SS), obtained from a site located 4 miles west of Sapulpa, Oklahoma. SS was substituted for fine aggregate not naturally occurring in the smelter waste. The grain size distribution for SS is given in Table 1. This sand is typical of the fine to very fine sands that are relatively abundant in most parts of Oklahoma.

Figure 1. Sized particles of HRT.

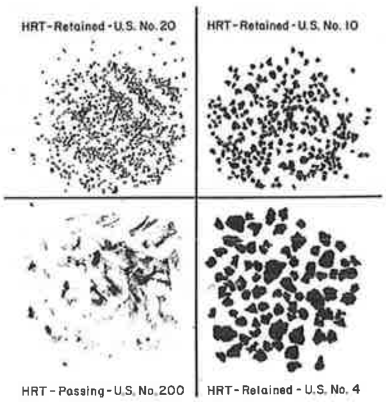


Figure 2. Sized particles of HBT.

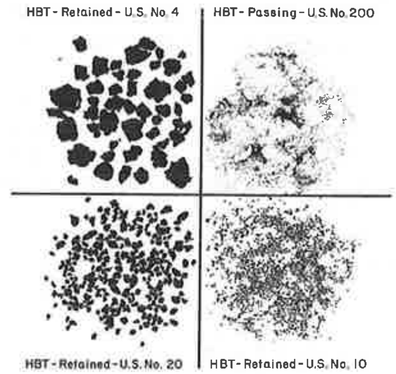


Table 1. Physical properties of test materials.

Test Material	Grain Size Distribution, Total Percent Passing U. S. Standard Sieve									Bulk Specific Gravity	Water Absorption (percent)
	½ In.	⅜ In.	No. 4	No. 10	No. 20	No. 40	No. 80	No. 100	No. 200		
HRT	99.7	95.3	90.2	59.6	24.5	10.9	4.0	3.0	1.6	2.86	3.78
HBT	99.1	96.2	69.8	31.9	11.5	4.4	1.6	1.2	0.7	2.37	4.87
BT	100.0	99.7	90.4	64.2	29.7	10.1	2.4	1.7	0.8	3.14	5.00
BCT	100.0	100.0	98.7	85.6	41.1	23.9	12.5	9.4	4.9	2.18	3.41
SS	100.0	100.0	100.0	100.0	100.0	100.0	87.0	35.4	13.1	2.66	—

Figure 3. Sized particles of BT.

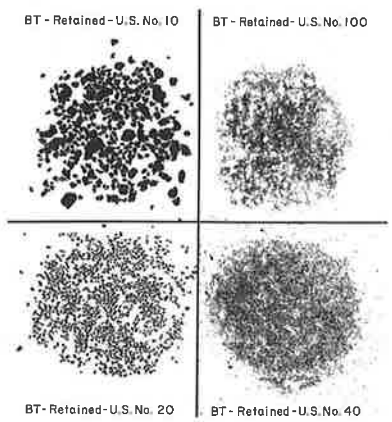
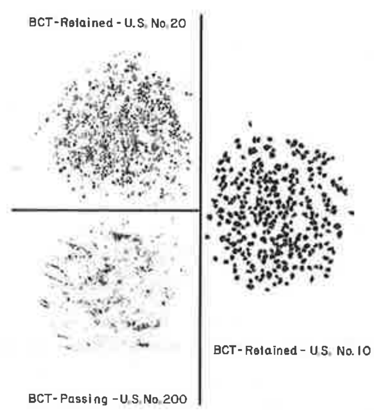


Figure 4. Sized particles of BCT.



Asphaltic cement utilized during the study was furnished by the Allied Material Corporation, Inc., of Stroud, Oklahoma. This material had a standard penetration of 85 to 100 at 77 F and specific gravity of 1.02.

USE OF SMELTER WASTE IN ASPHALTIC MIXTURES

Testing Procedures

The objective part of the study was to determine the feasibility of using zinc smelter waste as an aggregate in sand-asphalt mixtures. Hot-mix/hot-laid sand-asphalt mixtures are generally used in base course construction but can be used as a surface course for pavements carrying limited light loads.

Gradation requirements for sand-asphalt base courses (hot-mix/hot-laid) conforming to Section 708A of the Standard Specifications for Highway Construction of the Oklahoma Highway Commission are shown in Figure 5a. No type of smelter waste tested had a natural gradation within specification limits. Therefore, it was necessary to either crush or sort and recombine the smelter waste or add additional fines in the form of SS (fine sand).

In the process of combining the tailings and SS, a maximum amount of smelter waste was used, and a midpoint gradation was not achieved. Table 2 gives the percentage used of each smelter waste and SS; Figure 5b shows the resulting gradations of each combination of smelter waste and SS.

The Hveem gyratory method of mix design was used during the study, and mix design procedures were followed using percentages of smelter waste and SS, then repeated using 100 percent smelter waste. Smelter wastes were both crushed and sorted and recombined to yield the same gradation produced by the addition of fine sand.

Evaluation of Test Results

A summary of the results of laboratory mix designs for all test mixes (smelter waste-fine sand and 100 percent smelter waste) is given in Table 3. Figure 6 shows the relation between Hveem stability and asphalt content for the mixes, as computed by a total mix weight basis. Figure 7 shows relations between asphalt content and percentage of total voids. Figure 8 shows obtained relations between asphalt content and compacted unit weight of the mixes.

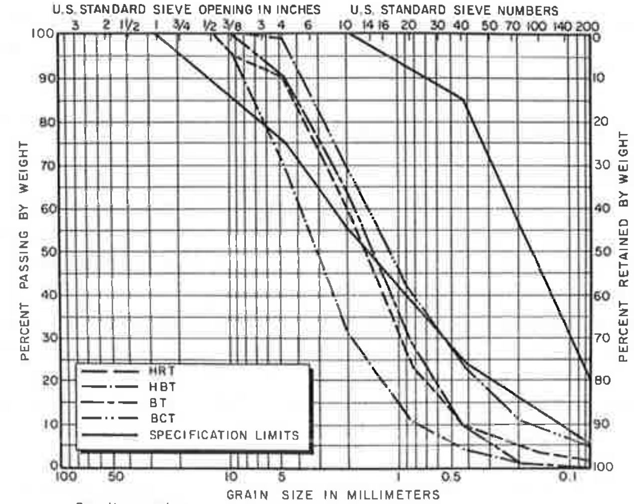
Figure 6 shows a very interesting phenomenon. A small difference of only 3 to 5 percent in Hveem stability can be observed for an increase of 1 percent asphalt content. This is somewhat unusual because a percentage difference in Hveem stability of as much as 10 percent would be expected with conventional aggregates. Meredith (2) reports a lack of response to density, stability, and flow in shell-asphalt mix designs as determined by using the Marshall method. Because of this problem, much emphasis is placed on the percentage of voids for the total mix. Earle (14) also mentions problems in determining asphalt contents using the Marshall tests with blast furnace slag. Engineering judgment coupled with a proven field mix is generally used in slag-asphalt mix designs in Great Britain.

As may be seen in Figure 6, the largest differences in stability among the various smelter wastes were obtained in mixes with larger percentages of SS. The stability values occurring at the optimum asphalt content for smelter waste/sand-asphalt and 100 percent smelter waste-asphalt exceed the minimum 20 percent stability required by the Oklahoma Department of Highways specifications for sand-asphalt mixtures, as obtained values ranged from 25.5 to 43.7 percent (Table 3). Use of either 100 percent smelter waste or smelter waste-sand did not greatly affect obtained stability for a given smelter waste.

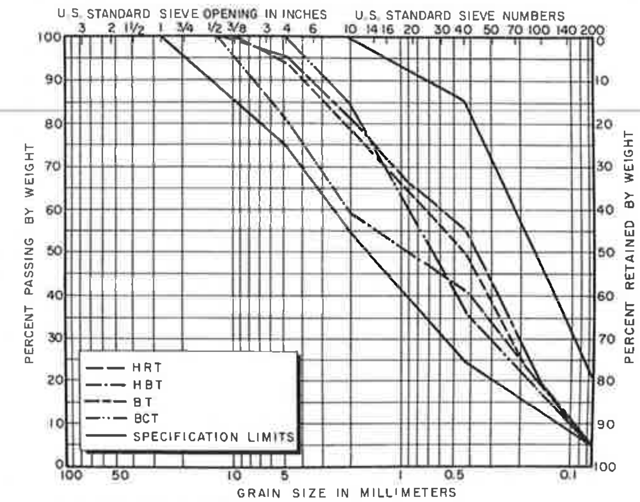
The unit weights of the compacted samples (110 to 130 pcf for all except BT) tend to be lower than usual for conventional aggregates, primarily because of low zinc waste specific gravities and/or high aggregate angularity. Unit weights of approximately 140 lb/ft^3 can be expected from samples containing aggregate with a specific gravity of 2.65.

Optimum asphalt contents for the mixtures were within the allowable range of 3.0 to 8.0 percent required by the Oklahoma Highway Commission. For all samples at

Figure 5. Grain size distribution curves for smelter wastes with and without fine sand.



a. Smelter wastes.



b. Smelter wastes and sand.

Table 2. Percentages of smelter waste and Sapulpa sand used for sand-asphalt.

Smelter Waste	Smelter Waste Utilized (percent)	Sapulpa Sand Utilized (percent)
HRT	55	45
HBT	60	40
BT	50	50
BCT	80	20

Table 3. Hveem gyratory mix design results.

Test Material	Optimum Asphalt Content (percent)	Stability (percent)	Unit Weight (lb/ft^3)	Voids of Total Mix (percent)
60 percent HBT and 40 percent SS	8	34.5	128.7	11.0
100 percent HBT	8	39.5	126.0	6.8
55 percent HRT and 45 percent SS	7	26.4	122.0	14.7
100 percent HRT	7	25.5	114.3	19.0
80 percent BCT and 20 percent SS	4	43.7	117.9	14.8
100 percent BCT	4	43.4	114.8	16.6
50 percent BT and 50 percent SS	6	37.7	139.4	16.5
100 percent BT	6	43.0	158.7	11.0

Figure 6. Hveem stability versus asphalt content for zinc smelter waste mixes.

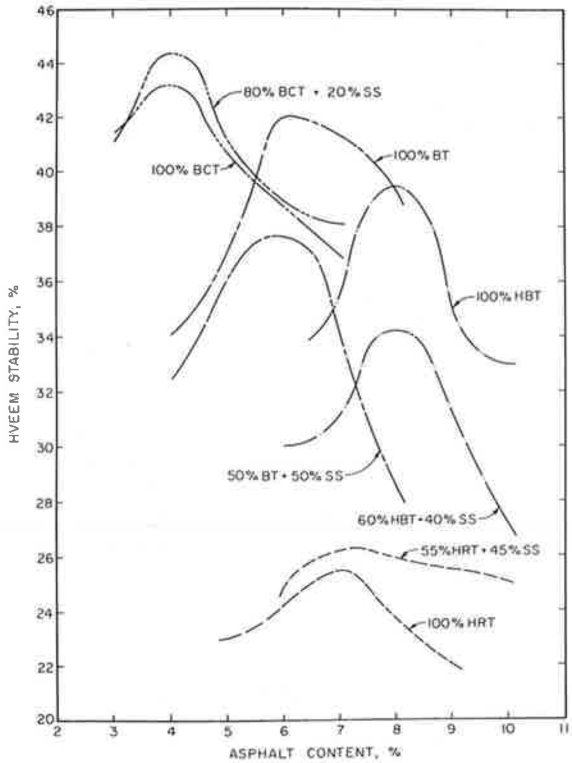


Figure 7. Relation between asphalt content and percentage of total voids for zinc smelter waste mixes.

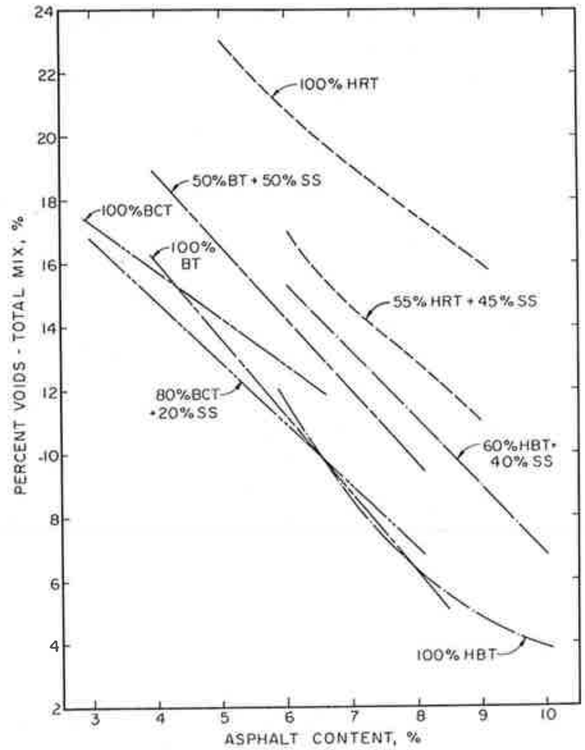
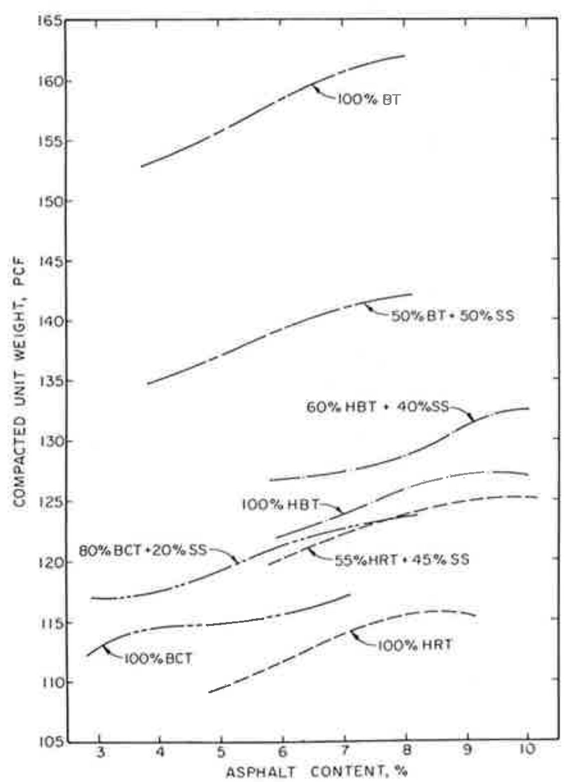


Figure 8. Compacted unit weight versus asphalt content for zinc smelter waste mixes.



optimum asphalt content, the compacted unit weight is less than 82 percent of the maximum theoretical density. The probable cause of this behavior was the high angle of internal friction of the zinc tailings. Direct shear tests on the four zinc smelter wastes produced friction angles Φ ranging from 47 to 54 deg; these very high values are attributed to the effects of interlock and angularity of the waste particles.

A current problem in Oklahoma is obtaining higher skid resistance of pavement surfacing, as many limestones and dolomites used in Oklahoma highway construction are susceptible to traffic polishing. The sharp angular texture of the four zinc smelter wastes indicated a potentially high skid resistance; thus the Oklahoma Department of Highways test for insoluble residue (OHD-L-25) was carried out on the smelter wastes. The test consists of soaking the aggregate in HCl until all reaction ceases and measuring the amount of treated sample retained on a No. 200 sieve. Aggregate used in the wearing course must contain at least 30 percent insoluble residue. All four zinc smelter wastes exceeded test requirements: HBT contained 70.1 percent insoluble residue, HRT contained 88.7 percent, BT contained 68.3 percent, and BCT contained 79.9 percent.

Although additional detailed testing and evaluation are perhaps needed to form a final opinion, nevertheless the test results imply that zinc smelter wastes can be substituted for conventional aggregates in sand-asphalt mixtures, by adding fine sand or crushing and/or sieving and recombining the smelter waste, to produce required gradation. Also, the smelter waste appears feasible for use in surface courses where improvement of skid resistance is needed, when blended with coarser grained limestones or dolomites.

USE OF SMELTER WASTE IN PORTLAND CEMENT CONCRETE MIXTURES

The four zinc smelter wastes were also used as fine aggregate in trial PCC mix designs, but all mixes would not "set" quickly, and obtained strengths were very low. Some samples could be crushed by hand after 28 days of curing.

Retardation of setting resembled that described by Schaeffer and Peyton. Schaeffer (15), in 1932, described problems experienced in England with efflorescence and scaling of concrete from release of water-soluble substances by aggregate. Peyton (16) reported that weathering of sphalerite (ZnS) contained in chert aggregate produces zinc carbonate (smithstone), which causes severe retardation of set if present in amounts greater than 0.3 percent by weight of cement.

To verify this possible cause of behavior, the alkali reactivity of all four smelter wastes was determined (using ASTM Designation C 289), and all were found to be highly reactive. Thus, use of these zinc smelter wastes in PCC mixes is not recommended.

USE OF SMELTER WASTE IN STABILIZED AGGREGATE

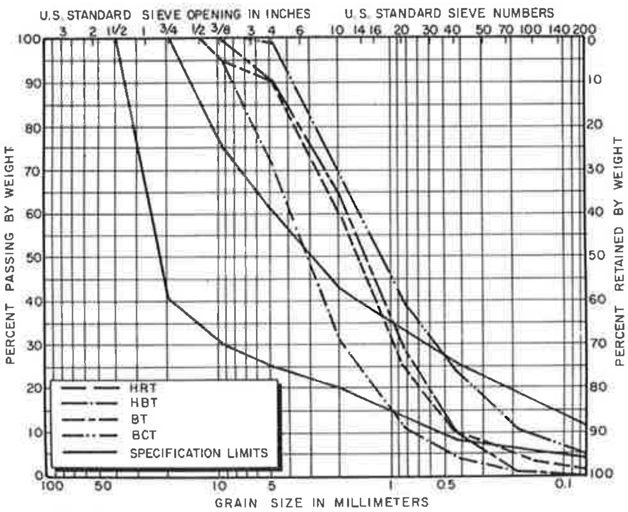
This phase of the investigation concerned the feasibility of using the zinc smelter wastes in the manufacture of mechanically stabilized aggregate mixtures for use as base material.

Stabilized aggregate base courses, as defined by Oklahoma Department of Highways specifications, should consist of blended coarse aggregate, sand, stone dust, or other inert finely divided mineral matter and a soil binder. At least 40 percent of the total mix retained on the No. 4 sieve should be uniformly graded. Material passing the No. 40 sieve is required to have a plasticity index of 6 or less and a liquid limit of 25 or less.

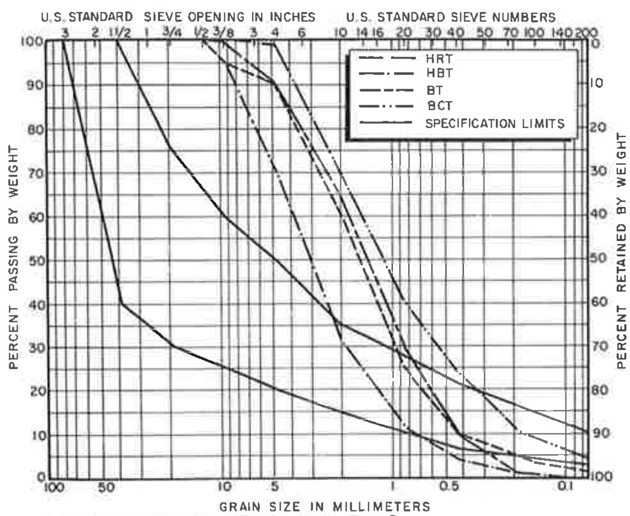
Stabilized aggregate is specified (by the Oklahoma Department of Highways) as either Type A or Type B, the difference being in acceptable gradation.

Figure 9a shows specification limits for Type A and grain size distributions for the four smelter wastes, while similar information for Type B stabilized aggregate is shown in Figure 9b. From Figure 9 it may be noted that all four types of smelter waste need additional coarse-grained material to meet requirements for either type of stabilized aggregate. However, it is a common procedure to blend aggregates and

Figure 9. Grain size distribution for zinc smelter waste compared to limits for stabilized aggregate.



a. Smelter waste and specification limits, Type A.



b. Smelter waste and specification limits, Type B.

produce the required gradation; thus the zinc smelter wastes appear feasible for consideration in manufacture of stabilized aggregate.

A lack of vegetative cover was observed near stockpiles of the tested smelter wastes. Soluble zinc leaches into the soil and may kill or damage vegetation. This may or may not be desirable when the tailings are used in stabilized aggregate but may not be of major importance when used under the center section of a road with wide surfaced shoulders. Conversely, this property may be an advantage if the smelter wastes are used in aggregate bases of low-traffic rural roads or under improved shoulders because they would retard growth of vegetation through the pavement structure.

It is therefore concluded that zinc smelter wastes may be an excellent potential source of fine aggregate for use in stabilized aggregate base courses.

CONCLUSIONS

Results of this study, although tentative, indicate that zinc smelter wastes may be used as aggregate in particular phases of highway construction. The following conclusions are made:

1. When natural gradation is modified by addition of fine sand or the material is either separated or crushed and recombined to meet particular gradation requirements, it may be used in sand-asphalt mixes. Resulting stability values are above minimums of the Oklahoma Department of Highways, and optimum asphalt contents are within specification limits.
2. The Hveem gyratory method appears adequate for mix design. However, the sharp angular texture of the aggregate, with resulting high angles of internal friction, causes unusual Hveem stability values.
3. Addition of zinc smelter waste to asphaltic concrete surface course mixes containing limestone aggregate susceptible to polishing should increase mix skid resistance.
4. Because of cement-aggregate reactivity, zinc smelter wastes should not be used in PCC mixtures.
5. All four of the zinc smelter wastes appear satisfactory for use in manufacture of mechanically stabilized aggregate mixtures.

It should be noted that three (HRT, HBT, and BCT) of the four smelter wastes are available at zero material cost. Although transportation costs may prevent economically feasible use of smelter wastes at some distance from their locations, this type of material should be especially attractive to small municipal and county road-building agencies that are forced to operate on extremely limited budgets. Further, use of these wastes would help to reduce the pollution problems that they present in their current condition and location.

RECOMMENDATIONS FOR FURTHER RESEARCH

Further research should be concerned with determining skid resistance of asphalt-zinc smelter waste mixtures; it should be quite high. Consideration should also be given to stripping, degradation, weathering, and validity of mix design procedures. A relation between field experience and laboratory mix design should be established. In addition, other types of zinc smelter waste (of which several exist in Oklahoma alone) should be investigated using the procedures described here. The wide variation in physical properties of the four samples used in this study indicates that generalization of results should not be attempted.

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UTILIZATION OF ASH FROM COAL-BURNING POWER PLANTS IN HIGHWAY CONSTRUCTION

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Studies conducted at West Virginia University have shown that the engineering properties of bottom ash are comparable to those of conventional construction materials. Nevertheless, bottom ash and fly ash have not been widely used in highway construction in the United States. During the past 3 years, considerable experience has been gained in West Virginia and the surrounding states in the utilization of coal ash in highway construction. The applications have included use of bottom ash or fly ash or both in nonstabilized base courses, portland cement and bituminous stabilized base courses, bituminous surface courses, lightweight structural fill, and underdrain filters. It was found that ash materials can be used satisfactorily in these applications providing their unique properties are recognized and appropriate construction techniques are employed. It is shown that these materials do not always fit within the framework of existing materials and construction specifications and that adherence to these specifications does not always ensure satisfactory performance. In fact, in some instances better performance and greater economy were achieved with ash mixtures that did not meet existing specifications. If ash is to be used effectively in highway construction, new specifications must be developed that take into consideration the unusual properties of ash and the specialized construction techniques required.

•IN recent years, many areas of the United States have been faced with a growing and potentially serious shortage (11) of suitable natural aggregates for use in highway construction. In addition, the necessary handling and processing operations have resulted in a dramatic increase in the cost of natural aggregates. At the same time, the production of coal ash (bottom ash and fly ash) by coal-burning electric utilities in the United States has continued to increase, while concern over protection of the environment has made satisfactory disposal of the ash increasingly more burdensome. For example, in the 5-year period from 1966 to 1971, the annual production (1, 4) of fly ash and bottom ash has increased from 25.2 to 42.2 million tons, while utilization (1, 4) increased from 3.1 to 8.6 million tons. Although, on a percentage basis, the increase in ash utilization (177 percent) greatly exceeded the increase in production (67.5 percent), utilization still lagged production substantially, e.g., by 33.6 million tons in 1971. Thus, there are ever-growing stockpiles of ash in many parts of the United States where the need for materials for highway construction is currently acute or will become so in the foreseeable future. It, therefore, seems logical to direct our attention toward increased utilization of fly ash and bottom ash in highway construction.

There is considerable information available on the properties of fly ash and its applications (2, 3), and there is a substantial body of information that indicates that fly ash can be used successfully in highway construction, notably in embankments (6, 8), portland cement concrete (2, 3), and various pavement elements (9). In contrast, relatively little appears in the literature about the properties of bottom ash. However, the results of a recent study (7) conducted at West Virginia University have indicated that the engineering properties of many bottom ashes compare very favorably with those of conventional highway construction materials. Nevertheless, fly ash and bottom ash are not widely used in highway construction in the United States. In the case of bottom ash (boiler slag), this is primarily because of the lack of laboratory and field data on the properties, construction handling, and performance of this material. A similar reason cannot be cited for lack of utilization of fly ash because more than 10 years of laboratory and field experiences have been reported by the British in various publications (2, 6, 8, 9, 10). In the authors' view, reasons other than those mentioned are largely responsible for the current state of practice in the United States. Among the more important reasons are (a) reliance on somewhat rigid material and construction specifications to exclude utilization of coal ash materials rather than consider their acceptance upon proof of equal or better performance; (b) lack of consideration of the unique properties of coal ash materials, i.e., applying criteria developed for conventional highway aggregates to materials inherently different; (c) lack of foresight in developing the necessary techniques to effectively utilize these materials prior to an acute shortage of conventional aggregates; and (d) hesitancy of organizations to support research that may profit a single industry.

During the past 3 years, fly ash and bottom ash have been used, either singly, in combination with each other, or with other materials, in a variety of highway and highway-related applications in West Virginia and the surrounding states. These applications have included the use of ash in nonstabilized and stabilized base courses, paving mixtures, lightweight structural fills, the grouting of landfills, and filters for underdrain systems. A considerable amount of experience on the utilization of ash in highway construction has been accumulated as a result of these applications. It is the purpose of this paper to share some of that experience.

BASE COURSE APPLICATIONS

Nonstabilized Bases

In the authors' experience, one of the first attempts to utilize nonstabilized bottom ash in base courses, while satisfying standard highway specifications, was in the 1971 construction of the access road to the Law School-Computer Center complex on West Virginia University's Evansdale campus. Bottom ash produced by the Fort Martin Station of the Allegheny Power System was used as it came from the ash hopper without screening or additional treatment. It was found that the material in this condition would pass the specified gradation, abrasion, and sulfate soundness requirements of the West Virginia Department of Highways for class 2 base courses as given in Table 1.

The bottom ash was placed with a conventional spreader box and was compacted using a 10-ton tandem steel-wheeled roller. It was found that the bottom ash could be spread and compacted very well when placed at the optimum moisture content, or slightly above, as determined by the standard Proctor compaction procedure. In fact, the densities achieved generally equaled or exceeded the required 95 percent of the laboratory maximum dry density, which was 85.0 lb/ft^3. However, it was found that the bottom ash lost stability when it dried out, and it was necessary to keep the material wet in order that paving and other construction equipment could be operated satisfactorily on its surface. The relatively low compacted density is attributable in part to the low specific gravity of the ash (2.32), but it is felt that the angularity of the ash particles and their porous surface texture (7) may be contributing factors. The behavior of this ash upon drying is characteristic of a uniformly graded material, even though it would classify before compaction as a well-graded material by the Unified Classification System. It is suspected that degradation during compaction may have played an important role in this phenomenon. This cannot be confirmed, however, because the material was not sampled after compaction. In any event, the confinement provided by the placement of the overlying bituminous concrete base and surface courses resolved the problem, and no further difficulty was experienced.

Similar behavior was observed in the utilization of untreated bottom ash in base courses for shoulders and lightly traveled access roads constructed as a part of the relocation of West Virginia Route 2 in the Ohio Valley south of Wheeling. In this application, bottom ash from Ohio Power's Cardinal Plant at Brilliant, Ohio, was placed at an average moisture content of 14 percent and compacted with two passes of a 10-ton tandem steel-wheeled roller followed by four passes of a 30-ton pneumatic roller. This material also became unstable upon drying, even though it met gradation and quality requirements and had been compacted to densities in excess of 95 percent of the standard Proctor value.

In contrast to these two experiences, higher densities and excellent dry stability were achieved on another West Virginia Route 2 base course application where a mixture of bottom ash and blast furnace slag was used. Bottom ash from American Electric Power Company's Mitchell Plant was blended with blast furnace slag, meeting ASTM No. 467 grading, in order to satisfy the gradation requirements of the West Virginia Department of Highways for class 1 crushed-aggregate base course. Because of variations in gradation of the ash, the percentage of slag required to satisfy the Department's gradation requirements varied from 15 percent to 40 percent by weight of aggregate. A comparison between the requirements for class 1 crushed-aggregate base course and the properties of a typical ash-slag mixture is given in Table 2.

The mixture was placed and compacted in two lifts to a total thickness of 9 in. Although compaction water content varied, it was generally within the specified limits of 6 to 8 percent. Final compaction was obtained by means of 4 to 6 passes of a 30-ton pneumatic roller. Field measurements indicated that the compacted dry density generally exceeded the specified value of 95 percent of the laboratory maximum dry density, which was 105 lb/ft^3. This experience was encouraging because it proved that untreated ash could be used to construct a satisfactory base course when the proper gradation and combination of materials were employed. It also called attention to the need for further research into the basic properties of bottom ash by itself and in combination with other materials.

In an effort to find a solution to the problem of loss of stability upon drying, a laboratory study was conducted at West Virginia University using bottom ash and fly ash from the Fort Martin Station. It was found that the addition of fines in the form of fly ash provided the required binder and that good initial density and dry stability could be obtained. Although various combinations of the materials were studied, it was found that best results were obtained in the laboratory with a mixture of 70 percent bottom ash and 30 percent fly ash. However, the unique properties of coal ash again manifested themselves when it was found that the greatest initial stability was produced at a moisture content several percent below optimum.

Fortunately the authors had an opportunity to follow this study into the field when the engineers for the Allegheny Power System elected to use a mixture of bottom ash and fly ash as the base course for the reconstruction of the access roads to its Fort Martin Station. Although the roads do not carry a large volume of traffic, many of the vehicles are trucks carrying ash and weighing in excess of 30 tons. Because the engineers were not constrained by material and construction specifications, they chose to experimentally determine the relative proportions of bottom ash and fly ash to blend to obtain a well-graded mix with good compactness. Based on laboratory studies, independent of the West Virginia University tests, they also selected a mixture of 70 percent bottom ash and 30 percent fly ash.

The materials were placed in trench-like excavations approximately 6 to 8 ft in width and varying from 2 to 5 ft in depth depending on the topography of the particular location. Sections at essentially natural grade were excavated approximately 28 in. through an existing roadway and the natural subgrade. Deeper sections were used to replace existing side-hill fills. Initially, a drainage layer consisting of 7 in. of compacted bottom ash was placed. This was followed by successive lifts of a bottom ash-fly ash mixture. Loose lift thicknesses of approximately 12 in. were utilized for the bottom ash-fly ash mixture. The mixed materials were dumped from trucks and spread to the desired loose lift thickness by a small bulldozer.

Initially, the 70-30 bottom ash-fly ash combination was tried, but difficulty was encountered because of excessive moisture and an accompanying loss in stability during compaction. A 60-40 combination was then tried and proved to be a satisfactory blend for the working conditions encountered. Combination of the materials was accomplished with a front-end loader simply by alternately dumping and mixing bottom ash and fly ash in volumetric proportions estimated to achieve the desired combination. Laboratory grain size distribution tests, performed on field samples of the 60-40 and 70-30 ash mixtures, showed that the gradation curves for both mixtures fell within their expected tolerance bands. At the time of the mixing the fly ash was essentially dry, but the bottom ash had water draining from it. Generally, the stockpiled mixture was then left for varying periods of time during which some additional drainage of water took place. During the early stages of the work, bottom ash was taken directly from the decantation tank and as a result contained excessive moisture to the extent that water drained from it as it was carried by trucks to the construction site. Later, the bottom ash was stockpiled prior to use. This permitted drainage of the excessive water and resulted in better compaction. Compaction was first attempted with a three-wheeled, steel-wheeled roller that proved to be unsatisfactory. Loss of stability beneath the roller on slight grades made the use of this roller impractical, and it was replaced with a vibratory roller having rubber-tired rear driving wheels and a steel-wheeled front roller (Rayco Model 400RT-1). This roller gave good performance both on the bottom ash alone and on the bottom ash-fly ash mixture. Visual observation and the specification of minimum roller coverages were used for compaction control. Generally, 6 to 10 passes of the vibratory roller were sufficient to produce a stable, well-compacted bottom ash layer.

As many as 20 passes of the vibratory roller were made to produce a visually stable and compact layer of the bottom ash-fly ash mixture. However, field density measurements indicated that only modest densification was achieved for the last 10 passes; e.g., dry densities of approximately 100 and 103 lb/ft^3 were achieved for 10 and 20 roller passes respectively. Furthermore, it was observed that lower dry densities were obtained, for 20 roller passes, for each succeeding lift. For example, at one location compacted dry densities of 103.1, 100.9, and 99.9 lb/ft^3 were achieved for the first, second, and third compacted layers respectively. The increasing relative compressibility of the ash mixture probably accounts for this observation. Some slight evidence of instability was noted in the field during compaction, becoming more noticeable with increasing thickness of the ash mixture.

Field moisture contents varied from 16.2 to 19.3 percent, having an average of 18.1 percent for eight measurements. This moisture content was considerably in excess of the standard Proctor optimum moisture content of approximately 10 percent. However, the dry densities achieved in the field (93.6 to 103.1 lb/ft^3) ranged from 96.0 to 105.7 percent of standard Proctor maximum density (97.5 lb/ft^3). All of these measurements were made on the 60-40 bottom ash-fly ash mixture. A field moisture-density test conducted on a 70-30 ash mixture in place for 2 months indicated a dry density of 101.6 lb/ft^3 at a moisture content of 12.4 percent. This layer had no cover and had been exposed to traffic and the weather during the 2-month period.

The exceptionally high densities achieved for the "wet-of-optimum" moisture conditions are somewhat surprising. However, the type and magnitude of field compaction effort as compared to that used in the standard laboratory compaction test may suggest a partial explanation. In addition, although most fly ashes alone tend to exhibit a marked decrease in strength when compacted wet of optimum (5), the loss in strength of the fly ash-bottom ash mixture compacted wet of optimum is gradual. This behavior was observed while conducting laboratory cone penetrometer tests on compacted samples of a 60-40 bottom ash-fly ash mixture. Although the results are inconclusive, the strength of the mixture was a maximum when compacted slightly dry of optimum and decreased gradually with increasing compaction moisture content. Although the percentage of decrease in strength was as large as 100 percent, samples at high moisture contents (6 percent above optimum) still exhibited good strength. It is apparent that the presence of the granular bottom ash tends to limit the strength loss.

Current standard highway specifications for base course materials attempt to control the quality and hence the performance of the materials by specifying acceptable limits for gradation, soundness, abrasion, percentage of fines, and Atterberg limits of fines. Although many bottom ashes can satisfy soundness, abrasion, and percentage of fines requirements, they may not meet gradation requirements. The experience cited indicates that other materials can be blended with bottom ash to overcome the gradation deficiency. However, within the framework of existing specifications, mixtures of ash containing percentages of fly ash (fines) greater than those specified for base course materials would be unacceptable. It should be pointed out that the fines, in this case, are not only nonplastic but are actually cementitious. Thus, it appears that, for untreated base courses, strict adherence to standard highway specifications in all instances is neither satisfactory nor reasonable.

Stabilized Bases

Portland Cement Stabilization—The first known large-scale application of a portland cement stabilized bottom ash base course in the United States was in the 1971-72 relocation and reconstruction of West Virginia Route 2 south of Wheeling. The aggregate for this project consisted of a blend of bottom ashes from American Electric Power Company's Kammer Plant and its nearby Mitchell Plant. This blend was necessary in order to meet the West Virginia gradation specification for class 5 cement-treated aggregate base course. Variations in the gradation of the two ashes, as produced, necessitated some adjustments in the relative proportion of each used in the mixture. A typical mix consisted of 46 percent (dry-weight basis) of Kammer ash and 54 percent of Mitchell ash. It was specified that the mix be stabilized by the addition of 5 percent portland cement by weight of dry aggregate. The optimum moisture content and the maximum dry density were determined by the standard Proctor procedure to be 8 percent and 114 lb/ft^3 respectively. The material was placed in one lift and compacted with a 30-ton pneumatic roller to a thickness of 6 in. In general, the field densities achieved equaled or exceeded the specified 97 percent of the standard Proctor value. In this application, it is believed that excellent results were achieved at a substantial reduction in cost as compared to the use of conventional aggregates.

In order to study further the potential use of portland cement stabilized ash base courses, a study of cement-treated mixtures of bottom ash and fly ash was undertaken at West Virginia University during the summer of 1972. This study was conducted in cooperation with the West Virginia Department of Highways, which provided laboratory personnel to perform the sampling and testing. The laboratory facilities, technical guidance, and supervision of the work were provided by the authors.

Bottom ash and fly ash from Allegheny Power System's Fort Martin Station were also used in this study. Because consideration was being given to the use of the material in the reconstruction of secondary roads, one objective of the work was to produce a mixture that would have a high initial stability in order to permit traffic to use the roadway prior to the placement of the surface course. As indicated earlier, it was found that high initial stability could be obtained with a mixture of 70 percent bottom ash and 30 percent fly ash compacted on the dry side of optimum. Therefore, this blend of ashes and a compaction moisture content of 12 percent were adopted.

In order to explore the variations that might be expected from materials produced at a single power station, five different samples of bottom ash at Fort Martin were obtained for use in the study. Sample C was obtained directly from the bottom ash decantation tank, whereas samples A, B, D, and E were obtained from different locations within the stockpile area. In addition, two types of fly ash were used. One of these was dry fly ash (H) taken directly from the ash hopper, and the other was stockpiled fly ash (S) that had been exposed to the weather for some time. The grain size characteristics of these materials are shown in Figure 1. The average specific gravity of the bottom ashes was 2.33, whereas those of the hopper and stockpiled fly ashes were 2.36 and 2.41 respectively.

The compaction procedure used to produce specimens for strength testing in this study was one that produced a higher energy input (20,666 $ft\text{-}lb/ft^3$) than the standard Proctor procedure. The applicability of this procedure was discovered when a number of specimens were accidentally prepared using this compactive effort. It was found that the moisture-density relations obtained by this procedure more nearly duplicated those being obtained by compaction equipment in the field than did the standard Proctor procedure.

Compacted specimens of the cement stabilized ash were stored in the moist room and tested in unconfined compression at 8, 30, and 60 days. For comparison purposes,

Table 1. Comparison of base course (class 2) and bottom ash.

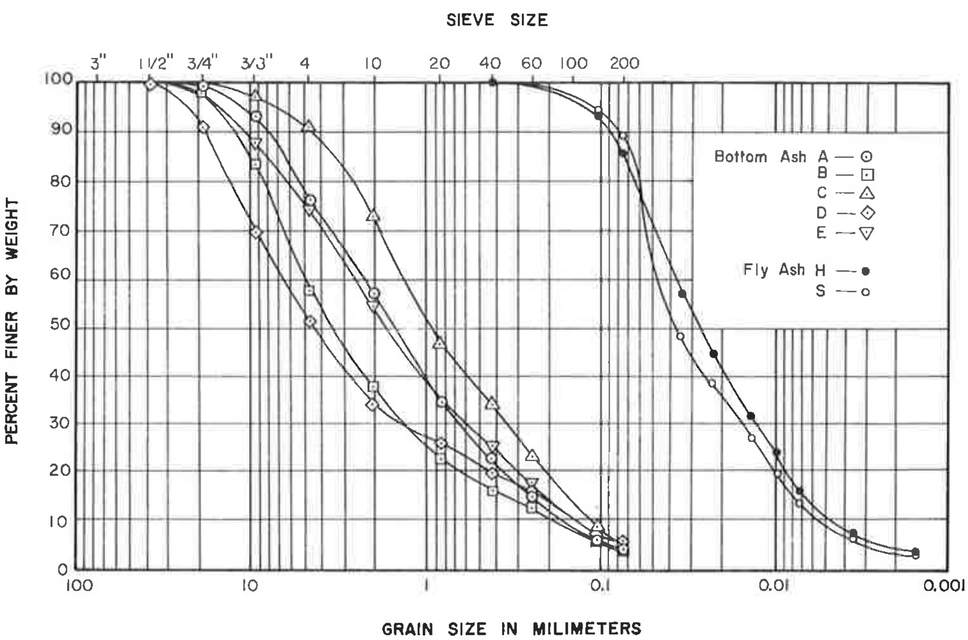
Material	Los Angeles Abrasion	Sodium Sulfate Soundness	Sieve (percent finer) 1½ In.	¾ In.	No. 4	No. 40	No. 200
Base course (class 2)	<50	<12	100	80 to 100	35 to 75	10 to 30	0 to 10
Bottom ash	27 to 40	4 to 8	100	97	70.3	23	4.5

Table 2. Comparison of base course (class 1) and bottom ash-slag mixture.

Material	Los Angeles Abrasion	Sodium Sulfate Soundness	Sieve (percent finer) 1½ In.	¾ In.	No. 4	No. 40	No. 200
Base course (class 1)	<50	<12	100	50 to 90	20 to 50	5 to 20	0 to 7
Bottom ash-slag mixture	37[a]	10[a]	100	78.6	40.6	13.1	2.5

[a]Maximum for Mitchell bottom ash only.

Figure 1. Grain size distribution of bottom ash and fly ash.



parallel series of specimens were prepared using two sources of limestone aggregate (PH and MA) commonly employed to construct cement stabilized base courses in north central West Virginia. The results of these tests are given in Table 3.

Although it is evident from Table 3 that the average strength of the cement stabilized limestone mixtures is greater than that of the cement stabilized ash mixtures, the difference is not large. In fact, several of the ash mixtures had greater 30- and 60-day strengths than did the PH limestone mix. The notable exception to this was the mixture prepared with bottom ash D and fly ash H (hopper ash). This mix gave every indication of having been prepared with reactive aggregates, in that the specimens displayed very noticeable evidence of expansion and cracking after having been cured for 8 days. It is felt that this was caused by the presence of pyrites in the bottom ash. One of the combustion units at the Fort Martin Station separates pyrites from the coal before it is burned, but unfortunately the pyrites are disposed of by dumping them into the bottom ash hopper. Although an attempt is made to segregate the stockpiles of ash from this unit, so that it will not be used where its reactive nature could cause problems, occasionally this ash is encountered, particularly in the older stockpiles. In any event, the combustion unit involved is being altered so that, in the near future, all of the ash collected at Fort Martin will be free of pyrite.

These experiences show that satisfactory portland cement-treated ash base courses can be produced using either bottom ash alone or an appropriate mixture of bottom ash and fly ash. Moreover, British experience has shown (9) that satisfactory base courses can be produced with cement-treated fly ash alone. However, it should be noted again that the use of the mixture of bottom ash (70 percent) and fly ash (30 percent), and the use of fly ash alone, in the preparation of cement-treated bases would not be permitted within the framework of most existing highway department materials and construction specifications in the United States.

Bituminous Stabilized Base—Bottom ash was allowed as an alternate base course aggregate in the repaving of some 40 to 45 miles of light-duty, rural secondary roads in West Virginia in the summer of 1972. The base course material was placed directly on the existing roadway in a single lift 2 to 6 in. deep. Most of the existing roadway was gravel or, at best, badly deteriorated chip and seal surface treatment. Existing ditches were cleared and regraded, and obvious soft spots in the roadway were replaced with granular material, but there were no other preparations prior to laying the base course. Ultimately, this material will receive a surface treatment; however, this will most likely be deferred until the 1973 construction season.

Both dry and wet bottom ash were used in various sections of the project. The wet bottom ash was furnished from the Kammer power plant, and the dry bottom ash was furnished from the Fort Martin power plant. As produced, the gradation and physical and engineering properties of dry and wet bottom ash are quite different (7). Gradation data for typical Kammer and Fort Martin ashes are given in Table 4. The wet bottom ash is of one size, predominately No. 4 to No. 16 mesh, and must be blended with other aggregates in order to meet the class 2 gradation requirements for base course material. The dry bottom ash more closely approximates the class 2 specification as given in Table 1.

The physical appearance of the two types of ash is quite different. The wet bottom ash is glassy and angular, resembling an angular crushed glass (7), and a very small percentage of the particles are spherical or string-like. Most of the particles evidence fractured faces, the result of the rapid quenching as the molten slag is dropped from the boiler into cooling water. Because of its glass-like surface texture and uniformity, wet bottom ash by itself possesses little internal stability and must generally be blended with other aggregates in order to produce acceptable bituminous mixtures.

In contrast, many dry bottom ashes produced in West Virginia give the appearance of a fine sand (7). Although generally lower in specific gravity than the wet bottom ashes, they are only slightly absorptive in nature. Saturated surface dry moisture contents are seldom more than a few percent, and it would appear that these materials are internally rather than externally porous. Many of the particles resemble wet bottom ash, i.e., black and glassy in appearance, but the predominant material is light in color with a sandpaper-like surface texture. In spite of the fact that the dry bottom ash is rather fine-graded (compared to class 2 base course, Table 1), it is, by itself, exceedingly stable, often giving Marshall stabilities in excess of 1,000 lb. This is attributed to the fact that it is well-graded and that the individual particles have a rough microtexture.

On those projects where wet bottom ash was used, it was necessary to blend the ash with other aggregate in order both to develop adequate stability and to meet the class 2 gradation specification (Table 4). Whenever possible a locally available aggregate, generally bank run gravel, was used. With 25 percent bank run gravel and 5 percent residual asphalt, Hveem stabilities for these mixtures ranged from 18 to 25. The mixes were "pugmilled" while cold at a central mixing plant and stockpiled for 10 days or more. The material was cold-laid with a paver or spreader box and, in some cases, end-dumped and leveled with a grader. Although several different compaction procedures were used, generally adequate compaction was achieved from several passes with a pneumatic roller followed by a steel-wheeled roller.

The dry bottom ash was used as produced by the power plant without any blending with other aggregate. The design asphalt content was approximately 7 percent, about 2 percent higher than that for the wet bottom mixes. At 7 percent asphalt content, the Hveem stability of these mixes was in excess of 40.

The lay-down characteristics of the dry bottom mixes were excellent either with a spreader box or with a conventional paving machine. Optimum densities were achieved with 3 to 4 passes from a pneumatic roller followed by one or two leveling passes with a steel-wheeled roller. Lifts up to 8 in. thick (uncompacted) were attempted with a spreader box with good results. With these thicker lifts it was necessary to work or track the mix with one or two passes of a grader before attempting initial compaction. Although not attempted, it is expected that lifts much thicker than 8 in. would be difficult to compact, the difficulty coming from instability during initial compaction. However, density checks with depth showed that good compaction was achieved throughout lifts up to 8 in. thick.

The Fort Martin dry bottom mixture was used as the top course in the shoulder construction on several miles of access road to the Fort Martin Station. The material was placed cold with a conventional shoulder or widening machine with very satisfactory results. The most satisfactory compaction was again achieved after three to five passes with a pneumatic roller.

The previously mentioned mixtures have been in service for less than a year, and it is too early to draw any meaningful conclusions as to their ultimate performance. In several instances these roads receive heavy loads from considerable coal truck traffic. Even under these heavy loads, with the exception of several base failures, there has been no appreciable rutting or shoving, and the performance to date is certainly encouraging.

BITUMINOUS PAVING MIXTURES

Not much engineering information has been published on the use of boiler slag as a major component in paving mixtures (2). As indicated previously, because of an inherent lack of stability, the wet bottom ash must generally be blended with other aggregates in order to produce a stable paving mixture. Typically these mixtures are sand asphalts and may be dense or open-graded. The use of boiler slag in wearing mixtures is permitted in the specifications of several states including West Virginia (in both standard and supplemental specifications), Indiana, and Ohio, and it has been used in cities such as Tampa, Columbus, and Cincinnati (7).

Wet bottom boiler slag has often been promoted as a premium aggregate for surface or deslicking mixtures. Any additional cost for this material is then justified on the basis of its hardness and angularity, desirable properties for a skid-resistant aggregate. Unfortunately, most wet bottom boiler slag is entirely lacking in aggregate microtexture, an undesirable property both in terms of skid resistance and in terms of the ability of the aggregate to retain its asphalt coating.

Wet bottom boiler slag has been used with some success in West Virginia in deslicking applications. A short section of US-119 near Morgantown was resurfaced with a thin deslicking overlay in 1969, as reported in an earlier paper (7). The dramatic reduction shown in the reported accident data before and after deslicking is evidence of the antiskid characteristic of this mixture. This mixture was a blend of river and limestone sand, fly ash, and Kammer wet bottom boiler slag meeting West Virginia wearing course 3 specifications as given in Table 5.

Considerable resurfacing has been done in the northern panhandle of West Virginia using a wearing course 3 mixture composed of approximately 50 percent wet bottom ash, 39 percent river sand, 3 percent fly ash, and 8 percent asphalt cement. The gradation of this mixture is also given in Table 5. The mixture is hot-mixed and hot-laid as a conventional sand mix in depths of ½ to 2 in. The mix is first broken down with a steel-wheeled roller, followed by several passes with a pneumatic roller. Various pavement sections have been in service for up to 6 years, and, although there has been some loss of surface aggregate, the rate of wear is not considered excessive. Performance under heavy truck traffic has been good with little or no tendency to rut or shove, and the surface texture of the pavement has changed little with service. It should be emphasized that, in this application, the boiler slag is considered to be an economical replacement for locally scarce natural aggregates and is not promoted as a premium skid-resistant aggregate.

The authors know of no reported use of dry bottom boiler ash in surface or wearing courses. The inherent stability of this material, along with an acceptable soundness and abrasion loss (Table 1), suggests that this material might be used as an acceptable surface mix for light or medium traffic. There is, however, a tendency for some of the more loosely agglomerated bottom ash particles to degrade under the action of heavy traffic. This has been observed with some of the Fort Martin base mixtures that have not received a surface treatment. In all fairness, these base mixtures were not designed as surface mixtures; as surface mixtures they would be considered deficient in asphalt and contain excessive voids (10 to 12 percent).

In summary, although wet bottom boiler slag is generally promoted as a premium skid-resistant aggregate, it should be emphasized that this is not the only application for this material. Experience in West Virginia has shown that it can successfully compete as an economic replacement for natural aggregates. Although little or no use has been made of dry bottom ash in wearing courses, its inherent stability suggests that it may be acceptable as a sand mix at a substantial savings in cost.

LIGHTWEIGHT STRUCTURAL FILL

As pointed out in the paper by Gray and Lin (5), much of the experience with the use of compacted fly ash in fills has been gained in Great Britain. Beginning with field trials in 1958, fly ash, or pulverized fuel ash (PFA) as designated by the British, has been used extensively and successfully in highway embankments and bridge abutment backfills (2, 6, 8). British experience has demonstrated that fly ash is a pozzolanic material because it self-hardens when compacted in a moist condition. It has been found that the pozzolanic activity is the greatest when moisture is added to fresh fly ash at its source. For effective compaction, loose lift thicknesses not exceeding 9 in. should be used and should be well "tracked" with a bulldozer prior to rolling. Compaction equipment giving the best performance includes tandem vibrating rollers with a dead weight of at least 1,700 lb, towed vibrating rollers with a dead weight of at least 3,000 lb, and self-propelled pneumatic rollers, 7 to 10 tons in weight, having a tire pressure of 30 to 36 psi. Generally, six to eight passes are required to meet the density specification. Experience has indicated that a minimum dry density of 90 percent of maximum British Standard (B. S. 1377:1967-Test No. 11) dry density should be specified. Smooth-wheeled (small, medium, or large) rollers, sheepsfoot rollers, grid rollers, and vibrating plates have not been successful in compacting fly ash.

The wet weight of compacted fly ash per cubic yard varies between 0.9 and 1.1 tons compared to approximately 1.4 for clay and 1.7 for sand. It is easily trenched; i. e., neat trenches can be excavated using a minimum of bracing. The inertness and alkalinity of fly ash make it generally harmless to essentially all types of embedded pipes.

Table 3. Results of unconfined compression tests on base course mixes.

Material	Unconfined Compression Strength (psi)[a]		
	8 Days	30 Days	60 Days
Bottom ash A and fly ash H	406	665	726
Bottom ash B and fly ash H	224	505	644
Bottom ash C and fly ash H	478	635	479
Bottom ash D and fly ash H	23[b]	112[b]	163[b]
Bottom ash E and fly ash H	487	512	653
Average	399	579	651
Bottom ash A and fly ash S	520	772	912
Bottom ash B and fly ash S	454	805	681
Bottom ash C and fly ash S	—	—	—
Bottom ash D and fly ash S	313	449	759
Bottom ash E and fly ash S	376	426	560
Average	416	613	728
Limestone PH	525	569	638
Limestone MA	616	961	906
Average	571	765	773

[a]Average of 3 tests. [b]Excluded from average.

Table 4. Comparison of base course (class 2) with wet bottom boiler slag and boiler slag-gravel mixtures.

Material	Los Angeles Abrasion	Sodium Sulfate Soundness	Sieve (percent passing)						
			1½ In.	¾ In.	⅜ In.	No. 4	No. 16	No. 40	No. 200
Base course (class 2)	<50	<12	100	80 to 100	—	35 to 75	—	10 to 30	0 to 10
Kammer (wet bottom)	26	2.5	100	100	99	96	16	3	0
75 percent Kammer and 25 percent gravel	—	—	100	99	92	85	40	11	1

Table 5. Comparison of wet bottom ash surface mixtures and wearing mixture 3.

Material Designation	Asphalt Content (percent)	Sieve (percent passing)					
		⅜ In.	No. 4	No. 8	No. 16	No. 50	No. 200
Specification	5 to 11	100	90 to 100	60 to 90	40 to 65	10 to 30	3 to 15
U. S. 119	7	100	95	85	48	16	6
Northern panhandle	8	100	95	80	52	14	6

However, certain precautions should be taken when utilizing fly ash as road and structural fill. Among these are the following:

1. Fly ash is a borderline frost-susceptible material. However, the use of adequate drainage and/or stabilization of the fly ash with lime or cement (5) is effective in eliminating or reducing frost effects.

2. Although, generally speaking, the sulfate content of most fly ashes is too low to be troublesome, it is possible that an exceptional fly ash may be encountered in which the sulfate content is sufficiently high to warrant some precautionary measures if it is to be used adjacent to concrete. In such rare cases, simply coating the contacting concrete with bituminous paint or rubberized componds should result in satisfactory protection.

In Great Britain, compacted fly ash is being used extensively as backfill for bridge retaining walls (8). Two favorable characteristics of the compacted fly ash are largely responsible for this practice, namely, the very low compressibility and the shear strength characteristics of the compacted fly ash. In both these instances the age-hardening characteristics of the fly ash are especially important. Not only will the settlement of the fly ash backfill be small, but the settlement of the foundation soil will be reduced because of the low unit weight of the fly ash as compared to conventional materials. Based on an active pressure analysis, the theoretical lateral pressure at the base of a typical wall may be negative or only slightly positive. In practice, however, significant positive lateral pressures can develop as a result of construction and design practices. Wilson and Pimley (10) reported positive lateral pressures in excess of those calculated on the basis of an equivalent fluid having a density of 14.8 lb/ft^3. They attributed their findings to the combined effects of a rigid wall and lateral pressures induced by the compaction process.

Fly ash was recently used as a lightweight structural fill in a landslide correction project on Route 250 near Fairmont, West Virginia. The project, implemented by a district maintenance force of the West Virginia Department of Highways, consisted of removal of the landslide debris, installation of an underdrainage system, placement of the fly ash fill, and sealing of the fill.

Initially, 6-in. perforated concrete pipes were placed in trenches at the base of the excavation to form an underdrainage system. The main collection pipe was placed immediately adjacent to the excavated slope and parallel to the roadway centerline. Three additional pipes were placed perpendicular to the roadway centerline and connected to the main collection pipe. All pipes were surrounded with approximately 3 in. of $\frac{1}{2}$-in. graded stone. An 18-in. thick blanket of 2-in. graded stone was then placed over the entire base area. Additionally, in the process of filling, an 18-in. thick layer of the same stone was placed between the fly ash fill and excavated slope to the bottom of the road subbase.

Approximately 5,000 tons of fly ash were utilized in the fill having $1\frac{1}{2}$:1 side slopes and an average height of 25 ft. The ash was hauled to the job in open trucks from its source, which was the Fort Martin Station of the Allegheny Power System. Water was added to the ash by spray nozzles as it left the storage hopper. For ease in dumping at the construction site, a layer of dry ash was placed in the truck before the wet ash. After the ash was tailgated from the trucks and additional water added, when required, it was spread by a road grader to an 8-in. lift thickness. Before effective compaction could be accomplished, the fly ash was "tracked" by using several coverages of the road grader or a bulldozer. Following the tracking operation, the fly ash was compacted to the specified density with a 10- to 12-ton pneumatic roller.

Based on standard Proctor compaction, the ash utilized in this fill had a maximum dry density of 92 lb/ft^3 and an optimum moisture content of 19 percent. For field control, a density of 95 percent of standard Proctor maximum dry density and a moisture content of 18 percent were specified. Field density test results indicated that densities ranging from 91 to 99 (average of 97) percent of standard Proctor maximum dry density and moisture contents ranging from 15 to 19 (average 16) percent were obtained. Approximately six to eight passes of the roller were required to obtain the specified density. On completion, the top of the fill and a portion of the slope were sealed by hand spraying with a coat of road tar (RT-12).

It is apparent from the preceding discussion that the acceptance of fly ash as an embankment material for highway construction would require changes in existing material and construction specifications. However, the advantages associated with its use, in terms of cost and performance, would suggest that such modifications of existing specifications might very well be warranted.

UNDERDRAIN APPLICATIONS

The authors are personally acquainted with three engineering projects utilizing bottom ash as an underdrain filter material. Although these projects were not strictly highway-related, the similarity to typical highway-related installations is apparent. Boiler slag (bottom ash) from American Electric Power Company's Kammer Plant was utilized to construct an underdrain system for a landslide correction project at the site of the McElroy Mine coal preparation plant near Moundsville, West Virginia. The underdrain system consisted of a trench in which 8-in. perforated asphalt-coated metal pipe was surrounded by river gravel for approximately 6 in. and then filled with boiler slag compacted into place. A concrete paved ditch was placed over the drain to collect surface water. The bottom ash was selected in lieu of river sand because it satisfied applicable filter criteria, was low in cost, and was readily available. This drain was installed in 1968 and has functioned satisfactorily ever since.

A multiple-purpose dam is being constructed on Charles Fork near Spencer, West Virginia, in which 1,800 yd^3 of boiler slag from the Willow Island Plant of the Monongahela Power Company is being utilized in the blanket and toe drain. The boiler slag met the filter criteria specified by the Soil Conservation Service and was selected on the basis of its lower cost as compared to available natural aggregates. Tests verified that the boiler slag did not contain sufficient quantities of soluble solids (204 to 240 mg/l) to be detrimental to the performance of the filter or affect the durability of concrete. A distilled water-boiler slag mixture gave pH readings ranging from 6.7 to 7.0 after 4 days. The boiler slag had a coefficient of permeability of 2.5×10^{-2}cm/sec at a void ratio of 0.77 (7). The boiler slag was placed by tracking with a small bulldozer while the slag was continually sprayed with water. After some field experimentation, the bulldozer, equipped with a front-end loader, was able to place the boiler slag in the filter zone on the relatively steel abutment slopes while operating over the compacted slag. Abutment slopes varied from 5:1 to 2:1.

Bottom ash from the Fort Martin Plant was recently utilized as fill behind a retaining wall and beneath floor slabs for a Holiday Inn addition near Morgantown, West Virginia. Because the bottom ash is free draining, it also serves as a filter drain behind the retaining wall. Perforated plastic pipe surrounded by gravel carries the water collected by the bottom ash backfill. Small tamping compactors were used to compact the bottom ash in 6- to 9-in. compacted lifts.

Experience with bottom ash thus far suggests that it is quite acceptable as an underdrain material, providing it meets the gradation requirements for a filter. In addition, the variability in gradation of the ashes studied (7) is generally equivalent to that specified for aggregates by ASTM. Therefore, the uniformity of the ash from load to load poses no problem. The permeability of the ash is good, being equivalent to that of a clean sand. The environmental effects of utilizing bottom ash in underdrains are negligible, with both respect to the drain itself and the surrounding appurtenances. Placement of the ash can be accomplished by conventional methods as currently used for clean granular materials.

APPLICABILITY OF EXISTING MATERIAL AND CONSTRUCTION SPECIFICATIONS

The experience that has been accumulated to date on the utilization of bottom ash (boiler slag) and fly ash in highway construction has raised serious doubts as to the applicability of existing materials and construction specifications. Most specifications have been developed after many years of laboratory and field experience with available natural soils or aggregates. Accordingly, when the same natural soils or aggregates are used in accordance with these specifications, adequate performance is usually ensured. However, the unique properties of bottom ash and fly ash lead to the possibility that an unsatisfactory result can be obtained even though existing specifications are met. On the other hand, it is possible that excellent results can be achieved with ash at a substantial savings in cost when gradation or other existing specification requirements are not met. Illustrations of both of these occurrences are included in the applications discussed in the preceding sections of this paper. Furthermore, it is possible that much of the savings that can be effected by the use of ash can be absorbed by the increased cost of screening and blending to satisfy existing gradation requirements, which may have little or no effect on the performance of the ash being used.

There is a very definite need to recognize that bottom ash and fly ash have unique chemical and physical properties and to consider these materials apart from conventional highway construction materials. If ash is to be used effectively in highway construction, new specifications must be developed that take into consideration the unusual properties of the ash and the specialized construction techniques required to ensure adequate performance. Assuredly, this will require additional study, both in the laboratory and in the field. However, interim specifications based on performance would open the way to expanded utilization of ash in highway construction and provide a fund of data on which more detailed material and construction specifications could be based.

SUMMARY

Although more laboratory and field research will be required in order to provide a sufficient store of knowledge to justify generalized acceptance of bottom ash and fly ash as highway construction materials, the experience described herein shows that much of the ash produced by coal-burning power plants can be utilized in one form or another in highway construction. As ash production increases and supplies of natural aggregates diminish in the years to come, it will become increasingly more desirable to utilize bottom ash and fly ash in a more productive manner. This will require the modification of existing materials and construction specifications to permit the use of ash in a variety of highway applications, with the provision that the resulting performance be equal to, or better than, that obtained with conventional construction materials. If this can be done, it will constitute a significant contribution to the conversion of a burdensome "solid waste" into a valuable national resource.

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