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Copper Mill Tailings, Incinerator Residue, Low-Quality Aggregate Characteristics, and Energy Savings in Construction

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page 16, column 2, line 15 from bottom and lines 10-11 from bottom
Change "19.93 kN" to "2032 kg"
page 16, column 2, lines 5-6 from bottom
Change "19.88 kN" to "2028 kg"
page 18, column 1, lines 6 and 12
Change "10.05-kN" to "1025-kg" and "1.913-kN" to "195-kg"
page 18, column 1, line 20, and column 2, lines 20 and 23
Change "62.27-kN" to "6350-kg"
page 26, column 1, lines 13-15
Change each "nt·s" to "N·s" and each "lb-s" to "lbf·s"
page 26, column 2, lines 4-6
Change each "nt" to "N" and each "lb" to "lbf"
page 27, column 1, lines 1-2 and following table
Change to "production lot (1 kN = 225 lbf):

Test Piece	Axial Load (kN)
CT 7-11-1	136.3
CT 7-11-2	115.6
CT 7-11-3	116.1
CT 7-12-3	119.7
CT 7-12-4	122.1
Average	121.9"

page 27, column 1, lines 9-11 from bottom and following table
Change to "table (1 kN = 225 lbf):

Test Piece	Shear Load (kN/coupling)
CT 7-11-4	28.0
CT 7-11-5	17.3
CT 7-11-6	23.6
CT 7-12-1	17.3
CT 7-12-2	19.6
Average	21.2"

page 27, column 2, line 13
Change "227 kg (500 lb) and 4536 kg (10 000 lb)" to "2.2 kN (500 lbf) and 44.5 kN (10 000 lbf)"
page 28, column 1, lines 3, 14-15, 18, and 22
Change each "nt·s" to N·s" and each "lb·s" to "lbf·s"
page 29, Abstract, line 15
Change "362 kg·s" to "3.6 kN·s"
page 30, column 2, line 21
Change "1145, 1105, and 1060 kg·s" to "11.2, 10.8, and 10.4 kN·s"
page 30, column 2, line 17 from bottom
Change "492, 487, 500, and 464 kg·s" to "4.93, 4.89, 5.02, and 4.65 kN·s"
page 31, column 1, line 11
Change "350, 360, 350, and 357 kg·s" to "3.43, 3.53, 3.43, and 3.50 kN·s"
page 31, column 1, line 39
Change "338, 349, and 388 kg·s" to "3.31, 3.43, and 3.80 kN·s"
page 31, column 2, line 35
Change "91 kg·s" to "0.89 kN·s"
page 31, column 2, line 44
Change "Tunnel momentum change, kg·s 504 351 338 452" to "Tunnel momentum change, kN·s 4.93 3.43 3.31 4.43"
page 31, Table 3
Change the momentum change values from kg·s to kN·s for each category: "Speed Trap Measurement: NM, 4.75, 3.51,–,4.96"; "Integration of Tunnel Acceleration: 11.17, 4.93, 3.42, 3.31, 4.50"; "Integration of Rear-Deck Acceleration: 10.8, 4.89, 3.53, 3.43, 4.10"; "High-Speed Film Analysis: 10.4, 4.89, 3.43, 3.80, 4.48"
page 32, column 1, lines 7 and 10
Change "91 kg·s" to "0.89 kN·s" and "457, 418, 457, and 506 kg·s" to "4.43, 4.11, 4.25, and 4.97 kN·s"
page 33, column 1, lines 7-8
Change "500 kg·s" to "4.89 kN·s" and "350 kg·s" to "3.34 kN·s"
page 33, column 1, text table
Change the momentum change values from kg·s to kN·s for each test: "Test 1:–, 11.22, 10.83, 10.39"; "Test 2: 4.75, 4.94, 4.89, 5.02"; "Test 3: 3.51, 3.44, 3.53, 3.43"; Test 4:–, 3.31, 3.42, 3.80"; "Test 5: 4.96, 4.48, 4.10, 4.48"
page 33, column 1, line 31
Change "350 kg·s" to "3.34 kN·s"
page 33, column 2, lines 5 and 7
Change "500 kg·s" to "4.89 kN·s" and "91 kg·s" to "0.89 kN·s"

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page 19, column 2, line 22
Change "frequently" to "infrequently"

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page ii, Library of Congress data
Change "[666'.89]" to "[66';893]"

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page ii, Library of Congress data
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Contents

Stabilized Copper Mill Tailings for Highway Construction

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The results of an investigation to determine the feasibility of using stabilized copper mill tailings in road construction are presented. Properties of three types of tailings were evaluated after preliminary testing of several tailings from Arizona, Idaho, and Utah. The properties of the three tailings are summarized, and detailed results obtained for one type are reported. Index properties of the untreated tailings, including physical and mechanical properties, are given. Engineering parameters of untreated tailings are reported, including compaction characteristics; compressive, tensile, and shear strength; compressibility; permeability; and erodibility by rainfall. Properties of cement-stabilized and asphalt-stabilized tailings are also presented. The results demonstrate that copper mill tailings have excellent engineering properties and can be successfully used in road construction. In particular, the results indicate that there is excellent potential for using tailings as compacted fill in embankments, compacted foundation and subgrade material, cement-treated base, emulsion-treated base, and stabilized material for lining canals, ponds, and reservoirs.

In normal copper-mining operations, the mined ore is crushed in gyratory primary crushers that reduce it to an average 15 cm (6 in) in size. The crushed ore is subjected to secondary and tertiary hydrocone crushers that reduce the final product to a size smaller than 1.27 cm (0.5 in). This fine ore is then wet-ground in ball mills. The ground ore, in an ore-water slurry and conditioned with reagents, is floated in flotation tanks to a rough concentrate of copper and other valuable minerals. Tailings from the flotation process are thickened, and the excess water is recovered from the slurry for reuse in the process. The slurry is either pumped into tailing-disposal ponds or classified for use as a hydraulic backfill in the mine.

More than 1 billion Mg (1.1 billion tons) of mine and smelter waste, including tailings, are produced every year by the basic mineral industry. A 1965 land survey by the U.S. Department of Agriculture showed that the United States contained 0.8 billion hm^2 (2 billion acres) of land despoiled by surface mining. While these millions of megagrams of industrial waste are being produced and stockpiled at high monetary and ecological costs, there is a considerable shortage of conventional construction materials for highway use in many parts of the country.

In view of the available volume and continued enormous rate of production of mill tailings (each unit of copper produced generates about 200 times its weight in waste), it appears that a concentrated effort is needed to evaluate the potential use of mill tailings in various aspects of highway construction and other engineering uses. This paper presents data from a comprehensive investigation that was aimed at determining the feasibility of using tailings as a construction material to replace the more scarce conventional materials.

PROPERTIES OF SELECTED TAILINGS

After a preliminary laboratory evaluation of 11 tailing specimens from Arizona, Idaho, and Utah, the engineering properties of the following three types of tailings were evaluated in detail:

1. Bisbee, obtained from the Cooper Queen Branch of the Phelps Dodge Corporation, Bisbee, Arizona;
2. Duval, obtained from the Sierrita Mine, Duval Sierrita Corporation, Sahuarita, Arizona; and
3. Magma, obtained from the Magma Copper Company, Superior Division, Superior, Arizona.

A summary of some index and engineering properties of the three tailings is given in Table 1. Additional data on these and other tailings are given elsewhere (3).

Because of space limitations, this paper deals only with the properties of compacted, cement-stabilized, and asphalt-stabilized Duval tailings. Similar results, however, were obtained for the Bisbee and Magma tailings and are given elsewhere (3).

UNTREATED DUVAL TAILINGS

Index Properties

The grain-size distribution for the Duval tailings ranged between 1.0 and 0.001 mm with a uniformity coefficient of 21. A large percentage of the materials (41.7 percent) passed the 0.075-mm (no. 200) sieve; this is attributed to the ball-mill grinding of the ore materials during processing. Since tailings are a by-product of rock-crushing processes, the individual particles are very angular in shape when observed under a light microscope.

As data given in Table 1 show, the specific gravity of 2.71 for Duval tailings was the lowest among the three types. The high values of specific gravity obtained for the other two tailings are attributed to large amounts of magnetite: 21.0 and 31.9 percent in Bisbee and Magma, respectively, versus 1.9 percent in Duval.

According to American Association of State Highway and Transportation Officials (AASHTO) classification, Duval tailings are classified as A-2-4 material.

Compaction Characteristics

The compaction characteristics of the three types of tailings were determined by using the standard AASHTO (ASTM D 698) and the modified AASHTO (ASTM D 1557) methods. The maximum dry densities and optimum moisture contents are given in Table 1. The resulting wide range in maximum dry densities among the three tailings appears to be attributable essentially to variations in the specific gravities of the tailings. The values of maximum density for Duval tailings are comparable to those of natural soils, whereas those for Bisbee and Magma tailings are significantly higher.

Compressive Strength

The compressive strength of the compacted Duval tailings was determined for specimens compacted at the maximum dry density and optimum moisture content as determined by the modified AASHTO compaction test. In addition, values of compressive strength for the specimens compacted at 95 and 90 percent of the maximum density at their corresponding moisture contents, dry and wet of optimum, were also determined.

All specimens were 10.16 cm (4 in) in diameter and 11.68 cm (4.6 in) in height—i.e., the compaction mold size. The specimens were tested immediately after compaction.

The results of the compressive strength tests are given in Table 2. These values were reached at strain levels lower than 3 percent, which indicates a brittle type of behavior. The compressive strength reaches its maximum value at or slightly below the optimum moisture content and then sharply decreases wet of optimum.

Double-Punch (Tensile) Strength

The tensile strength of compacted Duval tailings was determined by using the double-punch test (4). The test was conducted on compacted specimens similar to those discussed under the compressive strength test. Details of the testing procedure are given elsewhere (3).

The results of the double-punch tests, which are included in Table 2, indicate low values of tensile strength that do not exceed 11.72 kPa (1.7 lbf/in^2). The tensile strength reaches its maximum value at or slightly below the optimum moisture content.

Shear Strength Parameters

Direct shear tests and triaxial tests were conducted in the laboratory on the compacted Duval tailings at maximum dry density as determined by the modified AASHTO compaction test. Drained direct shear tests and consolidated drained and undrained triaxial tests were conducted.

The effective strength parameters for the Duval tailings ranged from $\bar{c}$ = 9.58 to $\bar{c}$ = 15.33 kPa (200-320 lbf/ft^2) and $\bar{\phi}$ = 36° to $\bar{\phi}$ = 37°. All specimens failed with a brittle stress-strain relation with axial strain at failure not exceeding 5 percent and exhibiting well-defined slip planes. The high effective angles of friction for the Duval and other tailings, given in Table 1, are in line with other values reported elsewhere (5, 6) and can be attributed to the high degree of angularity of copper mill tailings and consistency in their production methods.

Table 1. Properties of selected tailings.

Property	Bisbee	Duval	Magma
Color	Light brown	Grey	Dark grey
Specific gravity	3.13	2.71	3.92
Liquid limit	16.5	18.6	13.7
Plasticity index	Nonplastic	Nonplastic	Nonplastic
Shrinkage limit	11.8	16.4	12.0
Percentage passing No. 200 sieve	23.5	31.7	45.3
AASHTO classification	A-2-4	A-2-4	A-4
Optimum moisture content (%)			
Standard AASHTO	11.8	13.1	9.3
Modified AASHTO	8.9	10.8	7.7
Maximum dry density (g/cm^3)			
Standard AASHTO	2.125	1.796	2.566
Modified AASHTO	2.319	1.917	2.917
Effective cohesion, $\bar{C}$ (kPa)	14.37	9.58	7.66
Effective friction angle, $\bar{\phi}$	39°	36°	41°
Compression index, C_cx10^2	3.5	3.8	7.5
Swelling index, C_sx10^2	1.7	1.1	1.2
Coefficient of consolidation, C_vx10^{-3} (cm^2/s)	13.5	14.6	12.1
Coefficient of permeability, Kx10^{-6} (cm/s)	8.6	22.7	0.2
Rainfall erosion (%)	2.5	2.3	4.5

Note: 1 g/cm^3 = 62.4 lb/ft^3; 1 kPa = 0.145 lbf/in^2; 1 cm^2 = 0.155 in^2; 1 cm = 0.394 in.

Compressibility and Swelling Characteristics

One-dimensional consolidation tests were conducted on specimens compacted at the various density and moisture content conditions described previously under the compressive strength tests. Specimens were 6.35 cm (2.5 in) in diameter and were saturated before testing.

The summary of the test results included in Table 2 indicates very uniform characteristics. For Duval tailings, values of C_c ranged from 0.038 to 0.052, values of C_s from 0.0105 to 0.013, and values of C_v from 0.0125 to 0.0149 cm^2/s (0.08-0.096 in^2/s). The low compressibility (as shown by the low values of C_c and low swelling potential (as shown by the low values of C_s) are similar to those for highly compacted granular fine materials. The low values of C_v are indicative of materials with very low permeability, as will be shown later.

Permeability

The coefficient of permeability for the compacted tailings was determined by a pressurized constant-head permeameter. The permeability tests were conducted on specimens compacted at the density and moisture content conditions described under the compressive strength tests. Specimens were 6.35 cm (2.5 in) in diameter and 10.16 cm (4.0 in) in height.

The summary of the test results given in Table 2 indicates that the coefficient of permeability decreases as density increases, which is a characteristic of granular-type soils. As the dry density increased, the permeability sharply decreased as the moisture content approached optimum. Wet of optimum, as the density decreased the permeability increased again but at lower rates.

The permeability coefficient for the Duval tailings ranged from 22.7 × 10^{-6} to 303 × 10^{-6} cm/s (57.6 × 10^{-6} to 769 × 10^{-6} in/s). The permeability coefficients for the three tailings at maximum dry densities, which are given in Table 1, range from 2 × 10^{-7} to 2.27 × 10^{-5} cm/s (0.2-23.5 ft/year) for the Magma and Duval tailings, respectively. These values cover closely the lower and upper boundaries of the permeability range for soils of very low permeability.

Table 2. Engineering properties of compacted Duval tailings.

At Compaction Condition									
Relative Compaction^a (%)	Dry Density (g/cm^3)	Moisture Content (%)	Compressive Strength (kPa)	Tensile Strength (kPa)	Compression Index C_c	Swelling Index C_s	Coefficient of Consolidation C_v (cm^2/s)	Permeability Coefficient Kx10^6 (cm/s)	Rainfall Erosion (%)
90	1.726	4.2	161.3	7.6	0.052	0.013	0.0135	303	10.5
95	1.822	5.3	266.1	11.7	0.041	0.013	0.0149	177	8.9
100	1.918	10.8	280.6	11.7	0.038	0.011	0.0146	22.7	2.3
95	1.822	14.3	101.4	6.9	0.036	0.0105	0.0128	27.9	5.0
90	1.726	15.7	54.5	2.8	0.040	0.010	0.0125	39.2	5.8

Note: 1 g/cm^3 = 624 lb/ft^3; 1 kPa = 0.145 lbf/in^2; 1 cm^2 = 0.155 in^2; 1 cm = 0.394 in.
^{a}Modified AASHTO.

Table 3. Properties of cement-treated Duval tailings.

Amount of Cement by Weight (%)	Compressive Strength by Days of Curing (MPa)			Soaked Strength at 7-Day Cure (kPa)		Durability Loss After 12 Cycles (%)		Permeability Coefficient Kx10^{-6} (cm/s)	Rainfall Erosion (%)
	7	28	90	Tensile Strength[a]	Flexure Modulus	Wet-Dry	Freeze-Thaw		
0	-[b]	-[b]	-[b]	-[b]	-[b]	-[c]	52.9	22.7	100
2	1.55	-	-	172.4	220.6	15.2	14.4[d]	18.2	4.3
4	2.59	3.52	5.31	268.9	296.5	5.7	9.9	8.7	0.0
6	2.79	-	-	330.9	406.8	4.2	5.2	2.3	0.0
8	3.69	5.29	8.96	406.8	489.5	2.3	3.2	-	-
10	4.41	7.15	9.87	496.4	627.4	1.8	2.9	-	-
12	-	7.21	10.03	-	-	-	-	-	-

Note: 1 Pa = 0.000 145 lbf/in^2; 1 cm = 0.394 in. Dry density = 1.918 g/cm^3, and moisture content = 10.8 percent.
[a] Double punch.
[b] Specimens slaked in water.
[c] Specimens disintegrated in one cycle.
[d] Loss of 14.4 percent in five cycles.

Rain Erodibility

The erodibility of compacted tailings under the effects of rainfall was evaluated by using the Rotadisk Rainulator machine (7-9). In this study, an average rainfall intensity of 3.25 cm/h (1.28 in/h) was used for 15 min. These tests were conducted on specimens compacted at the conditions of density and moisture content described under the compressive strength tests. Details of the testing procedure are given elsewhere (3).

The summary of the test results given in Table 2 indicates that the least erosion occurred at the highest compacted density. Erosion was significantly greater dry of optimum than wet of optimum, at equal compacted dry densities. The erodibility values given in Table 1 correspond to those for specimens at maximum dry density values.

Discussion of Results

The tailings consistently included large, though nonplastic, percentages of fine particles. These particles are highly angular and thus contribute to very high angles of internal friction in such fine materials. When compacted, the tailings show high stability with relatively high compacted densities, particularly those tailings that have extremely high specific gravities. Their soil classification and their strength properties and compacted densities indicate their potential use as good to fair subgrade, foundation, and embankment materials. Because of their high erodibility by water (rainfall), they should be stabilized when exposed at the surface. The compacted tailings also exhibited very low compressibility, which indicates potential successful use in high embankments.

The very low permeabilities of the compacted tailings qualify them for potential use as reservoir and canal linings, particularly when they are protected with filter layers and not subjected to wet-dry conditions. These limitations can be removed when the tailings are stabilized with cement, as shown later in this paper.

CEMENT-STABILIZED DUVAL TAILINGS

Compaction Characteristics

The compaction characteristics of the cement-tailing mixes were determined by using the modified AASHTO compaction test. The compaction curve for the Duval tailings did not vary much in comparison with that obtained for the untreated tailings. This behavior is very common in most soils (10, 11). The cement used met the specifications of type 1 and type 2 portland cement as determined in accordance with ASTM designation C 150.

Compressive Strength

Compacted cement-treated tailing specimens were prepared by statically compacting mixtures of the Duval tailings with 2, 4, 6, 8, and 10 percent cement, at the optimum moisture content and maximum dry density, for the modified AASHTO compaction test. Specimens were 10.16 cm (4.0 in) in diameter and 11.68 cm (4.6 in) in height. The specimens were moist-cured for 7 days in a humid room, after which they were soaked for 4 h in tap water. After soaking, specimens were tested in compression according to ASTM designation D 1633. Other specimens with 4, 8, 10, and 12 percent cement were moist-cured for 28 days and 90 days, soaked, then tested in compression.

A summary of the test results is given in Table 3. The data indicate that the soaked compressive strength increased as the amount of cement and the length of the curing period increased. However, at cement contents greater than 10 percent, the increase in strength was not significant for the same curing period. A compressive strength of about 3.45 MPa (500 lbf/in^2) was achieved after 7 days of curing with about 8 percent cement.

Double-Punch (Tensile) Strength

Another group of specimens similar to those discussed above were tested in double punch after being soaked for 4 h in tap water and after curing for a 7-day period only. The test results, which are summarized in Table 3, indicate an increase in tensile strength as the amount of cement increased.

Flexural Strength (Rupture Modulus)

The flexural strength of cement-treated Duval tailings was tested according to ASTM designation D 1635. Beam specimens were statically compacted to the required density and moisture content. The beams measured 7.62x7.62x28.58 cm (3x3x11.25 in). After 7-day humid curing, the beams were soaked before testing in the third-point loading position.

The summary of the test results included in Table 3 indicates that rupture modulus R increased as the amount of cement increased.

Durability Losses (Freeze-Thaw and Wet-Dry Cycles)

The test procedures for the freeze-thaw cycles and the wet-dry cycles followed closely those outlined under

ASTM D 560 and D 559, respectively, except that the specimens were compacted to the maximum dry density at optimum moisture content under the modified and not the standard AASHTO compaction effort.

The summary of the test results in Table 3 indicates that the Duval tailings satisfied the recommended durability requirement of the Portland Cement Association (PCA) for soil-cement mixes used in base courses (12). PCA recommends that durability losses for A-2-4 soil-cement mixes should not exceed 14 percent after 12 cycles. The cement mixes with Duval tailings satisfied this criterion with the addition of only 4 percent cement by weight.

Permeability

Cement-treated specimens were tested in the pressurized constant-head permeameter. Amounts of 2, 4, and 6 percent by weight were used. Specimens were statically compacted to the maximum dry density and optimum moisture content obtained by using the modified AASHTO compaction test. After compaction, the specimens were humid-cured for 7 days before testing.

The results, summarized in Table 3, indicate a reduction in permeability with the increase of cement additive.

Rain Erodibility

Cement-treated specimens were tested in the Rotadisk Rainulator. Amounts of 2, 4, and 6 percent by weight were used. Specimens were compacted statically to the maximum dry density and optimum moisture content obtained by using the modified AASHTO compaction test. Specimens were humid-cured for 7 days after compaction. After curing, the specimens were subjected to six rain-dry cycles, each of which consisted of 1 h of rain at 3.25 cm/h (1.28 in/h) followed by 23 h of drying in an environmental room at 21°C (70°F) and 50 percent relative humidity.

The test results, summarized in Table 3, indicate a significant reduction in the rain erodibility of the tailings with the addition of cement. When 4 percent cement was added, no erosion was detected, and even when only 2 percent cement was added, a tolerably low level of erosion resulted.

Discussion of Results

The results given in Table 3 indicate that cement treatment significantly increased the compressive, tensile, and flexural strength of the compacted Duval tailings. With the addition of merely 4 percent cement by weight, the Duval tailings satisfied the PCA durability criteria for soil-cement mixes used in base courses. Cement treatment also significantly reduced rainfall erodibility and the permeability of the compacted mixes. These results indicate that cement-treated tailings could potentially be successfully used in embankments, for slope protection, in bases and subbases, and as reservoir and canal linings.

ASPHALT-EMULSION-STABILIZED DUVAL TAILINGS

The effect of asphalt stabilization of the Duval tailings was also evaluated. Because of the high amount of fines [material passing the 0.075-mm (no. 200) sieve], a slow-setting cationic emulsion (CSS-1) was considered for stabilization. No consideration was given to the use of liquid asphalt cutbacks from an energy conservation viewpoint, since petroleum fuel products are used as solvents in cutbacks whereas water is used in processing emulsified asphalts.

In the following tests, various percentages of emulsion (total weight of emulsion as percentage of dry tailing weight) were mixed with the tailings and compacted. For preparation of compacted tailing-emulsion mixtures, additional moisture of about 3 percent above the optimum moisture content was needed during the mixing operation to obtain uniform and homogeneous mixes. Before compaction, the samples were allowed to dry back to the required moisture content (at molding). The drying time required was obtained from charts developed by drying various mixes for various periods of time in the environmental room. The charts showed the relations between moisture content and time for various mixes. This procedure was followed in preparing all specimens used in the tests discussed below.

Compressive Strength

Compacted emulsion-treated tailing specimens were prepared by static compaction with 4, 8, 12, 16, and 20 percent emulsion at the maximum dry density and optimum moisture content for the modified AASHTO test. Specimens were 10.16 cm (4.0 in) in diameter and 11.68 cm (4.6 in) in height. After compaction, the specimens were cured for 7 days and 28 days in the environmental room. After curing, one set of specimens was tested immediately in compression and another set was soaked in tap water for 4 h before testing in compression.

Table 4 gives a summary of the test results, which indicate the following:

1. There is an optimum amount of emulsion at which the highest compressive strength occurs. When greater amounts of emulsion are used, lower strengths develop because of flow of the asphalt. When lesser amounts of emulsion are used, lower strengths result because of lack of bonding or adhesive forces.
2. The optimum percentage of emulsion is not necessarily the same for soaked and unsoaked compressive strengths. The optimum value is 4 percent for unsoaked strength and 8 percent for soaked strength.
3. At the optimum percentage of emulsion, values of compressive strength increased significantly in comparison with those for untreated tailing specimens.
4. Emulsion provides good waterproofing qualities to the tailing and thus protects it from disintegration while it soaks in water.
5. The values for unsoaked strength, however, are consistently higher than those for soaked strength.
6. The higher the amount of emulsion used, the higher is the axial strain at failure; this indicates increased plastic deformation and less brittle behavior.
7. The longer the curing time is, the higher is the compressive strength for both the soaked and unsoaked conditions.

Double-Punch (Tensile) Strength

Another group of specimens similar to those discussed above were tested in double punch. The test results, summarized in Table 4, indicate the following:

1. There is an optimum amount of emulsion at which the highest tensile strength occurs.
2. For both soaked and unsoaked tensile strength, the optimum amount of emulsion was 8 percent.
3. At the optimum percentage of emulsion, tensile strength values increased significantly in comparison

Table 4. Properties of emulsion-treated Duval tailings.

Amount of Cement by Weight (%)	Compressive Strength by Days of Curing (kPa)				Tensile Strength by Days of Curing (kPa)				Permeability Coefficient Kx10⁻⁶ (cm/s)
	Soaked		Unsoaked		Soaked		Unsoaked		
	7	28	7	28	7	28	7	28	
0	-[a]	-[a]	281	297	-[a]	-[a]	12	21	22.7
4	-[a]	-[a]	1160	1992	-[a]	-[a]	74	185	19.8
8	983	1723	1070	1833	67	210	123	255	11.3
12	621	1229	886	1251	81	145	99	166	7.4
16	559	980	564	1146	53	112	79	127	4.2
20	419	794	435	911	36	95	42	99	-

Note: 1 kPa = 0.145 lbf/in²; 1 cm = 0.394 in. Dry density = 1.918 g/cm³, and moisture content = 10.8 percent.
[a]Specimens slaked in water.

Table 5. Hveem strength test results for asphalt-treated Duval tailings.

Property	Value by Amount of Asphalt Emulsion				
	9 Percent	9.5 Percent	10 Percent	10.5 Percent	11 Percent
Bulk specific gravity	1.92	1.81	1.82	1.87	1.81
Air voids (%)	22.5	26.8	25.7	23.5	25.8
Stabilometer value S	34.6	34.7	45.1	31.0	31.1
Resistance value R	92.6	93.5	93.6	92.4	93.2
Cohesiometer value C	507	527	625	374	438
Swell (cm)	0.033	0.034	0.046	0.017	0.049

Note: 1 cm = 0.394 in.

with those for the untreated tailing specimens.

4. The values for unsoaked tensile strength are consistently higher than those for soaked strength.

5. The longer the curing period is, the higher is the tensile strength for both the soaked and unsoaked conditions.

Permeability

Compacted emulsion-treated tailing specimens were tested in the pressurized constant-head permeameter. Specimens were statically compacted to the maximum dry density at the optimum moisture content obtained from the modified AASHTO compaction test. Compacted specimens were cured for 7 days in the environmental room before testing.

The summary of the test results given in Table 4 indicates a reduction in permeability with increasing amount of asphalt. However, a comparison of the permeability coefficients for the asphalt-treated mixes (Table 4) and those for the cement-treated mixes (Table 3) indicates that, at an equal percentage of additive, lower permeabilities were obtained by using cement stabilization than by using asphalt stabilization. This can be attributed to the less efficient mixing of asphalt with such a fine-grained material, which results in a lower degree of uniformity.

Rain Erodibility

Emulsion-treated specimens were tested in the Rotadisk Rainulator. Specimens with 12 percent emulsion were compacted statically to maximum dry density and optimum moisture content obtained under the modified AASHTO compaction test. Compacted specimens were cured for 7 days in the environmental room before testing for six rain-dry cycles, as described earlier.

The results indicated an erosion loss of 2.33 percent at 12 percent emulsion, compared with 100 percent erosion for the untreated tailings. It is apparent, however, that cement is more effective than emulsion in eliminating erosion losses, with 4 percent cement additive (Table 3). This can be attributed to the nature of the asphalt in stabilizing aggregations of fine particles and not individual particles. Thus, when an aggregation is eroded, a large number of particles are dislodged from the asphalt-treated specimens, which results in the higher erosion level.

Hveem Strength Tests

The strength of the emulsion-treated mixtures (ETMs) was evaluated by using the Hveem procedure given by the Asphalt Institute (13) and modified as discussed below.

The specimens were mixed and compacted statically by using various amounts of emulsion. Tests of the compacted specimens were done in the following order: (a) swell test, (b) stabilometer test, (c) determination of bulk density, and (d) cohesiometer test. In addition to using static compaction instead of kneading compaction, the mixing, drying back, compaction, curing, and testing were all done at ambient temperatures of 21°C (70° ± 2°F). This is becoming an accepted technique for testing ETMs as indicated by Jimenez (14). Before testing, the specimens were fully cured in the mold for 72 h and then saturated. Curing beyond 72 h did not cause any significant reduction in moisture content.

Results of the Hveem strength test given in Table 5 indicate high strength parameters for the emulsion-treated tailings. The Asphalt Institute has presented interim guidelines for properties of ETMs in bases and surface courses (16). Using applicable data (for testing at 73° ± 3°F), the Asphalt Institute (15) recommended for fully cured and soaked specimens that minimum R-values of 78 and minimum cohesiometer values of 100 be accepted for base courses or temporary surface courses. The R-values and C-values given in Table 5 are significantly higher than the corresponding minimum values given above. The swell values given in Table 5 are consistently less than 0.762 mm (0.03 in), which is the limiting value for base courses as recommended by the Asphalt Institute (13, 4th ed.). The high values of percentage air voids for the tailing mixes in Table 5 are attributed to the relatively uniform grain-size distribution of the tailing.

Jimenez (14) reported Hveem test results for aggregate-emulsion mixes that were mixed, cured, soaked, and tested at ambient temperatures. These values were an R-value of 88, an S-value of 27, and a C-value of 540. The data given in Table 5 are in the range of magnitudes given by Jimenez.

Resilient Modulus

Specimens of emulsion-treated tailings were compacted by using a vibratory kneading compactor and tested under repetitive loading conditions to evaluate their fatigue properties. Details of the compaction and testing equipment are given elsewhere (3, 16, 17). The specimens measured 44.45 cm (17.5 in) in diameter and 3.80 cm (1.5 in) in thickness. One set of specimens was cured in the environmental room for 7 days, and another set was cured for 28 days before testing.

A summary of the test results for two sets of specimens at 8 and 12 percent emulsion content is given below (1 Pa = 0.000 145 lbf/in²):

Property	Curing Period (days)	8 Percent Emulsion	12 Percent Emulsion
Radial stress σ_r (kPa)	7	554	523
	28	635	607
Radial strain ϵ_r ($\times 10^{-3}$)	7	1.615	3.954
	28	0.963	1.524
Resilient modulus	7	343	132
E_r (MPa)	28	659	399

The results indicate the following:

1. As the emulsion content increased from 8 to 12 percent, radial strain ϵ_r increased significantly and radial stress σ_r decreased but at a much lower rate.
2. Increasing emulsion content significantly decreased the resilient elastic modulus E_r.
3. The longer the curing period was, the better were the mix properties—i.e., E_r and σ_r increased and ϵ_r decreased.

Note that the elastic moduli obtained for the tailing-emulsion mixes appear to be comparable to those usually accepted for surface, base, and subbase courses. Jimenez (18) has presented a summary review of reported moduli for surfaces, bases, stabilized bases, and subgrades. Because of the variability of mixes, materials, methods of testing, and temperatures at testing, among other factors, a very wide range of E-values is reported by Jimenez (18). The ranges given below, however, can be considered representative of the majority of the reported data (1 MPa = 145 lbf/in²):

Course	E (MPa)
Surface	690-1380
Untreated base	55-138
Stabilized base	138-690
Subgrade	1.7-138

A major concern in the tailing-emulsion mixes tested above is the high amount of voids caused by the high degree of uniformity in the grain size of the tailing. This may affect the durability of the mixes when they are used in surface courses. In view of the good E-values obtained, however, field verification of the use of the tailing-emulsion mixes should be done under actual traffic conditions.

CONCLUSIONS

The results of the investigation reported here demonstrate that copper mill tailings are an existing mining waste that has excellent engineering properties and can be easily adapted for use in road construction projects. In particular, the study shows the excellent potential for using the tailings as a compacted fill in embankments, a stable engineered fill replacement for poor soil conditions, a compacted foundation and subgrade material, a cement-treated base, an emulsion-treated base, or a compacted impervious layer for lining ponds, reservoirs, and canals. The tailings can also potentially be used as a filler in cement mortar and cement mixes or a cement-tailing grout and to produce high-quality synthetic aggregates (3, 19).

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Use of Energy-Efficient Sintered Coal Refuse in Lightweight Aggregate

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The development and evaluation of a synthetic lightweight aggregate that has particular application to the building and transportation construction industries are described. Bituminous coal refuse, a waste product obtained from five coal-preparation plants in Kentucky, was successfully sintered on a pilot-sized traveling grate to produce lightweight construction aggregate. An improved sintering-grate process was used. A means is thus provided for using a waste product while gaining an economic advantage during processing from the inherent fuel value of the refuse. A thorough laboratory investigation of the aggregate as a material for use in bituminous concrete mixes and structural lightweight portland cement concrete indicated that a satisfactory aggregate for these construction materials was produced. Using lightweight aggregate from sintered coal-mine refuse in concrete construction offers significant technical and economic incentives from the standpoints of reduced weight and the greatly reduced thermal conductivity of the products formed.

The demand for quality aggregates for various construction applications has resulted in shortages of aggregate in many parts of this country. Some areas are experiencing a depletion of all quality natural aggregates (1, 2). Environmental constraints and urban sprawl have curtailed production in some areas where aggregate supplies are abundant. Although U.S. reserves of natural aggregate are virtually inexhaustible, geographic distribution and quality do not necessarily coincide with need. This means high costs for transporting the heavy, bulky commodity.

The manufacture of synthetic aggregates and the use of by-product (waste) materials represent means that are being used to provide locally available aggregates and/or aggregates that have particular characteristics. Blast- and steel-furnace slag, power plant ash, and various mine tailings and wastes are by-product materials in current use. The most commonly manufactured synthetic aggregate is expanded lightweight shale (clay or slate), which is produced by heating the raw product to about 1040°C (2000°F) in a rotary kiln. At this elevated temperature, the gases generated expand (bloat) the material while the high temperatures stabilize it. A less commonly used method for producing synthetic lightweight aggregate is the continuous sintering-grate process, in which the raw material and an added fuel charge are placed on a traveling bed and ignited. As the product sinters (or burns), the particles fuse together and the carbon fuel burns, creating void spaces within the aggregate particles.

The Expanded Shale, Clay, and Slate Institute and the Lightweight Aggregate Producers Association promote the production and use of synthetic lightweight aggregates. All fuel requirements for rotary-kiln processing are provided from an external fuel source, whereas in the sintering operation only minimal external fuel is required and the bulk is contained in the raw feed material.

During the past four years, the Department of Civil Engineering at the University of Kentucky has been involved in research studies to develop uses for materials from bituminous coal refuse (3-9). Similar studies have been conducted in Great Britain, Pennsylvania, and elsewhere (10, 11). In the Kentucky research, the possibilities of converting the refuse into high-quality lightweight aggregate were examined. To determine the technical competence of synthetic lightweight aggregate produced from the sintering of bituminous coal refuse, a thorough laboratory evaluation was conducted on the sintered material. The basic goal of the research was to determine the suitability of the material for use as an aggregate in construction products.

The purpose of this paper is to briefly discuss the origin of coal refuse and the production of lightweight aggregate from sintered coal refuse and to more fully describe laboratory studies conducted on the sintered aggregate to determine its suitability for use in construction products.

COAL REFUSE

Coal refuse is a mixture of fragmented materials that are removed from run-of-mine coal during the cleaning and preparation process so that the quality of the coal will be improved. Sources of refuse materials include thin bands of shale and clay and other impurities and minerals inherent in or adjacent to the coal seam. It is easier and cheaper, with mechanized equipment, to extract a seam of coal with its unwanted impurities than to try to mine only the pure coal (9).

The processing is accomplished in preparation plants, some of which process as much as 18 000 Mg (20 000 tons) of coal a day. Since the coal has a lower specific gravity than the refuse materials, the coarser fractions are normally separated by heavy-media methods. Special frothing agents that attach to and float the coal are commonly used as a medium to separate the fine coal and the refuse (4).

Approximately 50 percent of the coal mined in the United States is processed in preparation plants. About 25 percent of the 272 million Mg (300 million tons) of coal is rejected, which produces 68 million Mg (75 million tons) of refuse annually. At present production rates in Kentucky, approximately 18 million Mg (20 million tons) of refuse are being produced each year (4,9). As the demand for coal increases during the coming years, it is anticipated that production of refuse will increase at an even greater rate because a larger percentage of the coal will be processed.

Environmental standards are demanding higher-quality, cleaner-burning coals that will require more intensive cleaning. This is particularly true in areas where lower-quality seams are now being worked because the higher-quality seams have already been mined. In addition, modern automated mines produce larger percentages of rejected materials because of the lack of selective mining. In addition, cleaned and processed coal results in a constant-quality product, lower transportation cost since the nonburning fraction is removed at the mine site, and an increase in the market price of several dollars per megagram over the price of run-of-mine coal.

An issue that currently faces the coal industry is how to dispose of the increasing quantity of refuse in an economically and environmentally acceptable manner (12). Conventional disposal practices involve either placing the refuse in large waste piles or pumping it behind retaining structures. Currently, it costs \$0.55-\$1.10/Mg (\$0.50-\$1.00/ton) to dispose of the refuse, or an industry cost in this country of over \$50 million/year. The per-megagram disposal costs are increasing because of the higher costs associated with more stringent environmental controls (9). Obviously, making use of coal refuse would eliminate the need for complex, permanent disposal facilities.

PROCESSING OF SINTERED AGGREGATE

Preliminary Processing

Preliminary pilot-scale rotary-kiln firings and bench-scale sinter-pot firings were conducted by using small samples of coal refuse (13). Rotary-kiln tests at the Texas Transportation Institute Research Center indicated that bituminous coal refuse responded to rotary-kiln processing and that, in spite of some handling problems, a lightweight product could be produced. However, no fuel benefit was obtained from the carbon content in the refuse because the generated heat exited through the stack and did not assist in further heating of the product. Environmental problems were also encountered because of the high sulfur content of the bituminous coal in the refuse.

Bituminous coal refuse was used in sinter-pot firings at McDowell-Wellman Engineering Company in Cleveland, Ohio. The test apparatus consisted of a balling disc and a sinter test pot, to which the refuse responded favorably. Laboratory analyses of the small quantity of sintered aggregate that was produced indicated a high-quality, lightweight product. As expected, exhaust gases from the batch sintering tests contained considerable smoke-sulfur emissions in the form of particulates of carbon and condensable hydrocarbons from the bituminous coal in the refuse. However, several tests in which simulated recycle draft was used within designed time-temperature cycles indicated that the raw materials should respond to a sintering system that involves multipass recycle draft.

Pilot Plant Processing

After the favorable preliminary results, pilot plant tests were conducted by using an improved sintering process to minimize exhaust draft quantities and to arrest combustibles in the draft stream through recycling and post-bed combustion. The intent of the pilot plant program was to demonstrate the feasibility of the process on a practical scale and to provide tonnage samples of aggregate for large-scale product evaluation.

The basic sintering process has been described as follows (14):

> In principle, the sintering process consists of charging a bed of fine moistened materials, which are then subjected to heat developed by combustion of fuel within the bed while individual particles are kept in quiescent state. An air draft is induced through the bed, made porous for the operation, and this draft combined with an ignited solid fuel provides combustion. Through heat transfer the sintering process is completed. Usually mixing, igniting, burning, and cooling are the main phases of the generic term "sintering".

When it was first developed, the sintering process was performed in a large vessel, but in 1906 the continuous sintering process was invented by A. S. Dwight and R. L. Lloyd. Although it has primarily been used for beneficiating metallic ores, this process has also been used for other purposes, including the processing of lightweight aggregate.

Refuse samples were obtained from five large coal-preparation plants in eastern Kentucky (see Figure 1). These plants are typical of "total cleaning" plants in the coal fields of that region. The plants are identified as follows: South-East Coal Company, Irvine plant—SEI; Island Creek Coal Company, Pevler plant—ICP; Beth-Elkhorn Corporation, Pike plant—BEP; Eastover Mining Company, Brookside plant—EOB; and U.S. Steel Corporation, Corbin plant—USSC.

A typical refuse sample is shown in Figure 2. The >25.4-mm (>1-in) material was initially screened from the samples. Bulk density, moisture content, and typical coal analyses of the raw refuse, as sampled, are given in Table 1. Before the refuse was processed on the traveling grate, it was permitted to dry to about 4.0 percent moisture content and hammer-milled until more than 90 percent passed a 9.5-mm (3/8-in) screen.

An extensive evaluation of processing conditions was made on the refuse obtained from the South-East Coal Company plant at Irvine (SEI). Fourteen materials-balance tests were conducted during the pilot plant program, not including a preliminary run. Sufficient data were acquired during the tests to establish a materials balance (optimum bed content of materials to be sintered), a draft flow circuit, and product analyses. The data collected were analyzed after each pilot plant test, and this information was used to establish processing conditions for subsequent tests.

The operation of the pilot plant involved delivering the crushed raw refuse and a selected amount of return (partially sintered material from previous runs) to a nodulizing-balling disc. This device used an inclined rotary pan and blended and nodulized the raw feed, particularly the fine material. The raw feed was discharged onto a 0.61-m (2-ft) wide by 5.5-m (18-ft) long traveling-grate machine positioned over active wind boxes, as shown in Figure 3. Ignition of the nodulized feed was accomplished by using natural-gas ignition torches. Various feed rates, percentages of returns, bed depths, machine speeds, ignition times, and recycle and exhaust wind-box flows were investigated. When the pilot plant was stabilized, as evidenced by relatively uniform conditions of operation, the >25.4-mm (>1-in) product (sinter cake) produced during the specific period was

Figure 1. Sites of Kentucky coal-preparation plants from which sample refuse was obtained.

Figure 2. Raw refuse as sampled.

collected and saved for subsequent evaluations. The <9.5-mm (<3/8-in) size was used for returns, as was a portion of the <25.4- to >9.5-mm size. The draft was incinerated in an afterburner, then exhausted to a scrubber, and finally to a stack (3).

Less extensive evaluations were made on samples obtained from the other four plants, and only small quantities of these were sintered. In addition, a sample that consisted of 70 percent SEI refuse and 30 percent noncarbonaceous (blue) clay was also sintered. The blue clay came from a location adjacent to the plant.

It was concluded that the improved sintering process, embodying strand cooling and draft recycling, could be favorably applied to the sintering of coal-mine waste materials. The sinter quality appeared satisfactory, and relatively low quantities of draft were available in a hot stream for final decomposition to produce a stack exhaust that was clear of visible emissions. The raw materials were relatively high in fuel content and had strong bloating characteristics. This necessitated the use of high return-bed permeability. These raw material factors limited the benefits of the improved sintering process because the high fuel content did not consistently enable complete strand cooling of the product. It was believed that the effect of these factors could be minimized by using refuse that contained a lower fuel value or a blend of inert materials, such as clay or sand, within the sinter charge.

EVALUATION OF AGGREGATE PRODUCT

Bulk quantities of the sinter cake material retained on the 25.4-mm (1-in) sieve were processed in the laboratory for use as graded aggregate in bituminous concrete and portland cement concrete. Figure 4 shows the sinter cake, which was crushed to a maximum size of 19 mm (0.75 in) by a laboratory jaw crusher and screened into various size ranges, as given below (1 mm = 0.039 in):

	Percentage Passing (by weight)		
Sieve Size (mm)	Concrete Grading	2.36-mm Grading	Open Grading
19.0	90-100	-	-
12.7	50-80	100	-
9.52	20-60	85-100	100
4.75	0-10	10-30	30-50
2.36	-	0-10	5-15
1.18	-	0-5	-
0.300	-	-	-
0.150	-	-	-
0.075	-	-	2-5

The 2.36-mm (no. 8) and open gradings are those used in bituminous mixes.

Laboratory tests were conducted on the discrete aggregate particles. Standard American Society for Testing and Materials (ASTM) procedures and specifications (15-19) were used unless otherwise noted. The test data are given in Table 2.

Dry-Loose Unit Weight

Unit weight was determined in accordance with ASTM C 29 by using the shoveling procedure (16). The concrete grading averaged 506 kg/m³ (31.4 lb/ft³). ASTM C 331 permits this value to be as high as 880 kg/m³ (55 lb/ft³) for concrete-graded lightweight coarse aggregate (16). The unit weight of the asphalt-mix-graded aggregates was around 550 kg/m³ (34 lb/ft³).

The low values are caused by a combination of low specific gravity of the sintered aggregate and a rough surface texture that prevents a tight packing condition. Most rotary-kiln-produced lightweight aggregates are slightly heavier and have smoother surface texture.

Bulk Specific Gravity and Water Absorption

Since the absorption of sintered aggregate is higher and more variable than that of conventional aggregate, a select procedure standardized by the Texas Highway Department as test method Tex-433-A (20, 21) was used. This method of testing is intended for use in determining the bulk specific gravity (both dry and saturated surface dry), apparent specific gravity, absorption, and rate of absorption of both fine and coarse lightweight aggregates.

As Table 2 notes, average dry bulk specific gravities ranged from 1.27 to 1.42 for the various gradings. After 100-min and 24-h soaks, the average bulk specific gravities ranged from 1.34 to 1.48 and 1.38 to 1.53, respectively. The average 100-min absorptions ranged from 4.26 to 5.07 percent and increased from 7.92 to 8.49 percent after 24 h. Specific gravities were slightly be-

Table 1. Data for as-sampled raw refuse.

Property or Material	SEI Sample Raw Refuse	SEI Sample Blue Clay	ICP Sample	BEP Sample	EOB Sample	USSC Sample
Bulk density (kg/m³)	1240	-	-	-	-	-
Moisture content (%)	8.1	10.6	12.5	-	9.1	11.6
Size (mm)	-25.4	-9.5	-9.5	-9.5	-9.5	-9.5
Ash[a] (%)	80.2	86.1	57.6	74.5	77.0	71.6
Volatile matter[a] (%)	12.0	8.4	19.3	13.5	13.8	14.2
Fixed carbon[a] (%)	7.8	5.5	22.8	12.0	8.7	14.2
Total sulfur[a] (%)	2.3	1.6	1.1	0.8	1.1	1.3
Heating value[a] (kJ/kg)	3220	2260	11 420	5820	5730	7990

Note: 1 kg/m³ = 0.062 lb/ft³; 1 mm = 0.039 in; 1 kJ/kg = 0.43 Btu/lb.
[a] Moisture-free condition.

Figure 3. Traveling-grate sintering process.

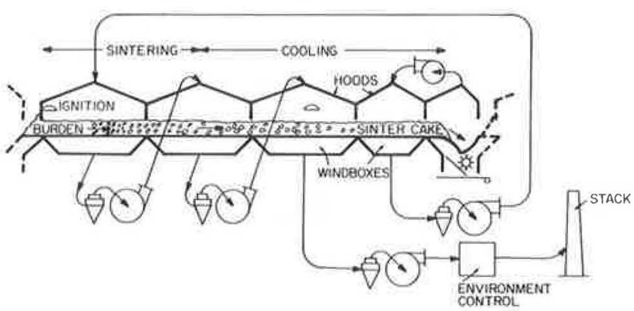
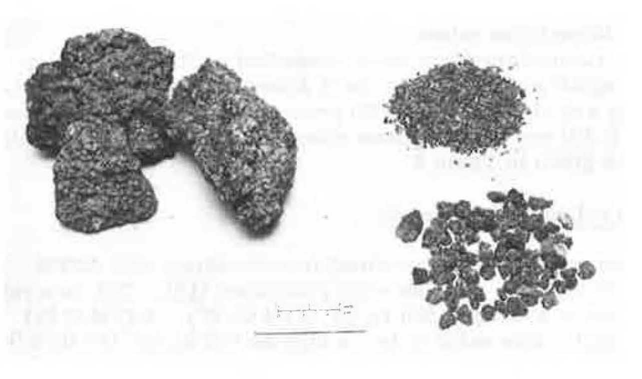


Figure 4. Sinter cake and crushed material.



low average in comparison with those of typical lightweight aggregates. Absorption values were average to slightly above average.

Soundness

ASTM standard test C 88 (16) was used to evaluate the soundness of aggregate; magnesium sulfate and five cycles of alternate immersion and drying were used. Loss values of approximately 20 percent were obtained (Table 2). ASTM C 33 specifications (16) permit the value to be as high as 18 percent for concrete-graded, normal-weight aggregate. The applicability of this test to lightweight aggregate is questionable.

Freeze-Thaw Durability

Test method Tex-432-A (21) was used to evaluate the resistance of the sintered aggregate to breakdown during alternate freezing and thawing. The samples were first saturated in water for 100 min and then subjected to 50 freeze-thaw cycles. Average losses for the asphalt-graded aggregates were approximately 5 percent; these increased to 30.5 percent for the coarser-graded concrete aggregate.

Los Angeles Abrasion

ASTM C 131 (16) was used to test abrasion loss. The coarser B grading represented the concrete-grading sizes, whereas the C grading was used for the asphalt gradings (Table 2). Average losses ranged from 43 percent for the concrete grading to 35 percent for the asphalt grading. Most specifications for normal-weight aggregate limit the loss to 40-50 percent for concrete aggregate and 35-40 percent for asphalt-graded aggregate.

Pressure Slaking

The test to determine the pressure-slaking tendency of synthetic coarse aggregate, Tex-431-A (21), is intended to be used to evaluate the amount of dehydration that has occurred in the production of synthetic aggregates fired in a rotary kiln. An average value of 8.0 percent was obtained for the sintered aggregate (Table 2). Six percent is generally considered a maximum value for rotary-kiln-fired lightweight aggregate.

Absolute Specific Gravity and 100-Min Saturation

The absolute specific gravity of the various gradings was obtained by using test method Tex 109-E, part 1 (21), and a pressure pycnometer. The 100-min saturation was calculated, by using test method Tex-433-A (21), from the 100-min absorption, dry bulk specific gravity, and absolute specific gravity. The value, expressed as a percentage, is the volume of voids in the aggregate filled with water divided by the total volume of voids available, after 100 min of soaking in water. An average value of 16 percent was obtained for the concrete grading (Table 2). About 15 percent has been established as a maximum value to ensure adequate freeze-thaw resistance for rotary-kiln-produced lightweight aggregate.

Loss on Ignition

Loss on ignition was tested to determine whether the material had been completely fired during the sintering process. Values were determined in accordance with ASTM C 114 (15) (referee method). Samples were heated to 950°C (1775°F). Average values were about 4-5 percent (Table 2). ASTM specifies that loss on ignition shall not exceed 5 percent.

Clay Lumps

The clay-lumps test was conducted as outlined in ASTM C 142 (16). The percentage values were all less than the

Table 2. Test values for sintered aggregate.

Test	Concrete Grading Value	Concrete Grading Range	2.36-mm Grading Value	2.36-mm Grading Range	Open Grading Value	Open Grading Range
Dry-loose unit weight (kg/m^3)	506	463-553	539	429-587	563	518-614
Dry bulk specific gravity	1.27	1.16-1.45	1.36	1.24-1.53	1.42	1.32-1.55
Bulk specific gravity						
100 min	1.34	1.22-1.52	1.43	1.30-1.60	1.48	1.38-1.61
24 h	1.38	1.26-1.56	1.48	1.36-1.64	1.53	1.44-1.64
Absorption						
100 min	5.03	4.61-5.48	5.07	4.05-7.18	4.26	3.81-4.59
24 h	8.49	7.71-9.43	8.42	7.48-9.64	7.92	6.67-8.86
$MgSO_4$ soundness loss (%)	22.7	5.3-44.2	21.5	7.0-46.9	21.1	8.9-46.6
Freeze-thaw loss (%)	30.5	12.3-41.2	5.1	3.2-7.4	5.0	2.4-8.3
Los Angeles abrasion loss[a] (%)	43.0[a]	40.0-47.2	34.6[b]	32.6-37.2	34.6[b]	32.6-37.2
Pressure slaking value	←		8.0	4.6-10.2		→
Absolute specific gravity	2.10	2.02-2.18	2.14	2.11-2.19	2.09	2.04-2.13
Saturation, 100 min (%)	16.37	13.00-22.52	18.82	15.75-23.86	19.46	16.79-27.81
Loss on ignition (%)	3.96	2.55-3.55	4.09	1.75-8.23	5.13	2.20-10.30
Clay lumps (%)	0.59	0.36-0.79	0.99	0.58-1.55	1.15	0.73-1.69
Organic impurities	←		None			→
Staining materials index	←		Light	Very light-moderate		→

Note: 1 kg/m^3 = 0.062 lb/ft^3.
[a]B grading. [b]C grading.

2 percent specified by ASTM (Table 2).

Organic Impurities

The organic-impurities test is used to detect the presence in natural sands of materials that might cause harmful effects in concrete products. The procedure followed was ASTM C40 (16) (alternate procedure B). No organic impurities were indicated in any of the aggregate gradings (Table 2).

Staining Materials

ASTM test C641 (16) is used to indicate the presence of iron compounds that produce staining in concrete products. Values ranged from very light to moderate (Table 2). ASTM specifications state that stain in aggregate must be classified as lighter than heavy, and all samples met that criterion.

BITUMINOUS CONCRETE

Bituminous (or asphalt) mixes that contained the sintered aggregate from the five plants were made and evaluated in the laboratory. The main attribute of this material in bituminous mixes is its skid-resistant characteristics and, to some extent, its lightweight characteristics. Both dense-graded and open-graded surfacing mixes were evaluated.

Asphalt Absorption

The percentage asphalt absorbed by the sintered aggregate was calculated according to the procedure outlined in Chapter 5 of the Asphalt Institute Manual, Series 2 (22). The percentage absorptions were based on the weight of the total aggregate. A sample composed of AC-20 viscosity-graded asphalt cement was used in the mixes.

Values of 5.4 and 6.4 percent were obtained for the open-graded and 2.36-mm (no. 8) gradings, respectively. These compare with water absorptions of 4.1 percent at 100 min and approximately 8 percent at 24 h. Asphalt absorption decreased as particle size decreased.

An examination of the coated aggregate under ultraviolet black light failed to reveal that selective absorption was occurring. Aggregate freshly mixed with asphalt and aggregate that had been coated with asphalt several months before were examined.

Evaluation of Dense-Graded Mix

Two dense-graded asphalt mixes were evaluated for medium traffic by using the Marshall design procedure of ASTM D1559 (17). Sintered aggregate was used for the coarse-aggregate fraction in each mix. Natural sand was used as fine aggregate in one mix, and manufactured limestone sand was used in the other mix. Agricultural limestone was used as mineral filler in both mixes.

The aggregates were combined to meet gradation requirements for Kentucky Department of Transportation (DOT) type B surface mix (23). The gradation limits are given below (1 mm = 0.039 in):

Sieve Size (mm)	Percentage Passing
12.5	100
9.5	85-100
4.75	60-80
2.36	40-60
1.18	25-50
0.300	5-20
0.150	3-12
0.075	2-6

The mix is similar to Asphalt Institute 6A mix. Since aggregates of widely varying specific gravities were used in the blend, it was necessary to use a volumetric proportioning procedure. A typical blend of normal-weight aggregates that meet the grading specification requires approximately 40 percent (by weight) of 2.36-mm-graded (no. 8-graded) crushed limestone aggregate for the coarse fraction. The percentage by volume is nearly the same. However, when the lightweight sintered aggregate (graded as 2.36 mm) was blended, a much lower percentage by weight was required to effect the approximately 40 percent by volume coarse fraction. Actually, the same volume of sintered aggregate could be obtained by using only 55 percent the weight of limestone required for the same volume. The volumetric blends of aggregates are given below:

Type of Aggregate	Volumetric Blend (%) Mix 1	Volumetric Blend (%) Mix 2
2.36-mm-graded sintered aggregate	35	40
Manufactured limestone sand	55	-
Natural river sand	-	40
Agricultural limestone	10	20

Since the coarse-aggregate fraction is generally con-

Table 3. Test results for dense-graded sintered aggregate mixes.

Marshall Design Value	Design Results at Optimum: Mix 1	Mix 2: Value	Mix 2: Range	Asphalt Institute Criteria[a]: Minimum	Maximum
Stability[b] (kN)				2.225	-
Unsoaked	9.210	8.245	6.676-9.6		
Soaked	8.455	7.105	6.265-8.4		
Flow (mm)				2.0	4.6
Unsoaked	3.35	2.90	2.34-3.33		
Soaked	4.19	3.00	2.64-3.38		
Air voids (%)	9.0	7.8	5.7-10.1	3.0	5.0
Voids in mineral aggregate (%)	19.6	21.5	20.0-23.8	16.0	-
Unit weight (kg/m³)	1910	1860	1800-1940	-	-
Optimum asphalt content by weight of mix (%)	8.5	10.2	10.0-10.5	-	-
Bitumen absorption by weight of aggregate (%)	3.12	3.11	2.63-4.33	-	-
Combined oven-dried bulk specific gravity of aggregate	2.167	2.113	2.064-2.187	-	-

Note: 1 kN = 225 lbf; 1 mm = 0.039 in; 1 kg/m³ = 0.062 lb/ft³.
[a]Suggested criteria for test limits for medium-traffic surfaces (24).
[b]At 60°C (140°F).

Table 4. Mix design test results for open-graded mixes.

Mix Parameter	14 Percent: Average	14 Percent: Range	16 Percent: Average	16 Percent: Range	18 Percent: Average	18 Percent: Range	20 Percent: Average	20 Percent: Range
	Value by Percentage Asphalt Content (by weight of total mix)							
Unit weight (kg/m³)	960	905-1035	1010	925-1135	1025	955-1135	1035	955-1125
Air voids (%)	34.9	30.4-36.8	30.7	25.6-33.8	28.8	23.3-31	27.5	22.3-30.4
Voids in mineral aggregate (%)	42.3	40.4-43.7	40.9	38.7-43.6	41.2	39.1-42.9	42.2	40.7-44.5
Marshall stability[a] (kN)								
Unsoaked	3.395	2.91-3.94	3.64	3.135-4.235	3.875	3.422-4.545	3.79	3.465-4.36
Soaked	3.345	3.2-3.51	3.415	3-3.77	3.715	3.311-4.34	3.420	3.135-3.645
Loss in stability after soaking (%)	1.5	-14.5-11	6.2	-0.6-10.9	4.1	0.6-12.1	9.8	-0.6-22.1
Effective asphalt content by volume of total mix (%)	7.5	5.9-10.2	10.1	7.9-12.6	12.3	10.2-15	14.6	12.5-17.4

Note: 1 kg/m³ = 0.062 lb/ft³; 1 kN = 225 lb.
[a]At 49°C (120°F).

sidered to influence the frictional properties of dense-graded asphalt mixes more than the fine fraction, it was believed that the blends would provide a satisfactory frictional level as a result of incorporating the highly skid-resistant coarse sintered aggregate and also permit use of locally available sands and mineral fillers.

Six Marshall specimens were made from each of four asphalt contents for both mixes. After measurements of bulk density, three of the specimens were evaluated for stability and flow by using the standard procedure of a 20- to 30-min water soak at 60°C (140°F) before testing. The other three specimens were soaked at 49°C (120°F) for 96 h before the 60°C stability and flow evaluations to determine the water sensitivity of the mixes. Measurements of maximum specific gravity were made on the loose mixtures according to ASTM D2041 (17), and voids and asphalt absorptions were calculated for each asphalt content.

Table 3 gives the design results at optimum asphalt content and the Asphalt Institute's suggested criteria for test limits for medium-traffic surface mixes (24). All of the suggested criteria for test limits were met with the exception of percentage air voids, which was slightly higher than the recommended content. This may be desirable, although the air voids content could probably be reduced to the 3-5 percent range by adding more mineral filler. But the rough, rugose texture of the sintered aggregate may preclude the attainment of extremely dense mixes.

Marshall stability values greatly exceeded the minimum suggested value, and flow units were within the suggested range. This indicates that very stable, workable mixes were produced.

The mixes were not sensitive to the effects of soaking in water. Moderate losses in stability were noted for the specimens with the lowest asphalt content for each mix (underasphalted), but only about 12 percent losses in stability occurred in the optimum and higher-asphalt-content specimens.

Evaluation of Open-Graded Mix

An open-graded asphalt mix was evaluated by using substantially the design procedures recommended by the Federal Highway Administration (FHWA) (25) and the Asphalt Institute (26). The grading used was the midpoint of the recommended aggregate gradation given below (1 mm = 0.039 in):

Sieve Size (mm)	Percentage Passing
9.5	100
4.75	30-50
2.36	5-15
0.075	2-5

The apparent optimum asphalt content was selected based on observation of asphalt drainage on the bottom of a clear glass dish; it was judged to be approximately 18 percent by weight of total mix. The AC-20 asphalt cement and sintered aggregate were mixed at 116°C

(240°F). No antistripping additives were used. Marshall-sized specimens were made at four different asphalt contents. The specimens were compacted at 104°C (220°F) by using 35 blows of a Marshall hammer on one end. This compactive effort is used by the Kentucky DOT in designing open-graded mixes and has been judged to be satisfactory. Three of the specimens at each asphalt content were tested by using the Marshall procedure after a 30-min soak at 49°C (120°F). The other three specimens at each asphalt content were soaked for 96 h at 49°C (120°F) before they were tested.

Table 4 gives the mix design results at the various asphalt contents for both the unsoaked and soaked specimens. Based on stability data, an asphalt content of about 16-18 percent is optimum, which is close to that estimated from appearance and drainage characteristics.

The soaked specimens averaged 5.4 percent loss in stability compared with the unsoaked specimens. At 18 percent asphalt content, the loss was only 4.1 percent. These values are well below the 50 percent maximum loss in unconfined compression permitted by the FHWA design procedure (25). Although the Marshall procedure is normally not used in designing open-graded mixes, it is believed that the index of retained Marshall stability after soaking is an applicable criterion for evaluating the resistance of open-graded mixes to the effects of water.

Voids parameters were quite high, as expected for open-graded mixes. Compacted unit weights were very low; this reflects the lightweight effects of the aggregate and high asphalt and voids contents of the mixes.

Skid Resistance

Skid resistance was evaluated in the laboratory by using a procedure developed by the Georgia DOT (27) [ASTM E 510 (17)]. Test samples were subjected to 5 million passes of the circular action of rubber wheels. At various intervals the brakes could be applied and the torque measured, and thus the polishing effect and the skid number could be derived. The results indicated that these mixes are as good as and compare well with the skid-resistant granites used in Georgia.

PORTLAND CEMENT CONCRETE

Typical concrete made in the Kentucky area contains a coarse aggregate that is normally limestone, siliceous river sand, portland cement, and water. Several mixes were made in the laboratory by using standard mixing conditions. Control mixes that contained limestone and experimental mixes that substituted sintered lightweight aggregate for the limestone were made. The purpose of the limestone mixes was to establish a control data base for comparison. Siliceous river sand was used in all mixes as the fine-aggregate fraction. Type 1 cement was used throughout the investigation. Cement factors of 7.8 and 9.2 bags/m^3 (6 and 7 bags/yd^3) were used.

Mix Design and Procedure

The limestone control mixes were designed by following the ACI 211.1 procedure of the American Concrete Institute (28). Fifty-five percent of the total volume of the aggregate was coarse limestone (no. 57s), which produced the best workable mix. In the sintered aggregate mix design, an equal volume of coarse lightweight aggregate [19 mm (0.75 in) maximum size] was substituted for the coarse limestone aggregate. The sintered aggregate was permitted to soak for 24 h before mixing.

After the 0.057-m^3 (2-ft^3) batches of concrete were thoroughly mixed, tests for slump (ASTM C143), air content (ASTM C173, volumetric method), and unit weight (ASTM C138) were made on the fresh concrete (16). Slumps were in the 75- to 100-mm (3- to 4-in) range, and air contents were between 5 and 6 percent. Wet unit weights for the sintered aggregate mixes ranged from 1740 to 1850 kg/m^3 (109-116 lb/ft^3) compared with 2330 kg/m^3 (145 lb/ft^3) for the control mixes.

Quality Evaluations

Test specimens were made to determine the properties of the hardened concrete. Six standard 152.4-mm (6-in) diameter cylinders were made for evaluating dry unit weight, compressive strength, and static Young's modulus of elasticity. Four prisms for freeze-thaw testing, two prisms for length-change determinations, and one prism to test for popout materials were made from each mix.

The normal-weight and lightweight specimens were made in accordance with ASTM C192 and cured as described in ASTM C330 (16). This necessitated removing the strength and length-change specimens from the moist room after seven days and storing them at a relative humidity of 50 percent until the time of testing.

Unit Weight

The unit weight of the hardened concrete was calculated in accordance with ASTM C567 (16). The cylinders were initially moist cured for 7 days and subsequently cured at 50 percent relative humidity for 21 days before testing. Unit weights for the sintered aggregate mixes ranged from 1700 to 1800 kg/m^3 (107-113 lb/ft^3) compared with a value of 2270 kg/m^3 (142 lb/ft^3) for the limestone mixes, a difference of 22 percent. ASTM C330 (16) permits unit weights of structural lightweight concrete of as much as 1840 kg/m^3 (115 lb/ft^3). Low unit weight is a significant feature of lightweight concrete.

Compressive Strength

The compressive strength of the concrete cylinders was determined as outlined in ASTM C39 (16). Three of the cylinders were tested at 28 days and the other three at 56 days.

Although 28 days is the standard curing period, 56 days is also used as a curing period for some lightweight applications. On the average, after 28 days of curing, a concrete mix that contained 7.8 bags/m^3 (6 bags/yd^3) of cement and was made with the control limestone had a strength of 35.8 MPa (5180 lbf/in^2) (see Table 5). A strength of about 28.3 MPa (4100 lbf/in^2) was obtained after 28 days by using the sintered concrete with the same cement factor. Generally, some strength is sacrificed when lightweight aggregate is used, particularly if it is as light as this material. But, with the 9.2-bags/m^3 (7-bags/yd^3) mixes, the lightweight aggregate at 28 days had on the average a strength nearly comparable to that of the typical 7.8-bags/m^3 limestone mix. After 56 days, the 7.8-bag limestone mix increased in strength to 36.9 MPa (5350 lbf/in^2); the 9.2-bag mixes of sintered lightweight aggregate increased in strength to an average of 35.2 MPa (5100 lbf/in^2). Because the sintered material has greater absorption, it releases water over a longer period of time. The water is thus available to continue hydration over the longer curing period. This phenomenon is quite common with lightweight concrete.

Table 5. Compressive strength of air-entrained concrete.

		Compressive Strength (MPa)								
		Limestone Control Mixes		Sintered Experimental Mixes						
				SEI						
Days of Curing[a]	Cement (bags/m³)	Average	Range	Raw Refuse	Blue Clay	ICP	BEP	EOB	USSC
28	7.8	35.8	32.9-38.6	28.3	-	-	-	-	-
28	9.2	-	-	32.8	35.7	33.6	33.8	31.1	31.6
56	7.8	36.9	34.1-39.8	31.8	-	-	-	-	-
56	9.2	-	-	-	39.0	35.3	35.0	32.9	34.0

Note: 1 MPa = 144.8 lbf/in²; 1 bag/m³ = 0.76 bag/yd³.
[a]Moist cure for first seven days.

Static Modulus of Elasticity

Testing to determine Young's static modulus of elasticity was accomplished as outlined in ASTM C 469 (16) at a concrete age of 28 days. The modulus for the 7.8-bags/m³ (6-bags/yd³) control mixes averaged 29 300 MPa (4 240 000 lbf/in²), whereas the modulus for the 9.2-bags/m³ (7-bags/yd³) sintered mixes ranged from 16 700 to 18 300 MPa (2 420 000-2 650 000 lbf/in²). As expected, values for the normal-weight mixes were higher than those for the sintered mixes.

Freeze-Thaw Durability

The specimens for the freeze-thaw test were prepared as specified in ASTM C 331 (16) by using air-entrained concrete. The resistance of the concrete to rapidly repeated cycles of freezing and thawing was determined in accordance with ASTM C 666 (16). The apparatus used followed procedure B—freezing in air and thawing in water. The process of weighing and testing for fundamental frequency was repeated at various cycle intervals. The freeze-thaw cycling is normally continued until the specimen falls below 60 percent of its initial dynamic modulus of elasticity or withstands 300 cycles of freezing and thawing, whichever comes first. All specimens exhibited excellent freeze-thaw durability. The test was discontinued after 350 cycles. All control and sintered specimens had durability factors of 100 percent.

Shrinkage

The determination of the change in the length of the hardened concrete was made in accordance with ASTM C 331 (16) and ASTM C 157 (16) by using 100 percent sintered aggregate. Measurements were taken after 28 and 100 days of curing. Shrinkage of the sintered mixes ranged from 0.023 to 0.036 percent after 28 days and from 0.056 to 0.066 after 100 days. Shrinkage of the control mixes averaged 0.040 percent after 28 days and increased to 0.048 percent after 100 days. ASTM specifications for lightweight concrete aggregate permit shrinkage of 0.10 percent. All specimens met this requirement and should not experience excessive shrinkage or expansion with curing.

Popouts

The tendency of aggregate to absorb water at high temperature and pressure was also evaluated by the so-called soundness, or popout, test in an autoclave soundness chamber. Specimens of 100 percent sintered aggregate were evaluated according to procedures described in ASTM C 331 (16) and ASTM C 151 (15). No popouts or other detrimental effects were observed in any of the specimens.

THERMAL ASPECTS

Materials that will be used in the future to construct buildings and other containments will have to be closely analyzed for their characteristics of thermal resistance. Aside from the energy conservation aspects of construction, the overall economics of construction must be carefully analyzed to establish acceptable lifetime costs.

Test specimens of structural concrete that contained the sintered aggregate and control specimens that contained limestone aggregate were evaluated. The specific mechanism used in the measurement of heat flow through the concrete section was the guarded hot plate [ASTM C 177 (19)]. The basic procedure is based on steady-state heat transfer between a hot (warm) plate and a cold plate (flat surface). When sintered aggregate was substituted for the normal-weight limestone aggregate, the overall heat-transfer coefficients decreased 55 percent for the concrete slab specimens.

More detailed information on the thermal aspects of the products is given elsewhere (3).

FINDINGS AND CONCLUSIONS

Bituminous coal refuse, a waste product obtained from five coal-preparation plants in Kentucky, was successfully sintered on a pilot-sized traveling grate to produce lightweight construction aggregate. An improved sintering-grate process that incorporates a sealed sintering facility with a multipass recycle draft was used. The process is particularly applicable to processing bituminous coal refuse because it alleviates some prominent air pollution problems and provides an environmentally acceptable process. The process provides a means for making use of a waste product while gaining an economic advantage from the inherent fuel value of the refuse. Using coal refuse as a raw material requires less than one-third the amount of added fuel required when native clays and shales are sintered conventionally.

Both dense-graded bituminous concrete mixes that contained sintered material as the coarse fraction and open-graded mixes that contained only sintered aggregate exhibited acceptable levels of stability, water sensitivity, and other design parameters. The mixes performed well in laboratory polishing tests, and the results indicate a high-friction, nonpolishing aggregate. Skid-resistant qualities are particularly important and timely, since increased emphasis is expected to be placed in the future on developing and using more highly skid-resistant paving materials.

The sintered aggregate performed very well in the portland cement concrete mixes. The test values indicated good compressive strength, excellent freeze-thaw durability, and no autoclave popouts with a unit weight in the range of 1760 kg/m³ (110 lb/ft³).

Using lightweight aggregate from sintered coal-mine refuse in concrete construction offers significant technical and economic incentives from the standpoints of reduced weight and the greatly reduced thermal conductivity of the products formed. In the thermal conductivity tests performed in this research, slab concrete with lightweight aggregate showed a 55 percent reduction in thermal conductivity over similar shapes made with normal-weight aggregate.

Bituminous coal refuse represents an essentially unlimited source of raw material for the production of lightweight sintered aggregate in the coal-producing areas of the United States. The lightweight properties and economical production costs of the synthetic aggregate will provide for relatively wide marketing areas. In view of the high costs and scarcity of fuels predicted for the future, the relatively low energy requirements of processing coal refuse will be even more attractive. In addition, the uncertainty of natural aggregate supplies in some areas and the desirability of more insulative building products and skid-resistant paving materials are apparent.

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Industrial Waste Products in Pavements: Potential for Energy Conservation

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Criteria for evaluating the potential performance of industrial waste products as pavement materials are outlined. It is shown that a net energy saving is realized over a selected analysis period whenever the energy saved in the production of raw materials for a pavement that contains waste products (in comparison with a conventional pavement design) is equal to or greater than a function of the energy cost of resurfacing and the times required for both the conventional and waste-product pavements to reach a present serviceability index of 2.5. The "marginal waste product" is (a) in energy terms, that material for which the energy saved in production of raw material is just equal to the additional energy cost of resurfacing over the analysis period, and (b) in economic terms, that product for which the cost per unit of energy saved is equal to the current unit price of energy. Potentially useful industrial waste products can be ranked according to these criteria. A performance criterion for waste materials requires that data be available on which to base reasonable estimates of serviceability history. Several examples of waste products that are currently used as paving materials are discussed, and a statistical study of the compressive strength of pozzolanic concrete correlated with available data on pavement performance is examined.

Certain industrial waste products such as fly ash and blast-furnace slag have long been known to the construction industry as useful ingredients for paving mixtures and other purposes. The use of such products has generally resulted from a combination of their low cost and the high-quality products attainable. It is quite probable that such waste materials would be used in construction even in the absence of environmental or energy considerations.

Environmental enhancement has provided some incentive for greater use of industrial waste products in construction. Previous studies of the use of such waste products have generally focused on particular materials and have usually emphasized the environmental advantages of removing those materials from stockpiles. But, if any significant environmental advantage is to accrue from the use of industrial waste products, the material in question must exist in large quantities within a large geographic area. Otherwise, such environmental effects as enhancement of the landscape and pollution reduction are not sufficient to justify the expense of research and testing. The number of industrial waste products that offer real potential for environmental improvement and have properties suitable for use in construction is therefore quite limited.

Energy conservation has recently emerged as a specific consideration in the design and construction of pavement projects, and the potential of industrial waste products in relation to energy conservation has not been fully explored. The waste materials that result from essential industrial processes and that then exist in a form that makes them useful for incorporation into construction projects with little or no additional processing offer opportunities for substantial savings in the energy required to produce the raw ingredients of a paving mixture.

It is the purpose of this paper to outline criteria that could be used to evaluate the potential usefulness of industrial waste products as paving materials. The application of the criteria proposed here should make it easier to identify waste products that are usable for pavement construction and to rank such products on a quantitative scale according to the energy they save.

ENERGY CONSUMPTION IN PAVEMENT CONSTRUCTION

The energy required to construct and maintain a pavement can be summarized as follows:

$$E = M + T + C + R + A + S \tag{1}$$

where

- E = energy consumption per unit area of pavement for some analysis period in years;
- M = energy required to produce materials for a unit area of pavement;
- T = energy required to transport materials to the job;
- C = energy required to mix, place, and compact materials for a unit area of pavement;
- R = energy consumption by road users during the analysis period;
- A = energy required to construct overlay on a unit area of pavement; and
- S = energy required for maintenance of a unit area of pavement during the analysis period.

Subscripts c and w are used in the following discussion to denote terms in Equation 1 associated, respectively, with conventional materials and waste materials. If it is then assumed that the energy expenditures for transportation of material, mixing, placing, compacting, and maintenance are about equal for all

Figure 1. Vehicle operating costs on conventional pavement.

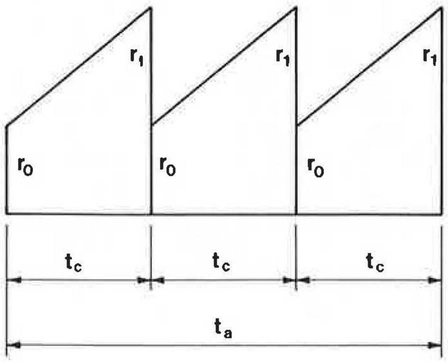
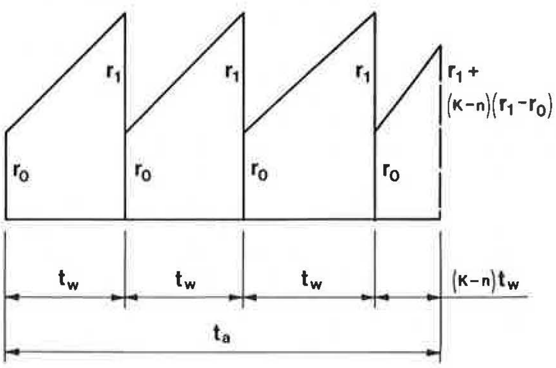


Figure 2. Vehicle operating costs on pavement that contains one or more waste materials.



types of pavement materials, Equation 1 becomes

$$(E_c - E_w) = (M_c - M_w) - (A_w - A_c) - (R_w - R_c) \tag{2}$$

The assumption that $T_c - T_w = 0$ is justified if two alternative designs are being considered that would originate from the same mixing plant, whether conventional or waste materials were used, and if sources of the respective materials are about equidistant. Significant differences in haul distance would require that the term $(T_c - T_w)$ be considered in computing energy costs.

ENERGY COST OF VEHICLE OPERATION

Haas and Hudson (1) have analyzed data from other sources and have concluded that the relation between pavement serviceability and average vehicle operating cost is reasonably approximated by a linear function. Translation of dollar costs into units of energy consumption should not significantly alter the linear relation as far as the problem under discussion is concerned as long as running speed remains constant.

Consider, therefore, a pavement constructed of conventional materials that requires a period of t_c years to reach a present serviceability index (PSI) of 2.5. In view of the approximate nature of the cost-serviceability relation, it is considered sufficient for the problem at hand to represent the serviceability history curve as a linear function also. Vehicle operating costs (in energy units) are therefore as shown in Figure 1. When the pavement is new and immediately after each resurfacing, vehicle operating cost is r_0 joules per vehicle kilometer. But after t_c years following construction and each subsequent overlay, the PSI has fallen to 2.5 and vehicle operating cost has risen to r_1 joules per vehicle kilometer.

If the analysis period of t_a years is taken to be an integral multiple of the resurfacing cycle time t_c, it is clear from Figure 1 that the average vehicle operating cost during the analysis period is

$$r_c = (r_0 + r_1)/2 \tag{3}$$

Now consider a pavement constructed by using one or more waste products, which requires a period of t_w years to reach a PSI of 2.5. Consider that $t_w < t_c$. Figure 2 shows the resurfacing cycle times and the analysis period t_a. Let n be the number of overlays placed during the analysis period and $K = t_a/t_w$. It can then be seen from Figure 2 that the average vehicle operating cost in this case is given by

$$r_w = 1/t_a\,[n \cdot (r_0 + r_1)/2\; t_w + 2r_0 + (K - n)(r_1 - r_0)/2 \cdot (K - n)t_w] \tag{4}$$

Equation 4 can be reduced to

$$r_w = (r_0/2K)[n + 2(K - n) - (K - n)^2] + (r_1/2K)[n + (K - n)^2] \tag{5}$$

The difference in the average energy cost of vehicle operation can thus be shown to be

$$r_w - r_c = \{[(K - n)^2 - (K - n)]/K\} \cdot [(r_1 - r_0)/2] \tag{6}$$

Since $(K - n) < 1$, it is clear from Equation 6 that $(r_w - r_c)$ is comparatively small in magnitude. But it also follows from Equation 6 and from the fact that $(K - n) < 1$ that $(r_w - r_c)$ is strictly negative.

The foregoing analysis depends, of course, on the assumption that overlays will be provided at the appropriate times and that such overlays will restore the PSI to a level approximately that of a new pavement. In view of the comparatively small magnitude of $(r_w - r_c)$ as given by Equation 6, it will be on the safe side to take account of the approximate nature of the analysis by concluding that

$$r_w - r_c \approx 0 \tag{7}$$

It therefore follows for a given traffic volume that

$$R_w - R_c \approx 0 \tag{8}$$

where R_w and R_c = the total energy expenditures by road users during the analysis period on the two respective types of pavement.

ENERGY COST OF RESURFACING

In the foreseeable future, it does not appear that a material will be available to take the place of asphalt as a binder for resurfacing courses. Consequently, the energy cost of an asphaltic concrete overlay (in joules per kilometer) will be the same for all types of pavements of a given width. Let a represent this unit energy cost of resurfacing. For a given thickness of overlay, a is constant; it then follows immediately that

$$A_w - A_c = an - a(t_a/t_c) = a[n - (t_a/t_c)] \tag{9}$$

where

A_w = resurfacing cost per kilometer in energy units for a pavement constructed with one or more waste materials over analysis period t_a years,

A_c = resurfacing cost per kilometer in energy units for a conventional pavement over analysis period t_a,
n = number of overlays required during t_a for a pavement constructed with one or more waste materials, and
t_c = period of years required for conventional pavement to reach a PSI of 2.5 after construction or overlay.

ENERGY CRITERION FOR SELECTION OF WASTE MATERIALS

Combining Equations 2, 8, and 9 gives

$$E_c - E_w = (M_c - M_w) - (A_w - A_c) = (M_c - M_w) - a[n - (t_a/t_c)] \quad (10)$$

Thus, a net saving in energy results from the use of waste products if

$$M_c - M_w \geqslant a[n - (t_a/t_c)] \quad (11)$$

Equation 11 can form the basis of a quantitative definition of the "marginal waste product" in terms of energy consumption and pavement performance. In those terms, the marginal waste product is that material for which the energy saved in raw material production is just equal to the additional cost of resurfacing over a selected analysis period.

For example, Wells (2) found that digested domestic wastewater sludge could be used in lieu of some fine aggregate or mineral filler in bituminous concrete and produce satisfactory results in terms of Marshall stability and air void content. But the biological stability of the dried sludge can only be ensured by heating it to 141°C (285°F) and premixing the sludge with hot asphalt cement before the binder is added to the hot aggregate.

An energy analysis of the use of digested wastewater sludge can be computed by using data developed by Barenberg and Thompson (3). The energy values used in the following discussion are based on the work of Barenberg and Thompson for typical job conditions but are for illustrative purposes only. Wells (2) showed that sludge solids could be incorporated into bituminous concrete to about 3 percent of the dry weight of aggregate. Since the digested sludge is a by-product, the energy required to make it available for pavement construction is essentially zero. If, then, the energy required to produce 0.9 Mg (1 ton) of aggregate for bituminous concrete is about 73.9 MJ (70 000 Btu), the gross energy saved by substituting 3 percent wastewater sludge should be about 2.44 MJ/Mg (2100 Btu/ton).

Let us consider only this gross energy saving for the moment and suppose that the sludge is to be evaluated for use in a 7.6-cm (3-in) thick bituminous concrete course that has a compacted density of 2307 kg/m^3 (144 lb/ft^3). Gross energy saving per unit area can thus be expressed as $M_c - M_w = (0.076)(2307)(2.44) = 0.429$ MJ/m^2 (340 Btu/yd^2). From data given by Barenberg and Thompson (3), the energy cost of a conventional bituminous concrete overlay 3.8 cm (1.5 in) thick is about 41.6 MJ/m^2 (33 000 Btu/yd^2). Application of Equation 11 then gives

$$M_c - M_w = 0.429 \geqslant 41.6[n - (t_a/t_c)] \quad (12)$$

or $[n - (t_a/t_c)] \le 0.01$. Equation 12 indicates that digested wastewater sludge is a submarginal material for the application being considered unless bituminous concrete made with this waste material can serve just as long as conventional bituminous concrete before requiring an overlay $[n \approx (t_a/t_c)]$.

When the energy required for biological stabilization of wastewater sludge is included in the calculation, the value of $(M_c - M_w)$ becomes negative. In this case, the sludge is worthy of consideration in a paving application only if it provides substantially better service than conventional bituminous concrete—an unlikely condition.

Let us turn now to a hypothetical example at the other extreme. It is not likely that a material would be considered on nonenergy grounds for use in a pavement if that material resulted in a pavement that required resurfacing twice as often as its conventionally constructed counterpart. But, if energy conservation is taken into account, the following expression results from Equation 11:

$$M_c - M_w \geqslant 41.6 \cdot (t_a/t_c) \quad (13)$$

If the conventional pavement is resurfaced twice during an analysis period of 40 years, the minimum required energy saving for the hypothetical waste material to achieve marginality from the energy standpoint is about 83.2 MJ/m^2 (65 900 Btu/yd^2). An energy saving of this magnitude seems well beyond any saving potentially available, given the present state of the art.

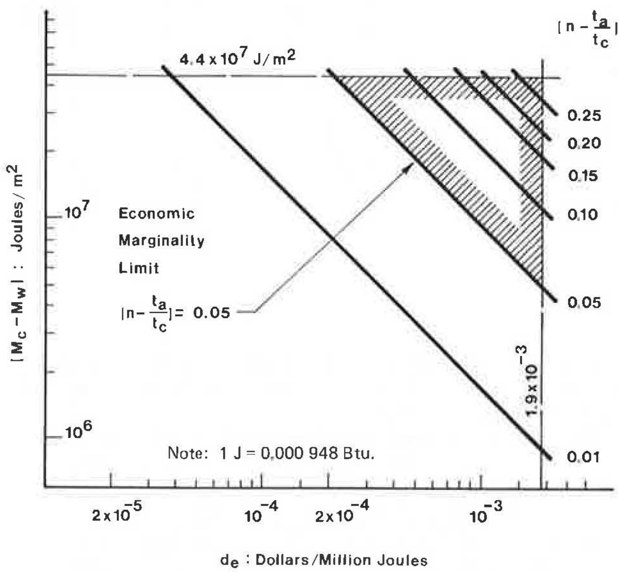
Hughes and Haliburton (4) investigated the use of zinc smelter waste as an aggregate component in bituminous concrete mixtures. The smelter waste required crushing and sorting or the addition of fine sand to meet specified gradation requirements so some expenditure of energy is required to prepare this waste material for use. According to Hughes and Haliburton (4), the highest proportion of fine sand added was 50 percent. If we use the data developed in a previous example, the gross energy saving that results from use of the zinc smelter waste can be estimated at about 50 percent of the energy expended in preparation of conventional aggregate, or about 40.7 MJ/Mg (35 000 Btu/ton).

For a 7.6-cm (3-in) thick bituminous concrete course with a compacted density of 2162 kg/m^3 (135 lb/ft^3), gross energy saving per unit area would be $M_c - M_w = (0.076)(2162)(40.7) = 6.69$ MJ/m^2 (5300 Btu/yd^2). If zinc smelter waste is to be at least marginal on an energy basis, Equation 11 (with a = 41.6) shows that the factor $[n - (t_a/t_c)]$ cannot exceed 6.69/41.6, or about 0.16. That is, bituminous concrete made with zinc smelter waste must be sufficiently serviceable to require resurfacing at intervals of not less than about 18.5 years if the corresponding period for conventional bituminous concrete is 20 years.

The examples presented here pertain to asphaltic concrete, but it should not be inferred from these illustrations that industrial waste products are potentially useful only in asphaltic mixtures. Furthermore, the examples have been selected to demonstrate a range in energy cost and are not intended to imply that a wide range of products can be used in asphaltic mixtures.

The foregoing examples tend to show that substitutions of waste products for conventional pavement construction materials are feasible from an energy standpoint only over a limited range of pavement performance. The zinc smelter waste may not be at the upper limit of this range, but it is difficult to conceive of a waste product or a combination of products so energy conserving that the factor $[n - (t_a/t_c)]$ could exceed about 0.25. In other words, for an analysis period of 40 years, it does not seem feasible to look for potential use of waste products as a pavement ingredient if the average frequency of resurfacing of such a pavement is expected to be more than once in 18 years compared with a standard

Figure 3. Cost of energy saved by use of waste products in pavement.



of once in 20 years. As the frequency of resurfacing of conventional pavements increases, the performance requirements for a waste material become more severe. For example, if $n - (t_a/t_c) = 0.16$, $t_c = 5$ years, and the analysis period $t_a = 40$ years, the marginal waste product must provide 4.9 years of service between overlays.

ECONOMIC CONSIDERATIONS

If the unit cost (present value) of an overlay is d dollars, the additional cost of a pavement in which one or more waste materials are used can be determined from the following equation, in terms of dollars per energy unit saved, in comparison with a conventional pavement:

$$d_e = d[n - (t_a/t_c)]/(M_c - M_w) \tag{14}$$

where d_e equals the cost per energy unit saved by the use of one or more waste products, which increase the required number of overlays per analysis period t_a from t_a/t_c to n.

The marginal waste product in economic terms can then be defined as that product that provides energy savings and pavement performance so that d_e is just equal to the current unit price of energy. Figure 3 shows the relation between d_e and $(M_c - M_w)$ for a unit cost of overlay equal to \$3.40/m² (\$2.84/yd²). The current unit cost of energy, which is shown in Figure 3, has been estimated from the bulk rate for natural gas in Ohio and a heating value of 37.3 MJ/m³ (1000 Btu/ft³). The approximate bulk price of natural gas was estimated at \$0.071/m³ (\$2/1000 ft³). The values of the factor d in Equation 14 were computed by adding the present values of future overlays on the basis of a 40-year analysis period, 20 years between overlays for conventional pavement, and an interest rate of 10 percent.

Figure 3 also shows the approximate energy per unit area of pavement required to manufacture the material for a crushed aggregate layer that is 25.4 cm (10 in) thick: 44 MJ/m² (3870 Btu/ft²). This value represents an upper bound on the energy saving for a 25.4-cm thick layer, since it implies the total substitution of waste material for conventional aggregate. The area in Figure 3 below this upper bound and to the left of the line that represents the current unit price of energy thus constitutes the "economic feasibility space" for the use of waste materials for the purpose of energy conservation.

It can be seen in Figure 3 that this economic criterion further narrows the practical range of the acceptable characteristics of waste materials for paving purposes. Materials that offer potential pavement performance as poor as that associated with $n - (t_a/t_c) = 0.25$ must provide a minimum energy saving of 34 MJ/m² (3000 Btu/ft²) to meet the test of economic marginality, whereas materials that offer performance almost as good as that of conventional pavement $[n - (t_a/t_c) = 0.01$ or $0.05]$ can still be attractive from the economic standpoint if they offer only one-tenth as much energy savings.

MATERIALS REQUIREMENTS OR ENERGY CONSERVATION

The foregoing analysis provides some guidelines for a search for industrial waste products that are potentially useful for pavement construction. Although it is clear that materials that are capable of performing nearly as well as conventional materials are very much to be preferred, the field of potentially useful materials is not so narrow as it might seem at first glance.

First of all, if energy conservation and not environmental enhancement is the motivation for using waste products, it is not necessary that the materials in question be available in massive quantities over a wide geographic area. Although this may still be desirable, it is likely that materials that possess suitable characteristics and are available in sufficient quantities to satisfy local or regional needs exist in various localities. The zinc smelter waste mentioned earlier is one example of such a material. Stack dust from lime plants may be another example. Cement-kiln stack dust is both widely available and energy conserving.

Second, the energy crisis promises to be a continuing problem in our society. For the foreseeable future, the conservation of energy can be expected to become an increasingly vital concern in our national life. Even though the general public does not yet seem to take energy conservation very seriously, it is conceivable that energy could become such an overriding national concern as to affect both the attitudes and the lifestyles of many citizens. In such an event, public attitudes toward pavement rideability could change and thus modify the economic relations that have been shown to define the parameters of acceptable waste products for pavement use. Furthermore, it will become increasingly difficult to obtain high-quality conventional materials. It may well be necessary for both the traveling public and transportation officials to reevaluate pavement performance standards in the light of a threatened shortage of both materials and energy.

In anticipation of the increasing importance of energy conservation, it would be beneficial to investigate now the potential usefulness of waste products somewhat beyond the present limits of economic marginality. Priorities for the investigation of new materials can be established on the basis of potential energy saving and expected performance.

PERFORMANCE AND LABORATORY TESTS

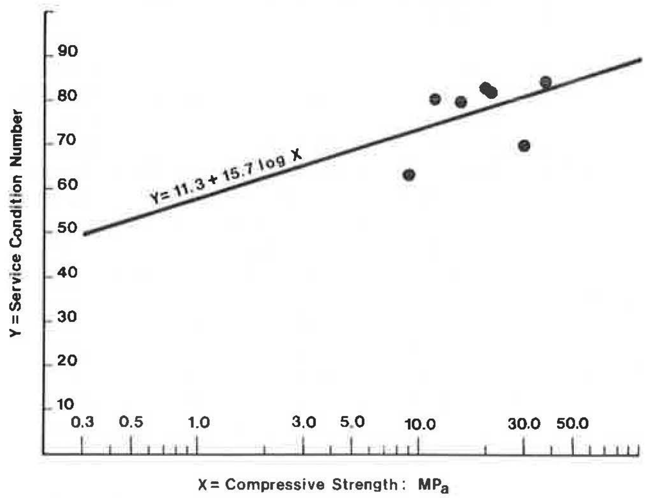
Use of a performance criterion for selected energy-conserving materials requires that data be available on which to base a reasonable estimate of the expected serviceability history of pavements that incorporate such materials. Laboratory tests can always be per-

Table 1. Pavement evaluation data for eight projects in Cook County, Illinois.

Street	From	To	Year Built	1978 Average Core Strength (MPa)	Present Condition Number	Surface Condition Index
Ela Road	Algonquin	Central	1974	11.89	81.0	84.0
Ela Road	Bradwell	Palatine	1968[a]	37.82	85.3	85.4
Ela Road	Dundee	Baldwin	1968	15.69	80.6	86.4
Ela Road	Freeman	Algonquin	1967	30.58	70.8	71.6
Howard	Gross Point	Frontage	1958	19.82	55.7	36.7
Plum Grove	Mecham	Old Plum Grove	1974	20.24	84.0	82.2
Potter	Golf	Ballard	1959	9.07[b]	65.0	65.0
Quentin	Palatine	Illinois	1964	21.59	82.6	76.9

Note: 1 MPa = 145 lbf/in².
[a]New surface, 1975.
[b]Core tested May 1977.

Figure 4. Service condition number versus current compressive strength for Cook County pavements (excluding Howard Street).



formed to determine physical properties of paving mixtures under controlled conditions. Test data such as modulus of rupture of concrete or Marshall stability of bituminous mixtures have long been used as indices of the probable performance of conventional pavements.

Of course, many variables other than the physical properties of paving materials affect the performance of any pavement. Traffic, subgrade, and environmental variables all influence pavement performance and thereby prevent a high correlation between the materials properties and the service life of a completed paving project. But effective use of an energy-conserving waste product does not require a precise statistical estimate of service life, at least not for the purpose of screening candidate materials for further research and development. It is only necessary at this stage to determine whether or not the expected service life of a pavement constructed by using the candidate material is close to that of a conventional pavement.

An example will show that such a determination can be made on the basis of laboratory data. Pozzolanic concrete pavement bases that contain lime and fly ash collected from the stacks of coal-burning utilities have been used for many years by various states. Some data are available on this material so that a comparison can be made between the compressive strength of cores and an independent measure of the expected remaining service life of pavements from which the cores were taken.

Table 1 gives data on eight pavements in Cook County, Illinois, that were constructed with pozzolanic concrete bases and have been included by the county in an extensive program of pavement evaluation. At the time this paper was written, these eight projects contained the only pozzolanic bases for which both core-test and evaluation data existed.

The surface condition index given in Table 1 is a number from 10 to 100 that summarizes a detailed visual evaluation of the pavement surface in which the number and severity of cracks and other surface anomalies are determined. This value decreases as the condition of the pavement surface deteriorates.

Service condition number, which also varies from 10 to 100, is a function of the expected service life of the pavement with which that number is associated. The service condition number is a weighted function of dynaflect observations, surface condition index, deflection number, and other data, including expected traffic. Values of 85 or more indicate an acceptable pavement with a service life of at least 10 years under expected traffic. Values from 80 to 85 indicate that a pavement service life of 10 years can be anticipated but that more maintenance must be expected and performance will be at a lower PSI. Values from 70 to 80 indicate that some reconstruction will be required within 10 years. Values of 60 or less indicate an expected service life of 5 years or less. As the service condition number drops below 60, the need for remedial action or reconstruction becomes more immediate.

Both surface condition index and service condition number, as used in this paper, are indices used by the Cook County Department of Highways and are principally based on data given by Chang, Phang, and Wrong (5, 6).

Although none of the pavements described in Table 1 is new, each pavement has an average compressive strength of core samples associated with pavement evaluation data taken at about the same time. It is thus possible to study the relation between the present compressive strengths of the pozzolanic concrete bases and the probable remaining service lives of the pavement systems of which such bases form a part. Howard Street has been omitted in most of the discussion that follows because its surface condition index is so poor that surface condition is clearly an overriding factor. The remaining pavements exhibit a much more limited range of surface condition values.

The relation between current compressive strength and service condition number was first studied at the ordinal level. For the seven pavements other than Howard Street, Spearman's rank correlation coefficient between present compressive strength and service condition number was found to be +0.57. The corresponding value with Howard Street included was +0.52. Calculations of Kendall's tau yield values of +0.43 without Howard Street and +0.36 with Howard Street. These correlation coefficients are all statistically significant at a level of confidence of about 0.15 and thus offer a fairly strong indication that a ranking of waste materials in accordance with laboratory compressive strengths reasonably approximates the ranking by service lives of pavements that incorporate the waste material in question.

Regression calculations in which data given in Table 1 (omitting Howard Street) were used yielded the following:

$$Y = 11.3 + 15.7 \log X \quad (15)$$

where Y = service condition number and X = average compressive strength of cores (kPa). The equation and the associated data are shown in Figure 4. The coefficient of determination (R^2) is 0.21. This is not high enough to be statistically significant, but it does indicate a discernible positive relation between present compressive strength and expected service life.

The table below gives the relation between core compressive strength and expected remaining service life as developed from Equation 15 and the interpretation of service condition number used by Cook County (1 MPa = 145 lbf/in^2):

Anticipated Remaining Service Life to PSI = 2.5 (years)	Minimum Compressive Strength of Pozzolanic Concrete Base (MPa)
≥10	23
5-10	1
<5	<1

This table is illustrative only, since compressive strengths of pozzolanic bases vary in accordance with the properties of material used in different localities. Its purpose is to demonstrate that laboratory data can be used to develop an estimate of expected service lives of pavements that is adequate for at least a preliminary study of waste materials that offer potential energy savings in pavement construction.

CONCLUSIONS

The following conclusions are considered to be supported by the work presented in this paper:

1. The use of industrial waste products in pavement construction produces net energy savings whenever the energy saved in the production of raw materials (compared with some conventional design) exceeds a specified function of the energy cost of resurfacing and the time required for both the conventional pavement and the project that contains the waste product(s) to reach a PSI of 2.5.
2. Under existing conditions, waste products that do not provide a pavement service life nearly equal to the life that can be attained by using conventional materials are not feasible from the standpoint of energy conservation.
3. Comparing the cost of energy saved with the current price of energy, as an economic criterion, further restricts the range of feasible waste products under current standards of highway performance.
4. A deepening of the energy crisis would widen the range of waste products that are usable in pavement construction by increasing the cost of energy and thereby raising some new materials above the marginal level from the economic standpoint. It is also possible that energy shortages could bring about changes in public attitudes toward standards of highway performance. Changes in such standards could render many more industrial waste products feasible for the purpose of energy conservation.
5. Potentially useful waste products that are somewhat beyond the current limits of economic marginality should be investigated in anticipation of the increasing importance of energy conservation.
6. It has been demonstrated that it is feasible to use laboratory test data to develop an approximation of anticipated pavement service life in order to apply the criteria presented in this paper for evaluating the potential usefulness of waste products for energy conservation.

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Cold Recycling of Failed Flexible Pavements with Cement

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Recycling of failed pavements as a means of conserving materials and saving energy and money is examined. Documentation of the use of this method dates back to the 1940s. More recent experience with in-place cold recycling with cement in several states is outlined. Two theoretical pavement projects are used to demonstrate in detail the energy and cost requirements associated with this method of pavement rehabilitation. It is concluded that strengthening the existing pavement material in place by means of cold recycling, with cement as the binder, can produce substantial savings in energy and costs.

Conservation of aggregates and energy and cost savings are possible by means of rehabilitation and reconstruction of failing or failed old flexible pavements. The reuse of existing roadbed materials and surfacing by recycling in place with cement stabilization is one of many alternatives. Considerable engineering judgment is needed to arrive at the proper rehabilitation alternative. In-place cement stabilization is one of the processes available to the highway engineer to help solve paving problems. The process is not new. Examples can be cited that date back to the 1940s. In this paper, several estimates of today's energy use and costs are presented as a guide to those concerned with energy conservation.

PAVEMENT RECYCLING

A noted highway administrator said recently, "The byword of the future is conservation—conservation of money, energy, and materials." The benefits of recycling are easily identified when the tasks required to construct a pavement are considered: obtaining sources of raw materials such as aggregates and binder, production of materials, transportation, and disposal of the old pavement. When the pavement is recycled, all of these raw materials are conserved, transportation costs are greatly reduced, and disposal of the old pavement need not be an environmental problem.

The two broad categories of recycling are (a) surface recycling, in which the objective is to improve pavement roughness and skid resistance, and (b) base and surface recycling, in which the objective is to improve the load-carrying capacity of the pavement as well as to improve surface conditions. In many cases, surface distortion, rutting, and cracking are associated with inadequate load-carrying capacity of the base. The most probable causes are insufficient base thickness, increased traffic, age, and poor drainage. The most common failure with stone-and-gravel base occurs when the subgrade is saturated and traffic loadings force wet subsoil up into the base. Aggregate interlock is then lost, and the structural capacity of the base is appreciably reduced. The problem is how to correct a pavement with a stone-and-gravel base that has been structurally weakened by soil infiltration.

ALTERNATIVES

Several means of increasing the load-carrying capacity of pavements are readily available: (a) overlaying with a substantial thickness of asphaltic concrete, (b) reconstructing by hauling out the old base and surface material and building a new pavement, and (c) strengthening the existing material by cold recycling in place with any of several binder materials and placing a new surface.

COLD RECYCLING IN PLACE WITH CEMENT

One of the oldest and best-documented stabilization binders is portland cement. Soil-cement or cement-treated base was originally developed to use inexpensive in-place or nearby borrow materials. Cement stabilization is adaptable for a wide range of materials. The process is economical because only portland cement and water are hauled to the jobsite. One of the older applications of soil-cement is in the rebuilding or reconstruction of failing granular-base roads. This is the highly successful process now called recycling.

Many references on the subject date back to the 1940s (1-6). A 1960 article (7) describes a rather small street project [20 100 m^2 (24 000 yd^2)] on which cost savings were more than $\$1.73/m^2$ ($\$1.45/yd^2$) as a result of using in-place cement-treated base instead of hauling out failing street material and replacing it.

In more recent years, the state of Nevada has used in-place recycling with cement to rebuild more than 800 000 m^2 (1 million yd^2) of old, failing granular-base roads (see Figure 1). Reported construction costs have ranged from $\$1.20/m^2$ ($\$1/yd^2$) for a 127 000-$m^2$ (152 000-yd^2) project built in 1969 to $\$1.65/m^2$ ($\$1.38/yd^2$) for a 1975 project that involved 14.6 km (9.1 miles) and 155 000 m^2 (186 000 yd^2). The assistant district engineer for maintenance of the Nevada Department of Highways has said, "As Nevada's supply of good, cheaply produced aggregate becomes rarer, as asphalt prices continue to escalate, maximum utilization of the aggregate in existing pavements and bases through recycling and stabilization methods could become more and more an economic imperative" (8).

In Louisiana in 1977, some 33 projects totaling 1.8 million m^2 (2.1 million yd^2) were awarded for the recycling of old and wornout granular-based asphalt roads (see Figure 2). According to Harvey D. Shaffer of the Louisiana Department of Highways (9),

> Cement stabilization of existing base and surfacing represents our most common compromise between two extremes . . . . This has proved to be a very cost-effective method of improving the rideability and structural qualities of the pavement. Other advantages over a simple overlay include [the following]:
>
> 1. The width of the riding surface can be increased . . . with only a small percentage of increase in construction cost . . . with no changes in foreslopes and ditches.
> 2. The expensive . . . procedures for removing and replacing isolated sections of road that have had base failures are not necessary . . . .
> 3. From the standpoint of environmental and energy considerations, this method is better than providing an equivalent overlay, since less new material is required.
> 4. The safety and appearance aspects are improved . . . . With a thick overlay you noticeably decrease the shoulder width as well as increase the elevation difference between riding surface and ditch.
> 5. . . . corrections in cross slope, rutting, etc., can be made in shaping the base course.

Figure 1. Breaking up old mat with a preparizer on Nevada project.

Figure 2. Soil-cement mixing on Louisiana project.

Figure 3. Old mat broken up and recycled into soil-cement on Virginia street project.

Figure 4. Reconstruction with soil-cement on Virginia street project.

Another important consideration in using thick overlay rather than recycling existing materials is the transition at bridges. Either the old base and surface have to be removed or, if the overlay is placed full depth on the bridge, the load capacity must be checked.

States that have been plagued recently with major pothole maintenance brought on by low structural base support during freeze-thaw and wet-dry cycles are examining the merit of cement-base stabilization. Because cement-stabilized bases are semirigid and have high-impermeability properties that are designed to resist wetting and freeze-thaw, they maintain uniform load-carrying capacity through all seasonal temperature changes.

The state of Virginia started using cement stabilization to rebuild old flexible streets about 14 years ago and has been active in this work ever since (see Figures 3 and 4). The Virginia Department of Highways and Transportation took bids in May 1977 for a 200 000-m^2 (237 000-yd^2) street rehabilitation project in Fairfax County. The cost of the cement-stabilized base was \$2.25/m^2 (\$1.88/yd^2). In 1978, the department awarded three small maintenance restoration projects in Fairfax, Hanover, and Augusta Counties that call for 182 000 m^2 (218 000 yd^2) of cement stabilization.

ENERGY REQUIREMENTS IN PAVEMENT CONSTRUCTION

To determine the total energy required in the construction of pavements, it is necessary to consider the energy required to produce the materials; the energy if any in the materials being used; the energy required to haul the materials from their source to the construction project; the energy in the fuel used to operate the machinery to mix, haul, spread, compact, and finish the base; the energy in the curing compound or tack coat and its application; and the energy in the surfacing material and its application.

In this paper, the total energy requirements and costs associated with the rehabilitation of two theoretical projects are compared. The first project is a highway that shows distress from age and increased traffic. The energy required to rehabilitate this highway by cold recycling in place with cement is compared with an alternative solution of providing a substantial AC overlay with some base patching and bringing the shoulder area up to grade with additional aggregate material. The second project is a failing granular-base street with curb and gutter. The energy required to cement-stabilize this street and place a new surface is compared with the energy required to haul out the old base and surface material because of grade restrictions and haul in new base and surface material.

The basic energy units used in the project calculations are as follows:

1. Work = 2.69 MJ (0.746 kW·h) (10).

Table 1. Summary of energy requirements and costs for two projects.

Project	Process	Energy (MJ/m^2)	Estimated Cost Range ($\$/m^2$)
1	Recycling existing base and surface in place with cement and placing new surface	420	3.75-5.25
	Asphalt-concrete overlay with some base patching and new shoulder material	770	5.00-7.00
2	Recycling existing base and surface in place with cement and placing new surface	400	3.90-4.80
	Hauling out old base and surface and replacing with new base and surface	590	6.80-8.40

Note: 1 MJ/m^2 = 793 Btu/yd^2; $\$1/m^2$ = $\$0.8361/yd^2$.

2. Diesel fuel = 38.7 MJ/L (139 000 Btu/gal) (11).
3. Gasoline = 34.8 MJ/L (125 000 Btu/gal) (11).
4. Cement production = 7327 MJ/Mg (6.3 million Btu/ton) (12).
5. Asphalt = 44 000 MJ/m^3 (6.636 million Btu/bbl) or 44 MJ/L (158 000 Btu/gal) (5). This does not include drilling for crude oil, at X MJ/m^3 × X dry holes = X MJ/m^3, plus X MJ/m^3 for transportation from well to refinery, at 4 percent asphaltic content = X MJ/m^3, to be added to asphalt values.
6. Diesel truck haul = 2.55 km/L (6 miles/gal) (13).
7. Asphalt concrete in place = 6.46 $MJ/(m^2/m)$ [130 000 $Btu/(yd^2/in)$] (14, 15). This includes 683 MJ/Mg (587 000 Btu/ton) for asphalt cement manufacture; 44 000 MJ/m^3 (6.636 million Btu/bbl), the

Table 2. Calculations of materials required for project 1 (highway recycling in place with cement).

Material	Calculation	Quantity
Cement	216 kg/m^3 density ÷ 1.05 = 2057 kg soil material	
	2160 - 2057 = 103 kg/m^3 cement	
	1.2 km × 6.7 m × 150 mm = 1200 m^3 × 103 =	124.2 Mg
Water	2160 kg/m^3 × 1200 m^3 × 8 percent =	208 000 L
Water for cure	8040 m^2 at 0.68 L/m^2 =	5470 L
AC	8040 m^2 × 88 kg/m^2 =	707.5 Mg
AC aggregate	8040 m^2 × 88 kg/m^2 × 95 percent =	672.1 Mg

Note: 1 kg/m^3 = 0.062 lb/ft^3; 1 kg = 2.205 lb; 1 km = 0.62 mile; 1 m = 3.28 ft; 1 mm = 0.039 in; 1 m^3 = 35.3 ft^3; 1 Mg = 1.1 tons; 1 L = 0.264 gal; 1 m^2 = 1.196 yd^2; 1 kg/m^2 = 0.2 lb/ft^2.

Table 3. Energy calculations for project 1.

Process	Calculation	Amount of Energy (MJ)
Rip by using motor grader with scarifier teeth	110 kW × 10 h × 70 percent[a] =	2 800
Pulverize with one single-transverse-shaft rotary mixer	220 kW × 10 h × 70 percent[a] =	5 500
Reshape with same motor grader		
Haul cement using six cement tankers (total)	6 × 160 km × 2 at 2.55 km/L × 38.65 MJ/L =	29 000
Cement	124 200 kg at 7.3 MJ/kg =	907 000
Mix with two rotary mixers, and water and mix	2 × 220 kW × 8 h × 70 percent[a] =	8 900
One water pump	2 kW × 3 h =	20
Two water trucks, 11 000 L each (208 000 L required), 19 round trips	19 × 3 km × 2 at 2.55 km/L × 38.65 MJ/L =	1 700
Compact and finish		
One 50-kW tamping roller	50 kW × 8 h × 70 percent[a] =	1 000
One 110-kW motor grader	110 kW × 8 h × 70 percent[a] =	2 200
One 40-kW self-propelled pneumatic-tired roller	40 kW × 8 h × 70 percent[a] =	800
One 11 000-L water truck, two round trips	2 × 3 km at 2.55 km/L × 38.65 MJ/L =	90
Cure		
One 6000-L bituminous distributor	160-km haul × 2 at 2.55 km/L × 38.65 MJ/L =	4 900
Heat and distribute	5470 L × 280 J/L =	0
Bituminous material	5470 L × 44 MJ/L =	241 000
Surface		
One rotary broom pulled by 40-kW tractor	40 kW × 4 h × 70 percent[a] =	400
Produce AC aggregate	672 Mg × 58 J/Mg =	39 000
Haul AC and aggregate to plant	707.5 Mg at 20 Mg/trip = 35 trips × 160 km × 2 at 2.55 km/L × 38.65 MJ/L =	170 000
Produce and place 38-mm AC surfacing	38 mm × 8040 m^2 × 6458 $MJ/(m^2/m)$ =	1 970 000
Total		3 384 000[b]

Note: 1 MJ = 947.8 Btu; 1 kW = 1.34 hp; 1 km = 0.62 mile; 1 L = 0.264 gal; 1 kg = 2.205 lb; 1 Mg = 1.1 tons; 1 mm = 0.039 in; 1 m^2 = 1.196 yd^2; 1 $MJ/(m^2/m)$ = 20.13 $Btu/(yd^2/in)$.

[a]Does not operate at full power all of working time.

[b]Total ÷ 8040 m^2 = 420 MJ/m^2.

Table 4. Calculations of materials required for project 1 alternative (highway patching and new AC overlay plus new shoulder and turnout gravel).

Material	Calculation	Quantity
AC	100 mm × 6.7 m × 1.2 km plus 5 percent for patching =	840 m^3
	840 m^3 × 2320 kg/m^3 =	1 940 000 kg
AC aggregate	840 m^3 × 2320 kg/m^3 × 95 percent =	1 851 000 kg
Gravel	100 mm × 2 sides × 1.2-m width × 1.2 km plus 10 percent for turnouts =	320 m^3
	320 m^3 × 2160 kg/m^3 =	691 000 kg

Note: 1 mm = 0.039 in; 1 m = 3.28 ft; 1 km = 0.62 mile; 1 m^3 = 1.308 yd^3; 1 kg/m^3 = 0.062 lb/ft^3.

Table 5. Energy calculations for project 1 alternative.

Process	Calculation	Amount of Energy (MJ)
Produce AC aggregate	1851 Mg × 58 MJ/Mg	107 000
Haul AC and aggregate to plant	1949 Mg at 20 Mg/trip = 98 trips × 160 km × 2 at 2.55 km/L × 38.65 MJ/L =	473 000
Produce and place AC concrete	840 m^3 × 6458 MJ/(m^2/m) =	5 425 000
Produce gravel	691 Mg × 58 MJ/Mg =	40 000
Haul	691 Mg at 20 Mg/trip = 35 trips × 160 km × 2 at 2.55 km/L × 38.65 MJ/L =	168 000
Place and shape with motor grader	110 kW × 10 h × 70 percent[a] =	2 800
Compact with vibratory roller	75 kW × 8 h × 70 percent[a] =	1 500
Total		6 217 000[b]

Note: 1 MJ = 947.8 Btu; 1 Mg = 1.1 tons; 1 km = 0.62 mile; 1 L = 0.264 gal; 1 m^3 = 1.308 yd^3; 1 MJ/(m^2/in) = 20.13 Btu/(yd^2/in); 1 kW = 1.34 hp.

[a]Does not operate at full power all of working time.
[b]Total ÷ 8040 m^2 = 770 MJ/m^2.

Table 6. Calculations of materials required for project 2 (recycling in place with cement on a city street with existing curb and gutter).

Material	Calculation	Quantity
Cement	2160 kg/m^3 density ÷ 1.05 = 2057 kg soil material	
	2160 - 2057 = 103 kg/m^3 cement	
	360 m × 6.7 m × 150 mm = 360 m^3 × 103 =	37 Mg
Water	2160 kg/m^3 × 360 m^3 × 8 percent =	63 m^3
Cure	2400 m^2 at 0.68 L/m^2 =	1600 L
AC	2400 m^2 × 88 kg/m^2 =	211 Mg
AC aggregate	2400 m^2 × 88 kg/m^2 × 95 percent =	201 Mg

Note: 1 kg/m^3 = 0.062 lb/ft^3; 1 kg = 2.204 lb; 1 m = 3.28 ft; 1 mm = 0.039 in; 1 m^3 = 1.308 yd^3; 1 Mg = 1.1 tons; 1 m^2 = 1.196 yd^2; 1 L = 0.264 gal; 1 kg/m^2 = 0.2 lb/ft^2.

Table 7. Energy calculations for project 2.

Process	Calculation	Amount of Energy (MJ)
Rip with motor grader with scarifier teeth	110 kW × 2 h × 70 percent[a] =	600
Pulverize with one single-transverse-shaft rotary mixer	220 kW × 4 h × 70 percent[a] =	2 200
Reshape with same motor grader		
Haul cement in two cement tankers (total)	2 × 40 km × 2 at 2.55 km/L × 38.65 MJ/L =	2 400
Cement	37 Mg at 7300 MJ/Mg	270 000
Mix with one rotary mixer, and water and mix	220 kW × 6 h × 70 percent[a] =	3 300
Two water trucks, 11 000 L each (63 000 L required), three round trips each	6 × 0.8 km × 2 at 2.55 km/L × 38.65 MJ/L	150
Compact and finish		
One 75-kW vibratory steel-wheel roller	75 kW × 6 h × 70 percent[a] =	1 100
One 110-kW motor grader	110 kW × 8 h × 70 percent[a] =	2 200
One 40-kW self-propelled pneumatic-tired roller	40 kW × 5 h × 70 percent[a] =	500
One 11 000-L water truck, two round trips	2 × 0.8 km × 2 at 2.55 km/L × 38.65 MJ/L =	50
Cure		
One 6000-L bituminous distributor	40-km haul × 2 at 2.55 km/L × 38.65 MJ/L =	1 200
Heat and distribute	1600 L × 280 J/L =	0
Bituminous material	1600 L × 44 MJ/L =	70 400
Surface		
One rotary broom pulled by 40-kW tire tractor	40 kW × 4 h × 70 percent[a] =	400
Produce AC aggregate	201 Mg × 58 MJ/Mg =	11 700
Haul AC and aggregate to plant	211 Mg at 20 Mg/trip = 10 trips × 40 km × 2 at 2.55 km/L × 38.65 MJ/L =	12 000
Produce and place 38-mm AC surfacing	38 mm × 2400 m^2 × 6458 MJ/(m^2/m) =	589 000
Total		967 000[b]

Note: 1 MJ = 947.8 Btu; 1 kW = 1.34 hp; 1 km = 0.62 mile; 1 L = 0.264 gal; 1 Mg = 1.1 tons; 1 mm = 0.039 in; 1 m^2 = 1.196 yd^2; 1 MJ/(m^2/m) = 20.13 Btu/(yd^2/in).

[a]Does not operate at full power all of working time.
[b]Total ÷ 2400 m^2 = 400 MJ/m^2.

Table 8. Calculations of materials required for project 2 alternative (removal and replacement of existing base and surface).

Material	Calculation	Quantity
Existing (removed)	215 mm thick × 6.7 m wide × 360 m long × 2080 kg/m^3 =	1079 Mg
New		
Crushed stone	140 mm thick × 6.7 m wide × 360 m long × 2160 kg/m^3 =	729 Mg
Prime	2400 m^2 × 0.68 L/m^2 =	1600 L
AC	75 mm thick × 2400 m^2 × 2320 kg/m^2 =	418 Mg
AC aggregate	418 Mg × 95 percent =	397 Mg

Note: 1 mm = 0.039 in; 1 m = 3.28 ft; 1 kg/m^3 = 0.062 lb/ft^3; 1 Mg = 1.1 tons; 1 m^2 = 1.196 yd^2; 1 L = 0.264 gal; 1 kg/m^2 = 0.2 lb/ft^2.

energy content of the material; and a haul of 0-16 km (0-10 miles) from plant to job.

8. Aggregate base in place = 2.96 MJ/(m^2/m) [5950 Btu/(yd^2/in)] (14, 15).

9. Aggregate production = 58.26 MJ/Mg (50 100 Btu/ton) (14, 15).

Table 1 gives the summary results of the calculations.

The original roadway of project 1 is an old gravel-base road 200 mm (8 in) thick and 6.7 m (22 ft) wide, with a 25-mm (1-in) surface treatment and some extensively patched areas. The project consists of rehabili-

Table 9. Energy calculations for project 2 alternative.

Process	Calculation	Amount of Energy (MJ)
Scarify and shape with motor grader	110 kW × 10 h × 70 percent[a] =	2 800
Load with skip loader	110 kW × 10 h × 70 percent[a] =	2 800
Truck haul	1079 Mg at 20 Mg/trip = 54 trips × 40 km × 2 at 2.55 km/L × 38.65 MJ/L =	65 500
Shape and reroll subgrade with vibratory roller	75 kW × 3 h × 70 percent[a] =	570
New base	2400 m^2 × 140 mm × 296 MJ/(m^2/m) =	99 500
Tack coat		
One 6000-L bituminous distributor	40-km haul × 2 at 2.55 km/L × 38.65 MJ/L =	1 200
Heat and distribute	1600 L × 280 J/L =	0
Bituminous material	1600 L × 44 MJ/L =	70 400
Produce AC aggregate	397 Mg × 58 MJ/Mg =	23 000
Haul AC and aggregate to plant	418 Mg at 20 Mg/trip = 21 trips × 40 km × 2 at 2.55 km/L × 38.65 MJ/L =	25 500
Produce and place AC surface	75 mm × 2400 m^2 × 6458 MJ/(m^2/m) =	1 162 000
Total		1 405 000[b]

Note: 1 MJ = 947.8 Btu; 1 kW = 1.34 hp; 1 Mg = 1.1 tons; 1 km = 0.62 mile; 1 L = 0.264 gal; 1 m^2 = 1.196 yd^2; 1 mm = 0.039 in; 1 MJ/(m^2/m) = 20.13 Btu/(yd^2/in).

[a]Does not operate at full power all of working time.
[b]Total ÷ 2400 m^2 = 590 MJ/m^2.

tating a 1.2-km (0.7-mile) long, 6.7-m-wide, 150-mm (6-in) thick area with soil-cement and applying a 38-mm (1.5-in) thick asphaltic concrete (AC) surface. The project area totals 8040 m^2 (9600 yd^2). The basic project requirements are given below (1 km = 0.62 mile; 1 kg/m^3 = 0.062 lb/ft^3):

Item	Quantity
Production	1.2 km/10-h day
Cement haul	160 km one way
Water haul	3 km one way
AC materials haul	160 km to plant
Cement content	5 percent by weight
Soil-cement density	2160 kg/m^3
Optimum moisture content	10 percent
Moisture in soil material	4 percent
Moisture added for evaporation	2 percent

Details of the calculations for project 1 are given in Tables 2 and 3. Calculations for the alternate solution to project 1—base patching and provision of a new 100-mm (4-in) thick overlay, a new shoulder, and turnout gravel—are given in Tables 4 and 5.

The original roadway of project 2 is an old gravel-base street with double bituminous treatment. The project consists of rehabilitating an area two blocks [360 m (1180 ft)] long, 6.7 m (22 ft) wide (face to face of gutter) with 150 mm (6 in) of soil-cement and a 38-mm (1.5-in) thick AC surface. The project area totals 2400 m^2 (2870 yd^2). The project requirements are given below (1 m = 3.3 ft; 1 km = 0.62 mile; 1 kg/m^3 = 0.062 lb/ft^3):

Item	Quantity
Production	360 m/7-h day
Cement haul	40 km one way
Water haul from city hydrant	0.8 km one way
Materials for AC	40-km haul to plant
Cement content	5 percent by weight
Soil-cement density	2160 kg/m^3
Optimum moisture content	10 percent
Moisture in soil material	4 percent
Moisture added for evaporation	2 percent

The calculations for project 2 are given in Tables 6 and 7. Calculations for the alternate solution—removal and replacement of the existing base and surface and use of a 75-mm (3-in) AC thickness on 140 mm (5.5 in) of crushed stone—are given in Tables 8 and 9.

It is interesting to note that in each case only a few items make up the major portion of the energy required and all the other items are minor in comparison. Certain assumptions must be made in any such calculations. Included in these comparisons is the energy content of asphalt, 44 000 MJ/m^3 (6.636 million Btu/bbl). This value does not include the energy required to drill for crude oil (including dry wells) or to transport it to the refinery (prorating the oil for its asphalt content). The total energy required could be considerable. The 7300 MJ/Mg (6.3 million Btu/ton) for cement includes all of the energy used in quarrying, transporting raw materials, and producing the cement at the plant.

SUMMARY

Cold recycling of failing flexible-base pavements with cement is usually undertaken when the objective is to improve pavement load-carrying capacity and surface conditions such as roughness and skid resistance. The obvious alternatives are (a) overlaying with a substantial thickness of asphaltic concrete, (b) reconstructing by hauling out and replacing the old base and surface, or (c) strengthening the existing material by cold recycling in place and placing a new surface. The examples cited in this paper illustrate the savings in energy and costs that can result from judicious use of the third alternative with cement as the binder. Such comparisons are possible only if all factors are considered.

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Abridgment

Characteristics and Performance of Low-Quality Aggregate in an Experimental Flexible Pavement

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As a major component of pavement structures, aggregates directly affect structural integrity and durability. Under traffic and in rigorous climatic conditions, the quality, properties, and behavior of aggregate play crucial roles in pavement performance and service life. Aggregate degradation, whether caused by chemical interactions such as moisture and freeze-thaw or mechanical causes during construction and/or traffic loading, contributes to pavement distress mechanisms. Complex interactions among load, stresses, strains, and climate, as well as how and where aggregates are used, may influence the extent of the effect of aggregate quality on pavement performance.

Assuming that high-quality materials yield better performance, it is desirable but not always feasible to use quality aggregates in pavement construction. In some regions, supplies of quality aggregate are scarce and the costs of transporting such materials are high. Many other areas have abundant local supplies of lower-quality aggregates and, in some cases, as with certain ash wastes from power plants, so-called inferior materials can be substituted in paving mixtures with little or no adverse effect on pavement performance.

These considerations have stimulated research into low-quality aggregates and their influence on pavement performance. In two such studies recently completed at Ohio State University (1, 2), the characteristics of local low-quality aggregates and their influence on the performance of flexible pavement were evaluated. The first study focused on identifying the mechanisms of aggregate degradation by means of a detailed laboratory evaluation that simulated climate and loading conditions. The second study evaluated the performance of such materials under service conditions.

AGGREGATE QUALITY AND DEGRADATION MECHANISMS

Selection of Materials

The aggregates used in these studies were acquired from local suppliers in central Ohio where the research facilities are located. First, local sources of aggregate were reviewed for the availability of materials, past performance history, and compliance with specifications. Five sources, designated plants 1 through 5, were selected to provide samples of no. 67, no. 57, and no. 8 coarse gravels and sands, which are defined as low quality by state specifications based on content of deleterious material (shale, chert, etc.). These aggregates were used in the laboratory evaluation program; comparable materials were later used to construct an experimental section of flexible pavement for field analysis and laboratory verification.

Laboratory Test Programs

To analyze the properties and performance of local low-quality aggregates, materials obtained from the five sources were subjected to test programs that included environmental simulation, moduli response, indirect tensile strength, and structural simulation of pavement response. Standard procedures were used to determine material properties. Aggregates were oven dried, sieved, and tested for sodium sulfate soundness loss and Los Angeles abrasion loss.

Aggregate quality was expressed in terms of the weight of deleterious materials retained on a 4.75-mm (no. 4) sieve rather than by weight of the total sample. Each aggregate was tested by the Ohio Department of Transportation (DOT) and Ohio State University (OSU) laboratories. The Ohio DOT laboratory used standard procedures. The OSU research team used a subjective but more stringent criterion:

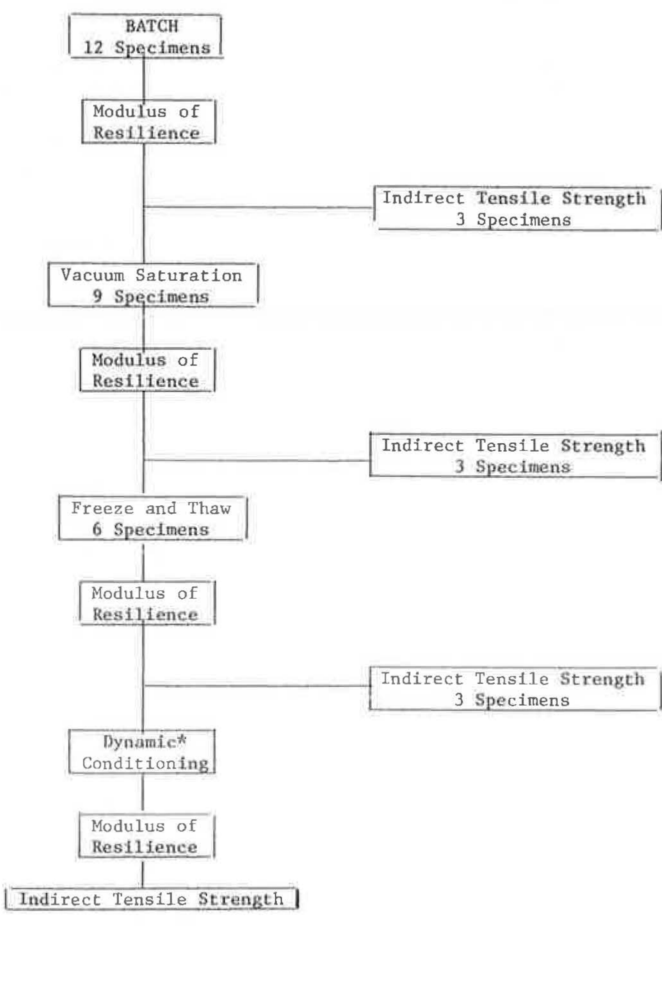
Pieces of aggregate were scraped against the bottom of a stainless-steel pan by using moderate finger pressure, and if the aggregate left a residue it was considered deleterious. The results of these two approaches were only marginally in agreement, but, since the study was largely concerned with qualitative rather than quantitative differences between aggregates, it was felt that the disparity in results would not seriously affect the validity of the findings.

After materials characterization and the assessment of aggregate quality, asphaltic mixes that met specifications for surface and base courses were designed, prepared, and evaluated.

The surface-course mixtures (one for each materials source) were prepared by using no. 8 coarse gravel and sand, one asphalt type (AC-20), and similar gradations. Since low-quality aggregates may be susceptible to moisture damage, the mixes were designed so that air voids were close to a minimum of 4 percent and the percentage of fines passing the 0.075-mm (no. 200) sieve was limited to 4 percent. At least 12 Marshall specimens were prepared for each mix type by using standard procedures. A minimum of three samples were prepared for each mix at three asphalt contents; these were analyzed for Marshall stability and flow, specific gravity of aggregate, loose mix and compacted specimens, and air voids and voids in mineral aggregate.

The base-course mixtures were prepared by using no. 67 coarse gravel and sand from each source, AC-20 asphalt cement, and an asphalt content of 4.7 percent by weight of total mix and were designed to have similar gradations. Two types of mixes were prepared for each source: (a) one set of samples in which the deleterious materials were removed and (b) another set in which the deleterious contents were as established by the OSU quality tests. These samples were prepared as follows.

Figure 1. Flowchart for environmental simulation testing program.



Sufficient material for 24 Marshall-size specimens from each source was oven dried and sieved, and the fractions were separated and stored. All deleterious materials (shale, chert, etc.) were removed from fractions larger than the percentage passing a 4.75-mm (no. 4) sieve and were stored. For each mix type, 12 specimens were prepared by recombining appropriate fractions to meet gradation limits and replacing deleterious materials with quality aggregate of comparable gradation. A second set of 12 specimens was prepared for each mix type in the same way except that deleterious materials were recombined in proportions established in the earlier tests of aggregate quality. Specific gravity and water absorption were determined for the coarse and fine aggregates and the loose mix by using standard procedures.

The durability of the base-course mixes was evaluated by using a specially designed environmental simulation program, which is shown in Figure 1. The program used a resilient-modulus approach; i.e., each batch of specimens was tested for resilient modulus and indirect tensile strength. As Figure 1 shows, samples were selected at random, subjected to vacuum saturation and freeze-thaw, and then retested for resilient modulus and/or tensile strength. The original program also called for dynamic load tests after environmental conditioning. But the effects of the freeze-thaw conditioning were so damaging that the specimens could not withstand dynamic loading, and this item was dropped from the program.

Laboratory Test Results

The gradation of the aggregates studied showed wide variation from one source to another. For example, a no. 57 gravel from plant 1 had 17.67 percent passing the 12.7-mm (0.5-in) sieve, whereas the same type of aggregate from plant 2 showed 51.2 percent passing the same sieve size. Similar variations were noted for gravels no. 8 and no. 67. Los Angeles abrasion tests showed similar plant-to-plant variations. The abrasion values for no. 57 gravel ranged from 24 to 40.2 percent. The values for no. 8 gravel were very similar, however—generally less than 30. Most aggregates did fall within specification limits. These aggregates also met limits for soundness loss but again showed variations according to source. No. 8 gravel from plants 1 and 3 had the highest losses (18.4 and 18.8 percent, respectively); for no. 67 gravel, plant 4 yielded 15.2 percent loss whereas plant 5 yielded 17.5 percent loss; and for no. 57 gravels and sands, plants 1 and 5 showed the highest soundness losses. The contents of deleterious material obtained by the Ohio DOT and OSU laboratories are given in Table 1. Again, the agreement in the results was only marginal and, although it is felt that the findings of the study are valid, this disparity would have to be considered in developing aggregate acceptance standards.

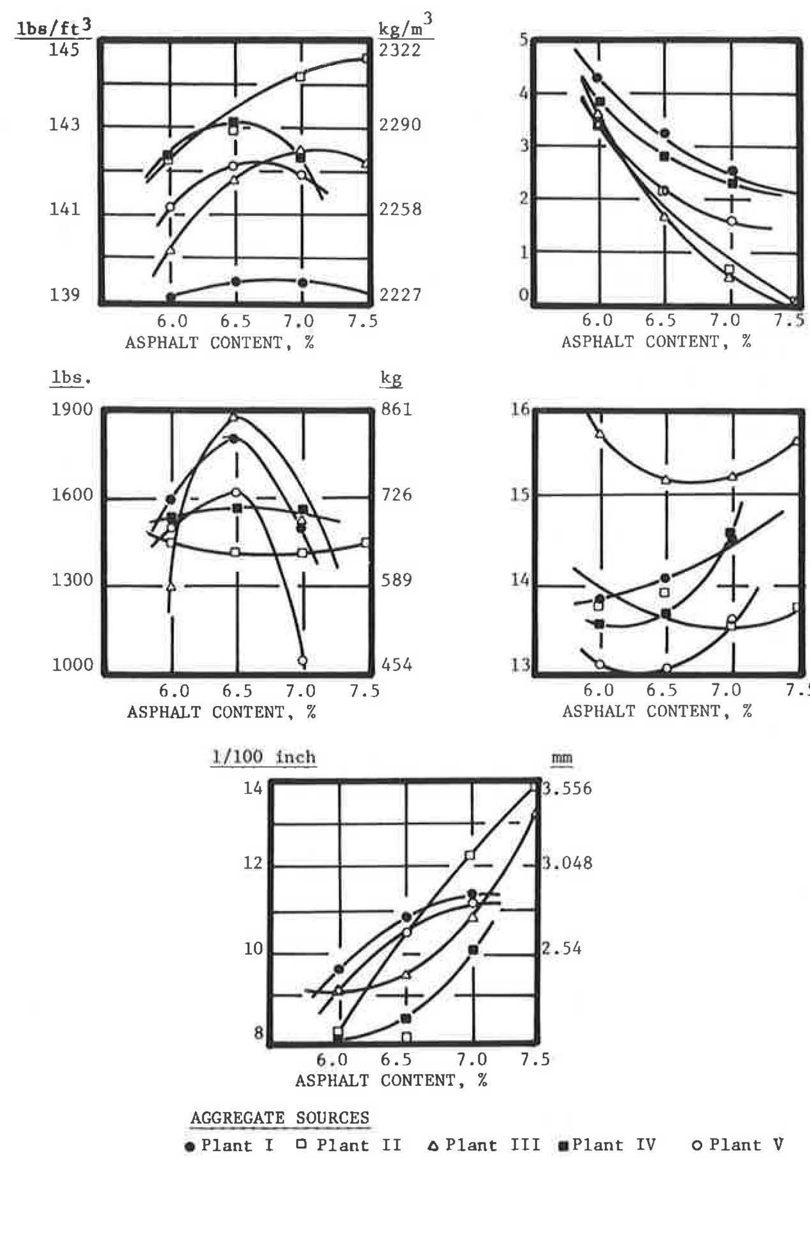
The results of Marshall tests on surface-course mix samples are shown in Figure 2. The mix stability and density of material from plant 2 were relatively insensitive to asphalt content, as was the stability of plant 4 mixes. At optimum asphalt contents, the air voids contents of all five mixes were very close to the lower design limit. To meet air voids requirements, the optimum asphalt contents would have to be reduced,

Table 1. Deleterious material in low-quality aggregates.

		Deleterious Content (%)					
		Shale and Shaly Material		Chert and Other Material		Total	
Type of Aggregate	Material Source (plant no.)	Ohio DOT[a]	OSU[b]	Ohio DOT[a]	OSU[b]	Ohio DOT[a]	OSU[b]
No. 8 gravel	1	2.8	1.70	0.2	2.10	3.0	3.80
	2	2.6	0.48	0.2	1.05	2.8	1.53
	3	1.3	4.02	0.1	0.54	1.4	4.56
	4	NA	2.57	NA	5.16	NA	7.73
	5	NA	4.08	NA	3.09	NA	7.17
No. 57 gravel	1	2.0	3.39	0.3	4.17	2.3	7.56
	2	0.5	0.32	0.1	3.60	0.6	3.92
	3	2.4	3.15	0.1	3.67	2.5	6.82
	3	2.1	NA	0.1	NA	2.2	NA
	3	3.5	NA	0.2	NA	3.7	NA
	4	1.1	0.71	0.1	2.54	1.2	3.25
	5	3.1	1.89	0.2	4.84	3.3	6.73
	5	3.6	NA	0.1	NA	3.7	NA
	5	2.4	NA	0.2	NA	2.6	NA
No. 67 gravel	1	2.0	5.94	Trace	10.42	2.0	16.36
	2	0.9	1.56	0.3	4.09	1.2	5.65
	3	3.6	5.81	0.1	6.10	3.7	11.91
	3	3.0	NA	0.1	NA	3.1	NA
	4	1.1	3.23	0.1	7.31	1.2	10.54
	5	1.9	5.63	0.3	9.77	2.2	15.40
	5	2.1	NA	0.1	NA	2.2	NA
	5	1.6	NA	0.1	NA	1.7	NA

Note: NA = data not available.
[a] Results obtained by Bureau of Testing, Ohio Department of Transportation.
[b] Results obtained by Materials Research Laboratory, Ohio State University.

Figure 2. Marshall mix design curves for surface-course mixtures that contain low-quality aggregate.



thereby reducing mix stability and density. It is probable that degradation and fines production during mix preparation and compaction contributed to the low observed values of air voids and flow. A more detailed investigation might yield data that would lead to a more optimal design. While this study was in progress, however, the state transportation department decided to prohibit the use of low-quality aggregates in surface-course mixtures, and subsequent research efforts focused on base-course mixture performance.

The results of the initial tests of resilient modulus and indirect tensile strength performed on the base-course mixtures are given in detail elsewhere (2). The results indicate that aggregate source and the inclusion or removal of deleterious materials had no significant effect, although the tensile strengths of specimens from which deleterious materials were removed were slightly higher than those of specimens that contained such materials. From a pavement designer's viewpoint, the differences in moduli and strength between the two types of specimens are not significant.

The results of tests on saturated samples led to a similar conclusion. There were no significant differences between materials from different sources, and the effects of including or excluding deleterious materials were only minimal. It was concluded that during initial stages of pavement life, before the structure experiences severe environmental exposure, the moduli and strength are, for all practical purposes, unaffected by the presence or absence of deleterious material in the coarse aggregate.

However, the effects of freeze-thaw conditioning were quite significant and resulted in substantial reduction of moduli and strength for all mixes, regardless of source or deleterious content. A review of data on field cores taken from pavements built with conventional materials showed substantially higher values before and after saturation and freeze-thaw—only a 25-50 percent loss in moduli and strength after environmental conditioning—whereas the samples tested in this study experienced a two-thirds reduction in moduli and strength after conditioning. As mentioned previously, the damage to specimens subjected to freeze-thaw conditioning was so severe that afterwards the samples could not withstand dynamic loading.

Since the removal of deleterious material from coarse aggregate does not ensure that the fine aggregate is satisfactory material, it was considered that deleterious sand might have contributed to the observed poor durability of these mixtures. Accordingly, a mixture in which plant 3 coarse aggregate was used and crushed limestone sand was substituted for the plant 3 sand was prepared and evaluated for resilient modulus. The experimental mixture that contained plant 3 sand and coarse aggregate and from which deleterious material was removed showed a modulus ratio of 0.22; the ratio for the mix that contained plant 3 nondeleterious coarse aggregate and crushed limestone sand was 0.44, nearly double the mix response. This suggests that deleterious material in the fine aggregate may also be detrimental to mixture performance and should be investigated further and be considered in developing specifications.

EXPERIMENTAL FLEXIBLE PAVEMENT PROJECT

Selection of Materials and Site

After the completion of the study described above, an experimental flexible pavement in which low-quality aggregates were used in the base course was designed and constructed near Newark, Ohio, and field and laboratory analyses of its performance were conducted. The Ohio DOT laboratory conducted materials characterization and quality tests on a number of aggregates and selected three classes of experimental mixtures:

1. Type A material used an aggregate that contained less than 2.5 percent deleterious material (using Ohio DOT standard procedures).
2. Type B aggregate had 2.5-3.5 percent deleterious material.
3. Type C aggregate had 3.5-6.0 percent deleterious material.

These three aggregates were incorporated into the base mixes and placed at various stations throughout the project.

Test Program

Dynaflect dynamic deflection measurements were taken on the project subgrade before construction, on the completed base course, and on the completed pavement after the placement of a standard asphaltic surface course. Field cores were also taken—three for each mix type—from the finished pavement after one year's service. The cores were tested for dynamic modulus and then sawed into three Marshall-size sections. The lower sections were tested for tensile strength while the remaining sections underwent the environmental-conditioning program of saturation and freeze-thaw used in the earlier study (Figure 1).

Bulk samples of each class of uncompacted paving mixes were obtained at the time of construction. Seventy-eight Marshall-size specimens were prepared by using standard procedures. After determination of specimen density, the samples were tested for resilient modulus and indirect tensile strength, and a number of randomly chosen samples underwent the same environmental-conditioning program used before.

Test Results

Initial deflections on the subgrade showed considerable variation in maximum deflection and surface curvature index, but measurements on the completed pavement showed a very uniform response (which indicated that subgrade support had improved and that the structure was uniform in thickness and stiffness characteristics). There were no significant differences in deflections on the sections constructed with different classes of materials; this suggested that the initial performance of all three types of mixtures was quite similar and met initial design requirements.

The results of initial resilient-modulus tests on bulk material samples showed unrealistically high moduli: 4248-10 324 MPa (616 000-1 497 000 lbf/in^2). It was felt that the presence of large aggregates in these specimens had distorted the stress field in these tests and yielded the higher moduli. Therefore, several cylinders were prepared from the bulk material, tested for compressive dynamic modulus, and sawed into Marshall-size samples, and the middle section was analyzed for resilient modulus. The dynamic moduli of these samples were within the expected range, but the resilient moduli (obtained from samples with two cut faces, which somewhat reduced the aggregate size) were nearly double the range of the dynamic moduli.

The average resilient moduli for field cores are

Table 2. Effect of environmental conditioning on resilient modulus and indirect tensile strength.

Mix Type	Sample Type	Resilient Modulus (MPa) Unsaturated	Resilient Modulus (MPa) After Freeze-Thaw	Indirect Tensile Strength (MPa) Unsaturated	Indirect Tensile Strength (MPa) After Freeze-Thaw	Ratio (before to after freeze-thaw) Resilient Modulus	Ratio (before to after freeze-thaw) Indirect Tensile Strength
A	Laboratory	5261	5379	2896	3841	0.55	0.71
	Field cores	2759	4069	2103	2896	0.76	0.71
B	Laboratory	7723	7186	3793	4303	0.47	0.60
	Field cores	2310	3855	1655	2262	0.72	0.59
C	Laboratory	9242	6131	4745	3821	0.51	0.62
	Field cores	2055	3614	1414	1365	0.69	0.38

Note: 1 MPa = 145 lbf/in^2.

summarized below (1 MPa = 145 lbf/in^2):

Mix Type	Sample No.	Dynamic Modulus (MPa)	Modulus of Resilience (MPa)
A	1	2359	2393
	2	2910	3689
	3	2703	2186
Avg		2655	2758
B	1	2255	2207
	2	2062	2407
Avg		2179	2310
C	1	1952	2000
	2	2076	2110
Avg		2014	2055

From the design viewpoint, the variations in moduli among types A, B, and C mixes are not highly significant. Greater variations can be found among standard mixes with similar aggregate qualities. On the basis of initial performance parameters, it was concluded that there were no significant variations among the three aggregate types that were caused by deleterious material content.

The effects of environmental conditioning on both bulk samples and field cores are summarized in Table 2. As in the earlier study, environmental conditioning (which was much more severe than actual conditions likely to be encountered in Ohio) significantly reduced mixture stiffness and strength. The degree of deterioration in strength increased with increases in the amounts of deleterious materials incorporated into the mixture. Accordingly, type C mixes showed the greatest reduction in moduli and tensile strength under environmental conditioning. Type A mixtures, which had the lowest deleterious content, experienced as much deterioration as type B mixes, which had an intermediate deleterious content. Again, it was suspected that the deleterious content of the fine aggregate might have contributed to the observed performance.

SUMMARY AND CONCLUSIONS

The results of the studies reported in this paper suggest that the presence of deleterious material in either fine or coarse aggregate may be detrimental to pavement performance. Laboratory and field data indicate that, although these materials may perform satisfactorily in the initial stages of pavement life, exposure to saturation and freeze-thaw conditions can result in substantial deterioration in mixture strength and stiffness. It is concluded that such materials should not be used in surface-course mixtures where climatic exposure is greatest. Such materials could be used in base courses if adjustments are made in layer equivalencies to account for lower durability characteristics.

The data also suggest that the presence of deleterious materials in coarse aggregate is not solely responsible for the poor performance exhibited by these mixtures. Deleterious fine aggregate may also adversely affect the durability of mixtures prepared with low-quality aggregates. This possibility is currently being investigated at Ohio State University.

ACKNOWLEDGMENT

This abridgment discusses the results of two investigations conducted by the Materials Research Laboratory, Engineering Experiment Station, Ohio State University. Financial support for these projects was provided by the Federal Highway Administration, U.S. Department of Transportation, and by the Ohio Department of Transportation. The studies were conducted in cooperation with the Ohio Department of Transportation. We express sincere appreciation to Leon O. Talbert, Harry Marshall, John Dixon, Stuart Schwotzer, Jorge Villacres, Gary Bokowski, Cheryl Helm, and other members of the Ohio DOT and OSU research staffs for their invaluable assistance.

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Performance of Incinerator Residue in a Bituminous Base

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A study of the use of incinerator residue as an aggregate in bituminous base (termed littercrete) is reported. Test sections consisting of the experimental hot-mixed littercrete base and a conventional hot-mixed asphaltic concrete base (termed blackbase control) and topped with a conventional surface were placed on a city street in Houston, Texas. The results of observations and core tests during the first three years of in-service performance are presented. Results of the laboratory and field evaluations show that the littercrete section is performing extremely well, almost identically with the conventional blackbase control section. The only distress that has occurred in both sections is minor cracking, which is limited to the conventional wearing surface and has not progressed into the bases.

The construction and maintenance of the ever-expanding U.S. highway system have created an increasing demand for quality construction materials. In terms of materials production, the strong lead the United States has enjoyed over other industrialized nations since the early 1950s has steadily diminished (1).

There is an abundant supply of source materials for the production of quality aggregates for the foreseeable future (2), but the distribution of these sources does not always coincide with the location of need. This has increased the cost and energy consumed in constructing transportation facilities. Attempts have been made to develop supplemental aggregate sources economically and realistically in an effort to fill localized demands. One such aggregate source that is under investigation is incinerator residue obtained from the burning of municipal solid wastes.

The feasibility of using solid wastes in highway construction and maintenance has been investigated by the Texas Transportation Institute (3). Several potential uses of solid wastes were studied, including its use as an aggregate replacement for base, subbase, and stabilized materials. This paper discusses the three-year in-service evaluation of municipal solid waste taken directly from an incinerator and used as an aggregate replacement in bituminous-base construction on a city street.

DESCRIPTION OF STUDY

The objective of this study was to determine the usefulness of municipal residue as an aggregate in bituminous-base construction (here called littercrete). The work consisted of construction, control, and evaluation of approximately 60 m (200 ft) of roadway on Bingle Road at the intersection of the old Hempstead Highway in Houston, Texas (see Figure 1). In the test section, which was contracted by the city of Houston and constructed in 1974 by Brown and Root, Inc., municipal incinerator residue from Houston's Holmes Road incinerator plant was used as the aggregate in a 152-cm (6-in) thick bituminous base. This was covered by approximately 3.8 cm (1.5 in) of conventional hot-mixed asphaltic concrete (AC) wearing surface. For purposes of comparison, a conventional aggregate bituminous base (called the blackbase control section) was constructed adjacent to the test section and evaluated. The pavement has been evaluated for three years and will be evaluated yearly for another three years.

ASPHALT PROPERTIES

The asphalt used throughout this study was an AC-20-grade asphalt cement refined at the Exxon Corporation's Baytown, Texas, refinery. Standard asphalt tests and asphalt extractions were performed on laboratory and field samples from both the littercrete and control sections during the three-year evaluation period (see Table 1). During the period of evaluation, the viscosity of the recovered asphalt for the control section has increased and the viscosity of the littercrete has decreased slightly. Although the penetration of the recovered asphalt samples fluctuates, it shows a definite tendency to decrease. "Hardening" of the asphalt is normally expected because of oxidation and other forces. The data given in Table 1 show that the asphalt in each section is performing essentially the same in service.

AGGREGATE PROPERTIES

The residue from the Holmes Road incinerator plant was passed through a 25-mm (1-in) screen, and the minus-25-mm portion was used in the study. Approximate composition of the incinerator residue from magnetic and visual separation on the plus-5-mm (no. 4) sieve material is given below:

Item	Amount
Moisture, stockpile (%)	15
Approximate composition (%)	
Plus-5-mm sieve material	54
Ferrous metals	5
Nonferrous metals	2
Wood, paper, charcoal	1
Ceramics	1
Glass	45
Minus-5-mm sieve material	46
Loss on ignition (%)	5
Apparent specific gravity (avg)	2.13

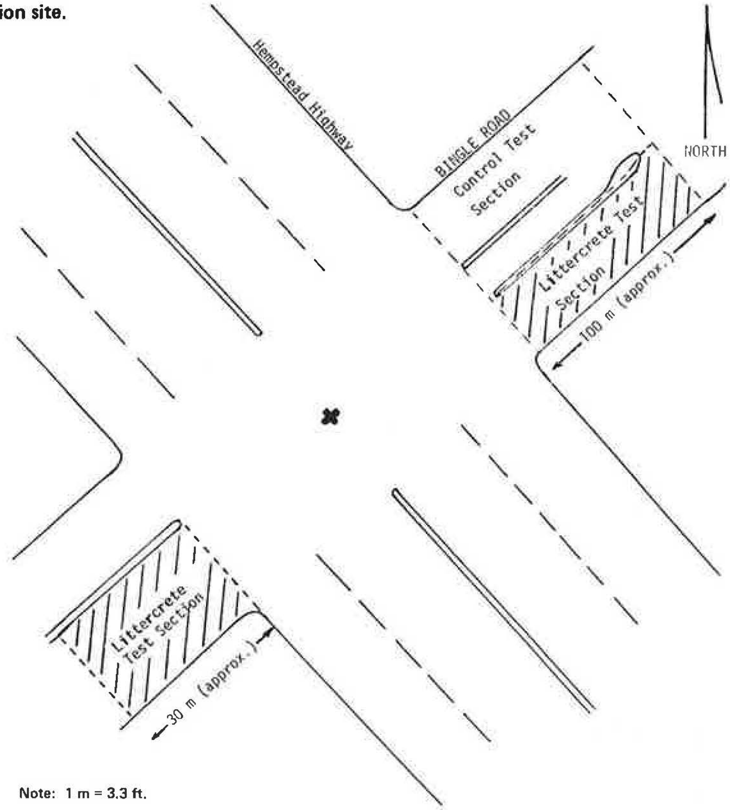
Loss on ignition was determined by firing the residue in a muffle furnace for 15 min at 1040°C (1850°F). With a composition of 45 percent glass, hydrated lime [$Ca(OH)_2$] was added at a rate of 2 percent by dry weight of aggregate to prevent the asphalt stripping from the glass portion. The Holmes Road incinerator residue appears to be similar in composition to the residue of other incinerators in the United States examined by the U.S. Bureau of Mines (4).

A typical sample of this aggregate was compared with a Texas class AA, type C grading specification (see Figure 2) and was found to be within specifications except in the range of minus 0.6 to minus 0.075 (no. 30 to no. 200). The aggregate was found to contain appreciable fines, which were not noticeable in the dry sieve analysis. When a wet sieve analysis was conducted, the aggregate was found to meet the Texas specification.

MIX PROPORTIONS

Trial mixes that contained 9, 10, and 11 percent asphalt by total weight of mix were mixed in the laboratory, compacted, and tested to determine the optimum asphalt

Figure 1. Test section site.



Note: 1 m = 3.3 ft.

Table 1. Summary of asphalt properties.

Item	Section	Penetration 4°C	Penetration 25°C	Viscosity 25°C (kPa·s)	Viscosity 60°C (Pa·s)	Viscosity 135°C (Pa·s)	Softening Point (°C)	Specific Gravity at 25°C	Recovered Asphalt Content[a]
Exxon AC-20 asphalt		16	54	230	260.6	0.37	51	1.02	
Laboratory-compacted	Littercrete	29	45	420	498.2	0.44	54	1.02	9.9
cores	Control	19	54	450	291.6	0.39	53	1.02	4.6
Field-compacted cores									
New[b]	Littercrete	10	27	1240	-	0.61	58	-	9.2
	Control	18	41	320	-	0.43	70	-	5.3
Six months[c]	Littercrete	11	32	700	-	0.48	57	-	10.8
	Control	15	37	700	-	0.45	55	-	5.3
One year[c]	Littercrete	10	28	1100	-	0.55	57	-	9.7
	Control	8	24	2440	-	0.66	62	-	4.9
Two years[c]	Littercrete	3	22	-	-	0.72	-	-	9.4
	Control	13	56	-	-	0.48	-	-	5.2
Three years[c]	Littercrete	6	30	1050	-	0.58	59	-	9.5
	Control	8	33	962	-	0.54	56	-	5.1

Note: $t°C = (t°F - 32)/1.8$; 1 Pa·s = 10 poises.
[a] Mix basis, 9 percent by weight of core samples taken from approximately the same place in the road.
[b] Taken immediately after construction.
[c] Time in service when cores were taken.

content of the mix. A discussion of the mix procedure, sample preparation, and test procedures used in the optimum mix design is given in the first study report (5). The seemingly high asphalt contents are attributable primarily to the low specific gravity of the aggregate, which averaged 2.13. Based on the results obtained, the recommended optimum mix design consisted of the following:

Material	Percentage by Total Volume	Percentage by Total Weight of Mix	Equivalent Percentage by Total Weight of Mix
Incinerator residue	80.9	89.0	91.0
AC-20 asphalt	17.4	9.0	7.4
Hydrated lime	1.7	2.0	1.6

Equivalent percentage by total weight of mix assumes a specific gravity of 2.65.

CONSTRUCTION SEQUENCE

Construction of the project began in July 1974. Approximately 460 m^3 (600 yd^3) of incinerator residue

Figure 2. Comparison of incinerator residue and Texas class A, type C gradation.

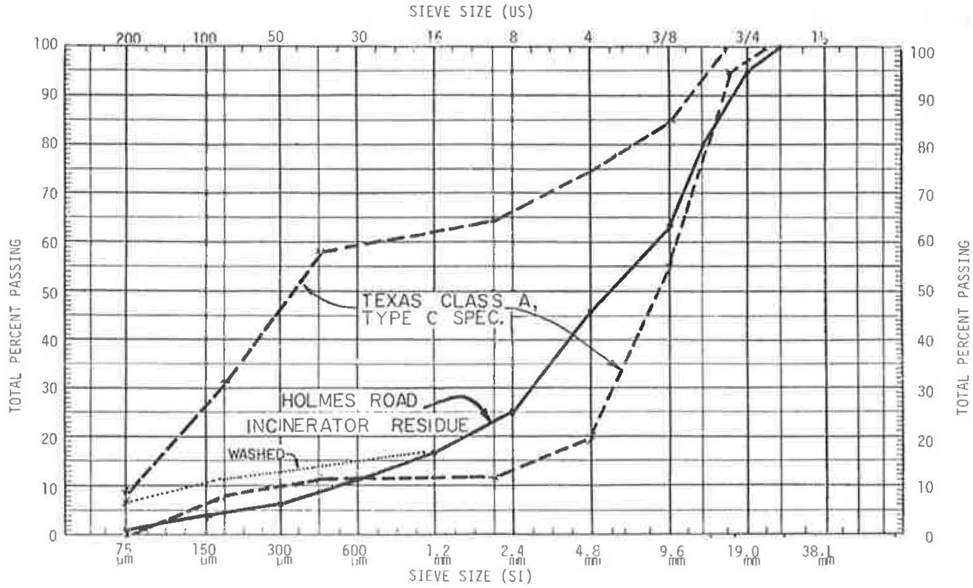


Figure 3. Longitudinal crack in littercrete section.

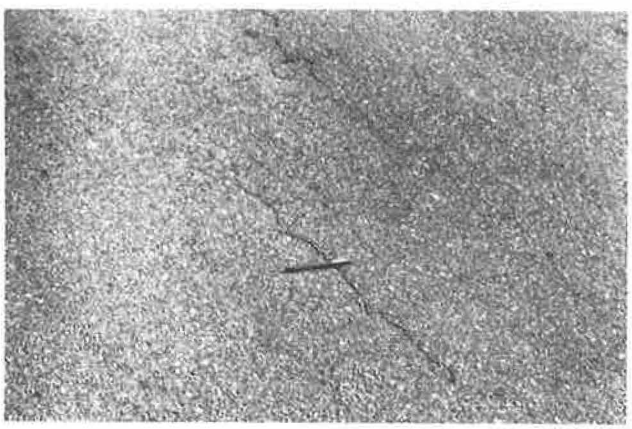


Figure 4. Cracked wearing surface.

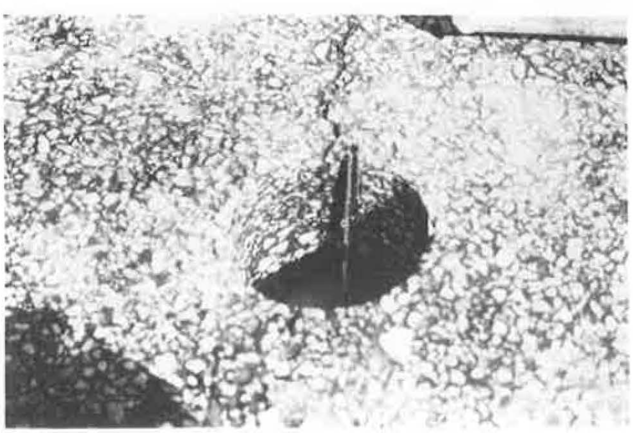
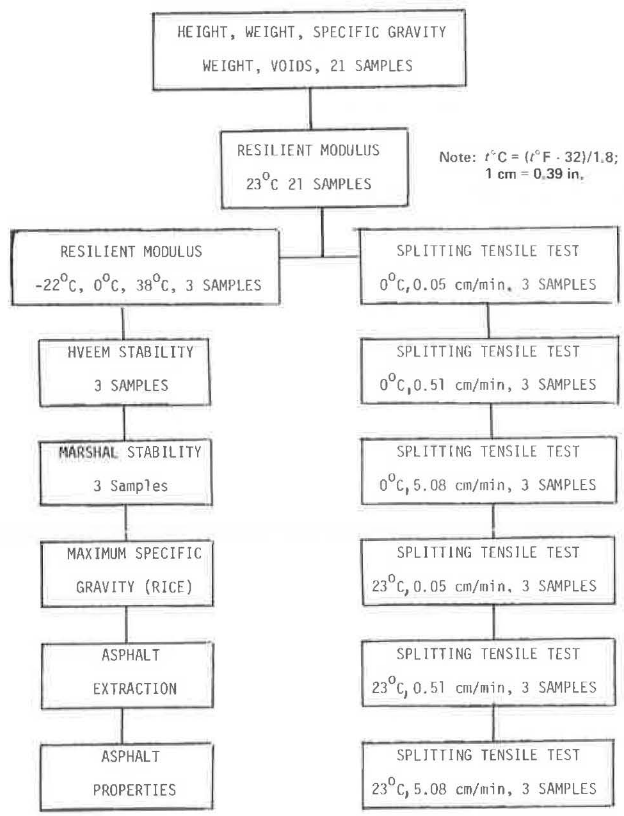


Figure 5. Testing program for core samples.



was stockpiled at the Holmes Road incinerator plant. The residue was spread into a 0.3-m (1-ft) thick lift, and bulk, hydrated lime was added to the residue at the approximate rate of 2 percent lime by dry weight of the material (to prevent asphalt stripping from the glass in the residue). Water was added, and the residue and lime were mixed to a depth of approximately 0.2 m (8 in).

The residue (with lime) was loaded into end dump trucks, hauled to an asphalt plant, and processed through a conventional pug mill at 150°C (300°F). The material was then transported to the construction site and placed into a conventional AC laydown machine. The material was placed in two 80-mm (3-in) lifts over 150 mm (6 in) of lime-stabilized subgrade, and each

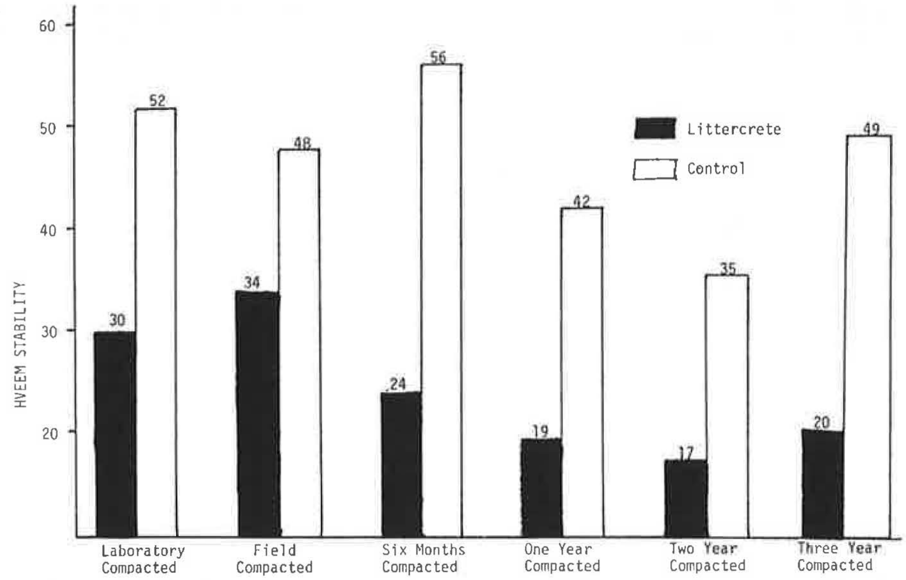


Figure 6. Hveem stability values for various times in service.

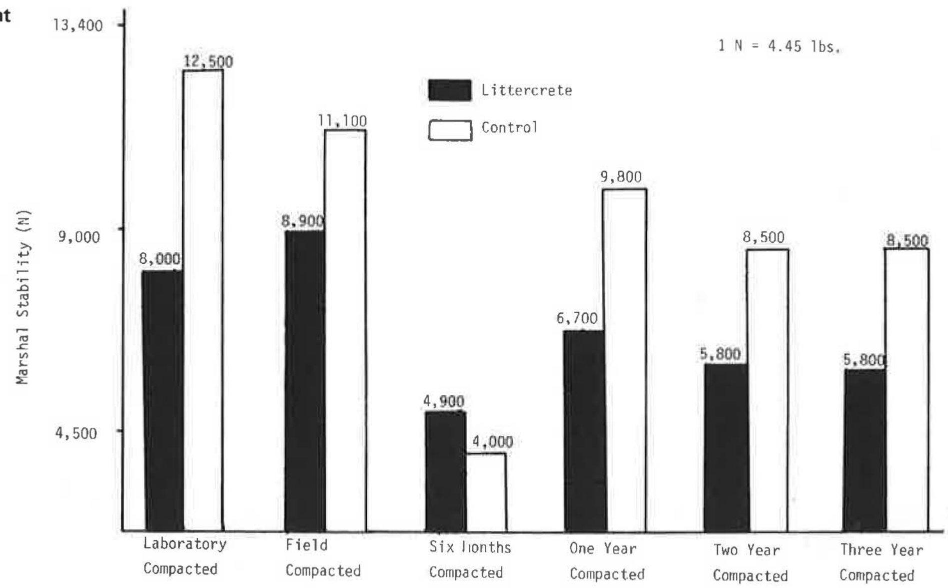


Figure 7. Marshall stability values at various times in service.

lift was compacted by using tandem, steel-wheel rollers and pneumatic rollers. Finally, a 40-mm (1.5-in) conventional AC wearing surface was placed over the base material. After the pavement had cooled sufficiently, base cores 100 mm (4 in) in diameter were taken for testing. Coring was repeated at six months, and at one, two, and three years.

A control section placed adjacent to the littercrete section consisted of 40-mm (1.5-in) maximum-size limestone aggregate, natural sand fine aggregate, and 6 percent asphalt (by total weight of the mix). The design used is typical of pavement systems for the city of Houston.

FIELD OBSERVATIONS

Visual examinations of the pavement sections were taken monthly for the first six months in service and at three-month intervals thereafter. These visual examinations were performed to observe the two pavement sections for evidence of cracking or noticeable failure. After two years in service, small longitudinal cracks 5-6 m (0.19-0.25 in) in size were noticed in both the littercrete and control sections (see Figure 3). During the third year of observation, it was noticed that, although there were slightly more cracks, the cracks had not increased in size. At this time a core sample was taken from the cracked section to determine the depth of penetration of the cracks.

The cracks were found to be in the wearing surface only (see Figure 4) and are believed to have been caused by thermal forces acting on the wearing surface. At this point it is difficult to determine conclusively the reason for this type of cracking. An intensive study of this question is scheduled for the next coring operation. One possible answer is that, although both base courses are deflecting under repeated loading, the wearing surface has become "brittle" and is forming cracks.

Although there are small cracks in the wearing surface, field observations indicate that the littercrete is carrying traffic and withstanding environmental effects very well. No rutting or shoving has occurred.

Figure 8. Ultimate strength versus unit deformation rate for one-year field-compacted littercrete and control sections.

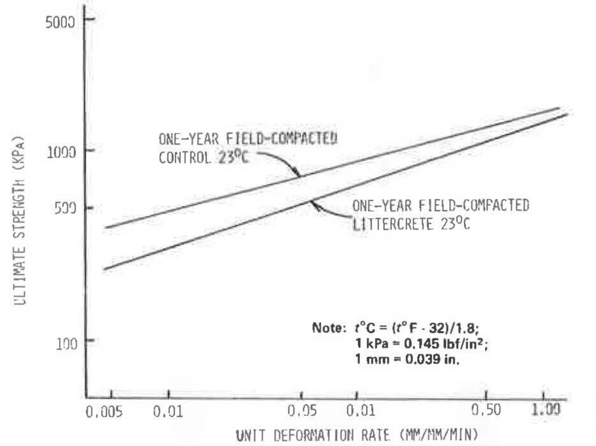
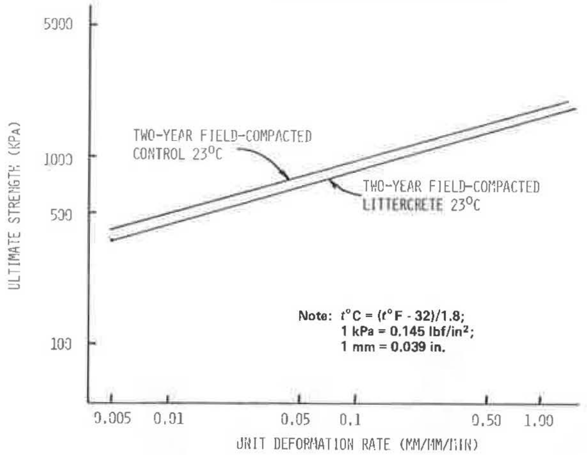


Figure 9. Ultimate strength versus unit deformation rate for two-year field-compacted littercrete and control sections.



The results of a traffic count taken for the test section in 1973 are given below:

Type of Vehicle	Vehicles per Day: Inbound Number	Inbound Percent	Outbound Number	Outbound Percent
Automobile	4362	80	3677	76
Pickup	984	17	1027	21
Single-unit truck	94	02	84	02
Heavy truck, more than two axles or two tires on rear	26	01	39	01
Total	5466	100	4827	100

The number of 80-kN [18 000-lbf (18-kip)] equivalent axle loads per day that the test section would experience in 1973 is 170, and the yearly average would be 62 100. Of the total traffic mix, it was found that approximately 78 percent were automobiles, 19 percent were pickups, 2 percent were single-unit trucks, and 1 percent were heavy trucks. If a growth rate of 5 percent/year is assumed, the test section would have been subjected to about two-hundred-thousand 80-kN equivalent axle loads in the three-year study period.

Figure 10. Ultimate strength versus unit deformation rate for three-year field-compacted littercrete and control sections.

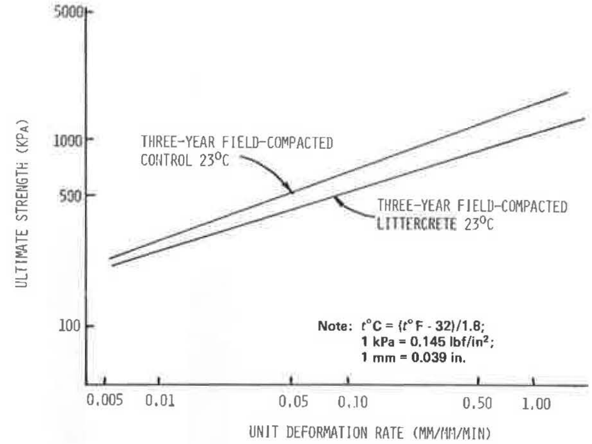
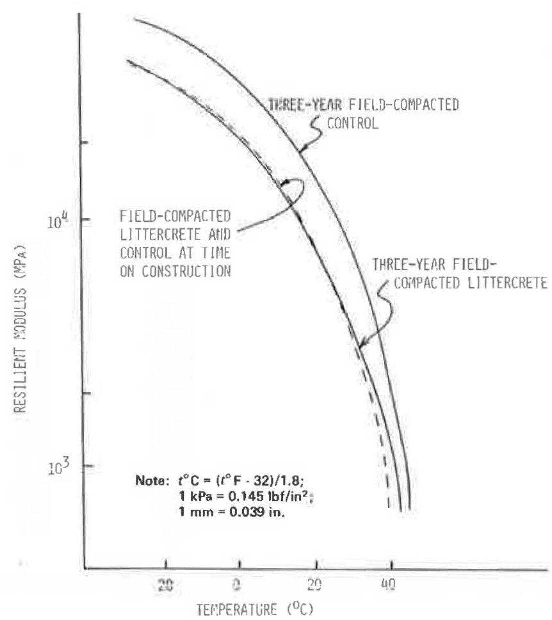


Figure 11. Resilient modulus versus temperature for field-compacted and three-year field-compacted samples.



LABORATORY OBSERVATIONS

Laboratory investigations conducted in this study were performed on the littercrete and the adjacent control test section to (a) compare the littercrete and control mix properties and (b) quantify the performance of the two pavement sections with time.

Field cores were drilled by using a trailer-mounted rig and a 10-cm (4-in) diamond-tipped core barrel. The testing program followed on the cores is shown in Figure 5. The procedure for the different tests performed and the means of data reduction involved are given in the first study report (5). Also given in the first report are the data used in the graphs and tables involving laboratory, field, and six-month field-compacted samples.

A summary of the results of the tests performed is given in the following sections of this paper. Whenever possible, the methods used in reducing and presenting the data are similar to the methods used by other investigators and indicated in the literature.

Marshall and Hveem Stability

Stability tests performed on the test samples consisted of the Hveem stabilometer test and the Marshall test (5-7). Summaries of these data are given in Figures 6 and 7. The values for the six-month field-compacted Marshall stability samples are very low in relation to those for the other samples; this could be caused by testing of an atypical sample. A Hveem stability of 30 (8) is the minimum criterion for the coarse-aggregate type of hot plant mixes in the state of Texas for AC-20 and AC-10. When they were placed, both mixes satisfied the Hveem criteria for base courses. The minimum Marshall stability criterion recommended by the Asphalt Institute, 3.3 kN (750 lbf) (8), was easily met by both the littercrete and blackbase control mixes when placed.

With time in service, the Marshall stability of the littercrete samples decreased from 8 to 6 kN (1820 to 1350 lbf) (a 26 percent reduction), while that of the control samples decreased from 12.6 to 8.6 kN (2830 to 1940 lbf) (a 31 percent reduction). The meaning of these reductions is not clear. Certainly some reduction in stability is to be expected. Still, the Marshall values are quite high, exceeding the recommended as-constructed minimum. On the other hand, the Hveem values for the littercrete are somewhat low. The literature yields no clues as to whether a reduction in Hveem values indicates future problems with the littercrete.

Splitting Tensile Test

Graphs of the ultimate tensile strength of the various samples versus the unit deformation rate are shown in Figures 8-10 for specimens that have been in service for one, two, and three years (5,9). These figures indicate that the two mixes have essentially the same ultimate splitting tensile strength for all three ages, which means the littercrete is holding up essentially the same as the blackbase control section in terms of load-carrying capacity.

Schmidt Test

Resilient modulus versus temperature relations for three years in service is shown in Figure 11 (5). The moduli of the three-year field-compacted samples were compared with resilient modulus at the time of construction. It can be seen that the samples tend to have a slightly greater value of resilient modulus. This would be expected, taking time and traffic into consideration. After three years in service (Figure 11), the littercrete is performing essentially the same as it did when it was placed, whereas the control section has somewhat "hardened".

CONCLUSIONS

The following conclusions can be stated:

1. Based on evaluation of recovered asphalt samples, the asphalt in both the littercrete and the control section is performing essentially the same in service.
2. Based on visual examinations of the pavement surface and cores as well as the densities of the cores, the littercrete section, after three years in service, is performing as well as the blackbase control section. No rutting or shoving has occurred.
3. Laboratory results of strength, as determined by Marshall stability values and splitting tensile strength, verify the observation that the littercrete is performing as well as the blackbase control section.
4. Laboratory results for Hveem stability indicate that the littercrete section has lost more stability than the blackbase control section. The implications of these results are not known.
5. Laboratory results of the resilient-modulus test indicate that after three years in service the littercrete is performing essentially the same as it did when it was constructed but the control section has "hardened".

As expected, some conflicting results were obtained. The Hveem stability values for the littercrete dropped significantly with time. What this means is not yet clear. Marshall stability values are still quite good. Splitting tensile strength and resilient modulus look excellent. And, of course, visual examinations prove that the littercrete section is performing essentially the same as the blackbase control section. So the prognosis looks good. After three years in service, the test results indicate that littercrete can perform in a manner acceptable by today's standards for asphaltic concrete pavements.

ACKNOWLEDGMENT

We wish to acknowledge the assistance of the city of Houston and also the Federal Highway Administration, U.S. Department of Transportation, for their sponsorship of the study on which this paper is based.

The contents of this report reflect our views, and we are responsible for the facts and the accuracy of the data presented. The contents do not necessarily reflect the official views or policies of the U.S. Department of Transportation. This report does not constitute a standard, specification, or regulation.

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Incinerator Residue as Aggregate for Hot-Mix Asphalt Base Course

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A laboratory and field evaluation of the use of municipal incinerator residue as aggregate in bituminous pavement construction is reported. A test installation that consisted of a 114.3-mm (4.5-in) thick bituminous pavement composed largely of incinerator residue was placed in Washington, D.C. The base was placed in two lifts, finished over compacted subgrade, and covered with 38.1 mm (1.5 in) of a conventional bituminous surface-course mixture. Details of the production of the residue, laboratory evaluation, asphalt plant operation, and placement and finishing of the test installation as well as recommendations and precautions for future projects in which incinerator residue is used are given. Preliminary results indicate that, with proper precautions, incinerator residue can be used as an aggregate substitute or extender in the construction of bituminous base. Its use, however, will be determined by the interplay of economic, environmental, and energy factors.

This paper describes the production and placement in Washington, D.C., of approximately 363 Mg (400 tons) of bituminous-treated street base in which municipal incinerator residue was used as the major aggregate component. This is the fourth experimental section to use incinerator residue as a component of a bituminous road pavement. Previous projects have been constructed and evaluated at locations near Houston, Texas, and Philadelphia and Harrisburg, Pennsylvania.

INCINERATOR RESIDUE

The approximately 140 municipal incinerators in operation in the United States produce approximately 4.54 million Mg (5 million tons) of residue annually. Incineration of municipal solid waste (household trash and garbage) reduces the volume of waste by about 90 percent and the weight by about 70 percent. This remaining unburned residue presents a severe disposal problem. For instance, the District of Columbia is faced with disposing of a residue stockpile of approximately 272 160 Mg (300 000 tons) that is accumulating at the rate of about 181 440 Mg/year (200 000 tons/year). This annual accumulation amounts to approximately 0.4 hm^2 (1 acre) of ground covered by about 24 m (80 ft) of residue. There are several potential uses for this material: for embankments (fill) and for structural layers in pavements in combination with binders such as asphalt. This paper deals with the use of residue with paving-grade asphalt cement as a binder.

Residues consist of materials that were not burned during incineration and may sometimes include fly ash that is recovered from stack effluents. Unburned materials consist of metals, ceramics and stone, and organic matter. The amount of unburned organic material is the result of incinerator operation and management as well as the variability of incoming waste and changing weather conditions, which may not permit adequate, offsetting adjustments of the incinerator operation to ensure complete combustion.

The approximate composition of a typical residue is (in percentages by weight) 50 percent glass, 30 percent metals, 15 percent ash, and 5 percent stone, porcelain, organics, etc. The residue is generally odorless, chemically inert, nonpolluting, and essentially structurally sound according to most standard criteria for natural aggregates used in bituminous-base construction.

The metallic components of incinerator residue are made up, as one should expect, of a rather large variety of objects—e.g., wire, cans, metal pipe, and shock absorbers. Some operations recover these objects for sale to scrap outlets. Removal is done by a trommel, which is a rotating slotted drum. Gradation of the trommeled residue complies with many specifications for bituminous bases; in cases in which the material does not comply, blending with small amounts of natural aggregate will usually produce a specification aggregate.

In summary, the use of incinerator residue in pavements should be considered as a means of disposing of the increasing accumulation of residues as well as the possible augmentation of aggregate supplies in some areas where inventories of quality and economical aggregates are low.

PROJECT LOCATION AND STRUCTURAL DETAILS

The site of the experimental section is the full width of 14th Street, S.E., from W Street to Cedar Street, in the Anacostia area of Washington, D.C. The location is a residential area that has medium residential traffic and on-street parking. The street is approximately 122 m (400 ft) in length and 9.1 m (30 ft) wide. The reconstructed section includes curb and gutter on both sides.

The subgrade soils were not tested and classified, but they were reported to be wet clay, probably classification A-6, with poor drainage. The structural section consisted of 152.4 mm (6 in) of bank run gravel subbase on the prepared subgrade. The 114.3 mm (4.5 in) of incinerator-residue asphalt base will be surfaced with 38.1 mm (1.5 in) of District of Columbia class C surface mix.

PRODUCTION OF RESIDUE FOR THE PROJECT

Residue was produced at District of Columbia Solid Waste Reduction Center No. 1. Fly-ash stack effluent is combined with the residue at this plant, and the residue is trucked to the District's Blue Plains facility and stockpiled for further processing and disposal.

The material was then trommeled to recover metals. Most, but not all, metallic objects larger than about 50.8 mm (2 in) were removed from the residue.

For this project, the material that remained after trommeling was transported to yet another location at Blue Plains for final processing and stockpiling to await later transportation to the asphalt hot-mix plant. Final processing consisted of mixing the residue with a front-end loader and dumping it onto a bar grizzly to remove particles and objects larger than approximately 25.4 mm (1 in). Finished material was graded uniformly and ready for incorporation into the final mixture. Samples were taken from this stockpile for testing at the Federal Highway Administration's Fairbank Highway Research Station laboratories.

PROPERTIES AND MATERIALS OF PROJECT RESIDUE

Gradation and Blended Aggregates

The gradation of the residue, particularly the minus-0.075-mm (minus no. 200) wash fraction, would not meet the District of Columbia specification for asphalt concrete. It was thus decided to permit the blending of as much as 30 percent natural aggregate (by weight of total aggregate) with the residue to lower the fines content. The natural aggregate was a 50:50 blend of sand and stone. The sand was Charles County concrete sand from a Waldorf, Maryland, source, and the stone was Shenandoah No. 67 dolomitic limestone, from a Millville, West Virginia, source. The aggregate data are given in Tables 1 and 2.

Asphalt

Asphalt was supplied by the Chevron Asphalt Company. The material was an AC-20 grade asphalt, steam-reduced at the company's Baltimore refinery. The crude source is unknown. The material is certified to meet AASHTO M-226-73.

MIXTURE DESIGN

Tables 3 and 4 give Marshall method data for mixes A and B. Figures 1 and 2 show the design curves for both mixtures. The job-mix formula (mix B) was as follows: 68.5 percent residue, 1.5 percent hydrated lime, 15.0 percent concrete sand, 15.0 percent dolomitic limestone, and 9.0 percent AC-20 asphalt cement (by weight of total mixture). The recommended temperature ranges are (a) for mixing, 146°C-152°C (295°F-305°F) and (b) for compaction, 138°C-143°C (280°F-290°F).

In reviewing the data in Table 3 for mix B, it is recognized that the above job-mix formula at 9.0 percent asphalt is less than optimal, primarily because of the low air voids value of 1.8 percent, which is below the lower limit of the generally accepted range of 3-8 percent. More desirable levels of air voids can be noted for 8.0 and 8.5 percent asphalt; however, at these asphalt contents, complete coating of the aggregate particles could not be achieved. Coatability was considered extremely important.

In light of the absorptive nature of the residue and the poor drainage of the wet-clay subgrade, it was desired that the mixture be as impermeable as possible to mitigate any stripping problems. Therefore, despite the low air voids obtained in the laboratory, 9.0 percent asphalt was selected for the job-mix formula. Other combinations of residue and natural aggregate—for example, a higher percentage of natural aggregate—that could have provided optimal mixture properties were not investigated.

Since field compaction is generally less than laboratory compaction [the District of Columbia Department of Transportation (DOT) specifies a minimum of 94 percent of laboratory density], it is expected that the air voids in the finished pavement would be more acceptable (higher than 1.8 percent). Further, considering the relatively low traffic volume involved and that the mixture is an underlying layer and not a surfacing, no significant increase in density above that provided by the rollers should be expected.

PLANT MIXING OPERATIONS

Asphalt Construction, Inc., of Washington, D.C., provided central plant mixing at its Brentwood, Maryland, facility, a central batch plant with a capacity of 1.81 Mg/batch (2 tons/batch). The plant was manually controlled and equipped with a bag house for air quality control. No major plant modifications were required to accommodate the incinerator residue. A small bar grizzly with openings of approximately 50.8 mm (2 in) was placed above the conveyor belt at the cold feed to remove occasional pieces of wire and stray metal. Very few pieces of metal were removed during the course of this project.

Hydrated lime was added dry to the pug mill from bag storage. To facilitate production, the lime content was slightly adjusted from 1.5 to 1.37 percent (by weight of total aggregate) to allow the use of a single 22.68-kg (50-lb) bag of lime per batch and thus eliminate weighing. The residue content was increased to compensate for the reduced lime.

After it was mixed, the material was conveyed to a heated surge silo and later trucked to the pavement location. Inert gas was not used in the silo. The maximum time of storage in the silo was approximately 7 h.

Before this experiment, two pilot runs were made at the plant. In the first run, residue from which metal had not been removed by trommeling was used. The results showed the untrommeled residue to be unacceptable because of screen blockage by some of the metal and because the mixed material contained wire and other metal objects that would prevent an acceptable finished surface.

The second pilot run used trommeled residue and provided an orientation exercise for plant personnel. This run pointed up two areas of concern that require consideration in future residue projects:

1. An unacceptable amount of dust was generated during plant operations.
2. Organic matter included in the residue burned in the dryer, which allowed a reduction in the burner fuel requirements of the plant but changed the gradation of the residue from that measured in the stockpile.

This situation and recommendations for corrections are considered more fully later in this paper.

Another problem that was encountered at the Brentwood plant was that the residue had a tendency to "hang up" and clog gates in the cold bins. The hot bins appeared to function normally, probably because the sand reduced the internal friction of the residue. This problem can be eliminated by using bin vibrators or by manually hammering on the bin when necessary. This aspect required considerable attention because the residue had a relatively high asphalt requirement and reduction of the residue will produce batches that are highly overasphalted. Several batches in the project were overasphalted and produced mixes that would ordinarily be considered unacceptable.

One characteristic of residue mixtures that should be kept in mind is that they are very sensitive to asphalt content. Anything that changes the proportions of binder to fines will produce mixtures that are overasphalted, with consequent loss in stability, or mixtures that are harsh and difficult to place and finish. If residue clogs the gates of the cold-feed bins and sand is not proportionately reduced, fines will be reduced and, if asphalt content remains constant, an overasphalted mixture will be produced.

PLACEMENT AND FINISHING

The base was placed by the Troxler Asphalt Company of Washington, D.C., on June 14, 1977. Haul time from the hot-mix plant to the job site was approximately half an hour. Use of a Blaw-Knox paver was followed by use of a steel-wheeled breakdown roller that also provided intermediate compaction. Finish rolling was done by a three-axle tandem roller according to District of Columbia specifications. The 114.3-mm (4.5-in) base was placed in two lifts. The project was held up for several days because of rain, and the subgrade was partially saturated when the residue base was placed.

Mixture appearance and workability were noticeably affected by temperature. At temperatures above approx-

Table 1. Gradation of aggregate.

Sieve Size (mm)	Percentage Passing: D.C. Residue	Sand	Stone	Hydrated Lime	Mix A[a]	Mix B[b]	D.C. Specification
25.0	100		100		100	100	100
19.0	98		91		98	97	90-100
12.5	91		50		91	86	71-91
9.5	80	100	26		80	75	60-85
4.75	53	98	3		54	53	45-65
2.36	39	90	2		40	42	33-52
1.18	30	79	0		31	34	22-40
0.60	24	53			25	26	14-30
0.30	19	12			20	16	6-21
0.15	15	5			16	12	3-31
0.075	11.7	0		100	13.0	9.5	2-8

Note: 1 mm = 0.039 in.
Gradation according to AASHTO T-27, washed analysis.
[a] Contains 98.5 percent residue and 1.5 percent lime (calculated).
[b] Contains 68.5 percent residue, 15 percent sand, 15 percent stone, and 1.5 percent lime (calculated).

Table 2. Specific gravity and absorption of aggregate.

Item	D.C. Residue	Sand	Stone	Hydrated Lime	Mix A[a]	Mix B[b]	D.C. Specification
Bulk specific gravity[c]							
Dry	2.174	2.601	2.821	-	2.176	2.313	-
Saturated surface dry	2.318	2.621	2.833	-	2.318	2.427	-
Apparent specific gravity	2.541	2.654	2.856	2.343	2.538	2.597	-
Absorption[d] (%)	6.9	0.8	0.4	-	6.8	4.9	-
Calculated surface area[e] (m^2/kg)	8.9	-	-	-	9.6	7.9	-

Note: 1 m^2/kg = 4.88 ft^2/lb.
[a] Contains 98.5 percent residue and 1.5 percent lime (calculated).
[b] Contains 68.5 percent residue, 15 percent sand, 15 percent stone, and 1.5 percent lime (calculated).
[c] AASHTO T-84.
[d] AASHTO T-85.
[e] Asphalt Institute, Manual Series 2.

Table 3. Marshall mix design data for mix A.

Property or Material	Avg of Three Specimens: 8.5 and 9.3% Asphalt[a]	9.5 and 10.5% Asphalt[a]	Single-Specimen Determination: 10.0 and 11.1% Asphalt[a]	12.0 and 13.6% Asphalt[a]	14.0 and 16.3% Asphalt[a]	Criterion
Appearance of mix	Dry	Dry	Dry	Good	Rich	
Specific gravity						
Bulk	2.018	2.057	2.065	2.076	2.050	
Maximum[b]	2.209	2.182	2.169	2.117	2.067	
Air voids (%)	8.6	5.7	4.8	1.9	0.8	3-8[c]
Voids in mineral aggregate (%)	15.1	14.4	14.6	16.0	19.0	Min, 14[e]
Voids filled with asphalt (%)	43.0	60.4	67.1	88.1	95.8	65-75[d]
Unit weight (kg/m^3)	203	207.1	207.9	208.8	206.3	-
Absorbed asphalt (%)	5.7	5.7	5.7	5.7	5.7	-
Effective asphalt (%)	3.3	4.3	4.9	7.0	9.1	-
Film thickness (μm)	3.4	4.4	5.1	7.2	9.4	Min, 6.0[e]
Dust/asphalt ratio	1.40	1.24	1.17	0.95	0.80	Max, 1.2[f]
Stability at 60°C (kN)	13.1	13.16	-	7.65	4.59	Min, 2.22[c,d]
Flow (mm)	4	4	-	5.6	7.2	2-4.6[c,d]
Effect of water on stability[g] (24 h at 60°C)						
Stability (kN)	-	10.51	9.72	-	-	Min, 2.22[g]
Flow	-	5	5	-	-	2-4.6[g]
Retained stability (%)	-	79.8	-	-	-	Min, 7.0[f]

Notes: 1 kg/m^3 = 0.062 lb/ft^3; 1 μm = 0.000 04 in; t°C = (t°F - 32)/1.8; 1 kN = 224.8 lbf; 1 mm = 0.039 in.
Mix containing 98.5 percent residue and 1.5 percent hydrated lime; compacted according to AASHTO T-245, 50 blows each side; mixed at 146°C-152°C and compacted at 138°C-143°C.
[a] Asphalt percentages for mix basis and aggregate basis, respectively.
[b] Based on an effective aggregate specific gravity of 2.471 measured by AASHTO T-209, bowl determination.
[c] Asphalt Institute, Manual Series 2, March 1974 [range of 3-11 percent voids is specified by AASHO (1)].
[d] AASHO (1).
[e] Campen and others (2).
[f] Goode (3) for AASHTO T-165 test.
[g] Not required by AASHTO T-245 but considered desirable.

Table 4. Marshall mix design data for mix B (average of three specimens).

Property or Material	8.0 and 8.7% Asphalt[a]	8.5 and 9.3% Asphalt[a]	9.0 and 9.9% Asphalt[a]	9.5 and 10.5% Asphalt[a]	10.0 and 11.1% Asphalt[a]	Criterion
Appearance of mix	Dry	Slightly dry	Good	Good	Rich	
Specific gravity						
Bulk	2.186	2.195	2.201	2.198	2.194	
Maximum[b]	2.270	2.256	2.241	2.227	2.213	
Air voids (%)	3.7	2.7	1.8	1.3	0.9	3-8[c]
Voids in mineral aggregate (%)	13.1	13.2	13.4	14.0	14.6	Min, 14[c]
Voids filled with asphalt (%)	71.8	79.5	86.6	90.7	93.8	65-75[d]
Unit weight (kg/m³)	136.4	137.0	137.3	137.2	136.9	-
Absorbed asphalt (%)	3.9	3.9	3.9	3.9	3.9	-
Effective asphalt (%)	4.4	4.9	5.5	6.0	6.5	-
Film thickness (μm)	5.5	6.1	6.9	7.5	8.1	Min, 6.0[e]
Dust/asphalt ratio	1.09	1.02	0.96	0.90	0.86	Max, 1.2[f]
Stability at 60°C (kN)	12.8	11.5	11.16	9.53	8.17	Min, 2.22[c,d]
Flow (mm)	3.6	3.6	4	4.36	4.87	2-4.6[c,d]
Effect of water on stability[g]						
24 h at 60°C						
Stability at 60°C (kN)	-	9.66	9.38	-	-	Min, 2.22[g]
Flow (mm)	-	4.36	4.36	-	-	2-4.6[g]
Retained stability (%)	-	84.0	84.0	-	-	Min, 70[f]
Four days at 48.9°C[g,h]						
Stability at 60°C (kN)	-	7.93	7.8	-	-	Min, 2.22[g]
Flow (mm)	-	4.8	4.8	-	-	2-4.6[g]
Retained stability (%)	-	68.9	69.9	-	-	Min, 70[f]

Notes: 1 kg/m³ = 0.062 lb/ft³; 1 μm = 0.000 04 in; t°C = (t°F - 32)/1.8; 1 kN = 224.8 lbf; 1 mm = 0.039 in.
Mix containing 68.5 percent residue, 1.5 percent hydrated lime, 15 percent sand, 15 percent stone; compacted according to AASHTO T-245, 50 blows for each side; mixed at 146°C-152°C and compacted at 138°C-143°C.

[a] Asphalt percentages for mix basis and aggregate basis, respectively.
[b] Based on effective specific gravity of 2.545 measured by AASHTO T-209, bowl determination.
[c] Asphalt Institute, Manual Series 2, March 1974 [range of 3-11 percent voids is specified by AASHO (1)].
[d] AASHO (1).
[e] Campen and others (2).
[f] Goode (3) for AASHTO T-165 test.
[g] Not required by AASHTO T-245 but considered desirable.
[h] After immersion period, specimens were conditioned for 1 h in 60°C water bath.

Figure 1. Design curves for mix A, with 98.5 percent residue.

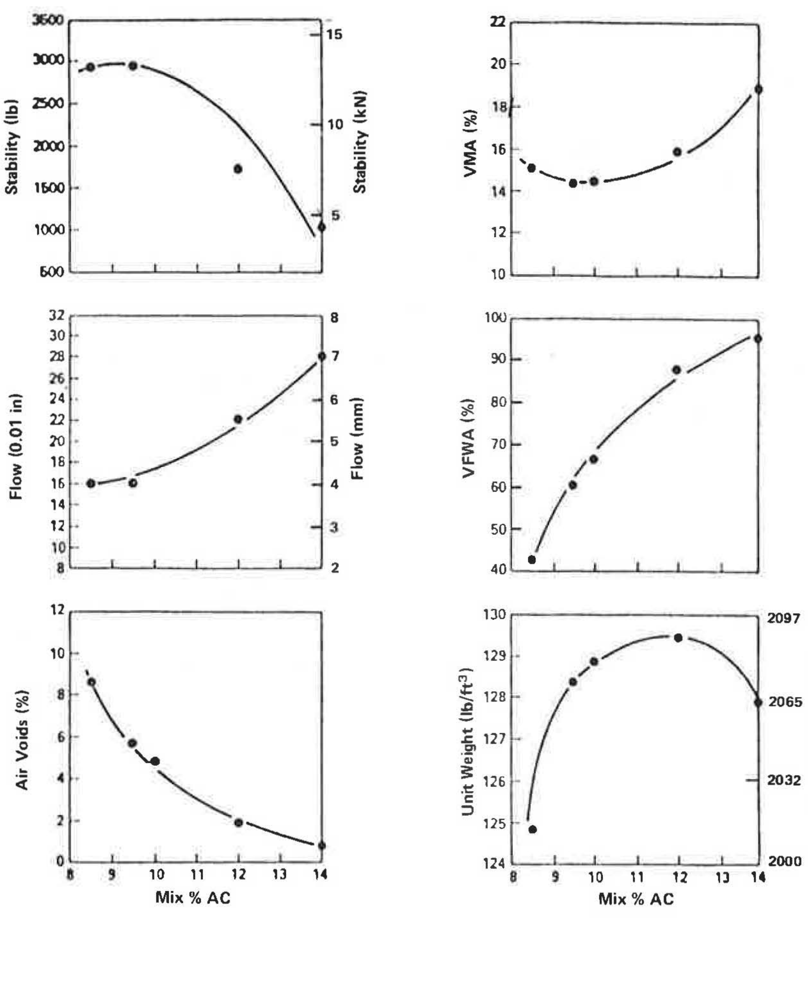
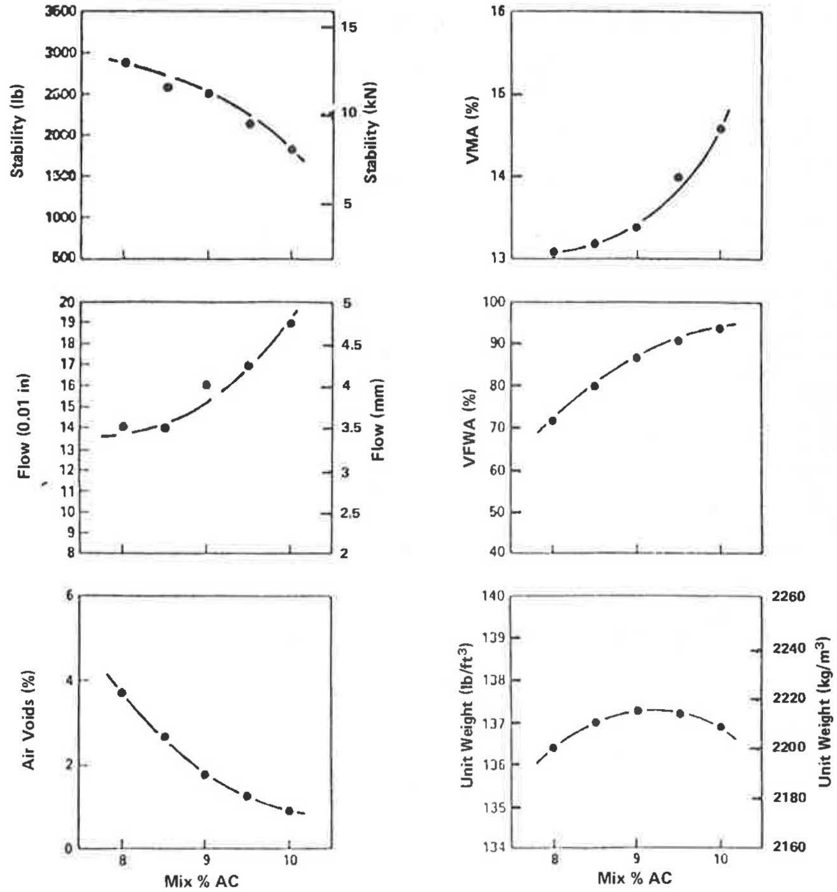


Figure 2. Design curves for mix B, with 68.5 percent residue.



imately 135°C (275°F), the mixture in truckbeds appeared "fat" (overasphalted) and very fluid and finished very easily. The finished surface at these higher temperatures was fat possibly because of the combination of high temperature and reduced fines. In any event, at elevated temperatures the roller had to be held back for at least 30 min to preclude the same sort of problems with shoving and lateral movement that are observed to occur with tender mixes. At temperatures below approximately 135°C, the mixture appeared harsh and stiff, and it required some effort to dump it from the truck to the paver hopper. In fact, one man could not move the material with a hand rake. In spite of this apparent harshness, the material finished quite well when paver speeds were slow enough to prevent tearing by the screed. Except in handworking, lower temperatures were desirable for this residue mixture.

On June 15, 1977, the day after placement, overasphalted (fat) sections were removed by scraping away approximately 6.35 mm (0.25 in) with a front-end loader over an approximate length of 30.48 m (100 ft) and a 1.83- to 2.44-m (6- to 8-ft) width.

PLANT MIX AND PAVEMENT CORE EXTRACTION

Pavement cores were taken by personnel of the District of Columbia DOT on June 23, 1977, from a section that appeared to be overasphalted as well as from a section that appeared to be normal (not overasphalted). Cores identified by a C suffix were from overasphalted sections, and those identified by a W suffix were from normal sections. Cores 2C, 3C, and 4C and 1W, 2W, and 4W were from station 5+50, and cores 5W, 7W, and 8W were from station 7+75. The analysis of extracted asphalt is based on the extract from a truck sample of the mixture.

Data indicate that, for the overasphalted section (cores 2C, 3C, and 4C), the average minus 0.075 mm (no. 200) (9.5 percent) is lower than the 13.2 and 11.9 percent shown for the normal sections (cores 1W-8W). In addition, the total of plus-0.6-mm (no. 30) natural aggregate for the overasphalted section (53.9 percent) is much higher than that for the normal sections (about 37 percent) and for the original gradation and an original laboratory Marshall design specimen (about 30 percent). Because a sufficient number of cores were not taken to permit testing for maximum specific gravity, air voids could not be computed. Cores will be taken from the same locations at a later date, and air voids will then be assessed.

Obviously, the possible occurrence of such variations in the gradation and material makeup of residue mixtures must be carefully considered, particularly when they are blended with natural aggregate, a process in which variations in proportioning can occur. Thus, variations from batch to batch in a hot-mix plant seem inevitable. Any air voids value reported on the basis of routine laboratory testing could probably be in error by at least 1 percent for conventional mixtures. This, together with the uncertainty of the composition of residue mixtures, suggests that air voids may vary, perhaps by 2-3 percent or more.

RECOMMENDATIONS FOR FUTURE PROJECTS

Two areas need additional consideration in future projects that use incinerator residue. These are residue production at the incinerator and mixing operations at the hot plant.

The properties of incinerator residue can vary considerably with time because of several conditions that have been mentioned, such as plant management, moisture content and composition of incoming solid wastes, and the method of metal recovery and later stockpiling of the residue. It is necessary to reduce this variability to reasonable limits if residue is to be used in the production of acceptable paving mixtures. The past philosophy of incinerator management has been to treat both the residue and the captured fines from the stack as a waste material that requires disposal. To realize the full potential of incinerator residue as a construction material, this concept should be changed and the residue considered not waste material but a commercial product.

One of the primary considerations is the uniformity of the product, which should be relatively easy to improve at the operational level. Of major concern is the amount of combustible matter remaining in the residue that will subsequently burn in the hot-plant drier. It is not necessary to remove this fraction, but it should be relatively constant to mitigate constant burner adjustment at the hot-mix plant. Uniformity is also necessary because burning of the material can change the gradation between the stockpile and the hot bins. Since mix design is usually based on stockpile gradations and since the characteristics of these mixtures are sensitive to fines and asphalt content, the amount of material that will be removed by burning should be predictable. In this connection, tests (such as loss on ignition) are being devised that can probably be correlated with loss through the drier.

The experiment reported here showed that dust at the hot-mix plant occurs in sufficient quantities to be rated as objectionable. Some of these fines can be eliminated by not including captured incinerator stack fines (fly ash) in the residue. This may or may not be difficult to do and will depend on the design and operation of the particular incinerator that is producing the residue. A secondary, but by no means insignificant, benefit of the removal of these fines is that they contribute heavily to the asphalt requirements of the mixture. Design asphalt content is quite high, and even a slight reduction of asphalt content, at present prices, will add to the economic advantages of using incinerator residue. Another aspect of dust removal and hot-mix-plant operation is that dust can block the photoelectric cell that controls the burner and either shut the burner off or prevent downward adjustment, which will produce overheated mixtures. Finally, the removal of dust will reduce the amount of natural aggregate that is necessary to meet gradation requirements, which will further improve the economics of using residue.

Another area that should receive consideration is a lime-slurry application at the stockpile. Because of the high glass content of the residue and the high probability of water stripping of asphalt from these glass particles, lime will be required as an antistripping agent unless an adequate agent is added to the asphalt. Experience has shown that slurried lime is more effective than dry lime as an antistripping agent. Since incorporating lime slurry at the hot-mix plant is not as practical as applying it at the stockpile, it is recommended that provisions be made for introducing slurried lime at the stockpile. Elaborate and complicated procedures and equipment are not necessary. After the amount of material to be treated is determined, the slurried lime can be applied either by a simple pump-and-spray arrangement or by conventional highway distributors that are readily available.

With regard to hot-mix-plant operations, two areas need to be considered: cold-feed control and temperature control. Cold-feed control should include a vibrator to prevent bridging in the bin as well as clogging of the gate.

Vibrators are common and can be found at many plants. In view of the sensitivity of workability to temperature, particular attention should be given to its control. This problem should be mitigated by reducing the variability in the amount of fines by means of modified incinerator operations, but temperature control should, nonetheless, receive close attention.

Future experiments would also be more effective if the amounts of material involved were adequate to permit plant adjustments to compensate for material variability and plant trim. A minimum of 907.2 Mg (1000 tons) appears to be a reasonable figure to ensure a supply of material that has low enough variability to allow for adequate evaluation of the field performance of pavements that contain incinerator residue.

Laboratory Extraction and Recovery of Asphalt

The results of physical tests on the asphalt recovered from a truck sample of mixture, taken at the time of placement, show that a normal amount of hardening of the asphalt occurred during the plant mixing process.

Pavement Performance

The performance of this material will be described in a future paper, after sufficient time has elapsed to observe trends in behavior and performance.

Use of Incinerator Residue in Surface Courses

Residential traffic was permitted on the base before the surface was applied, and this afforded the opportunity to observe the performance of incinerator residue as an aggregate in a surface course. Very soon after the surface was opened to traffic, the asphalt film began to strip away from glass particles in the mixture. Sometime later, traffic started to pull the exposed glass particles away from the asphalt matrix that binds them into the mixture. Since the mechanism of this action has not been studied to determine whether tire action, water, or a combination of these forces is the main cause, preventive measures such as additives or increased film thickness cannot be recommended. Cores taken during the period of performance evaluation will be examined to determine to what extent the stripping can be attributed to water action.

Since the same stripping action was observed at a Harrisburg, Pennsylvania, installation where residue was used in a surface course, it is recommended that incinerator residue be used only for base-course construction and not be used in surface courses until solutions and preventive actions can be provided.

CONCLUSIONS

Based on this experiment and on the results of similar efforts at Houston, Texas, and Philadelphia and Harrisburg, Pennsylvania, and on the performance data being generated by the Houston experiment, it can be concluded that municipal incinerator residue should be considered for-use as an aggregate in the construction of bituminous bases. Although it is too early to completely evaluate performance, early indications are that the material should perform adequately under medium-level traffic.

The operations of the specific incinerator that produces the residue should be considered, and attention should be given to the removal of fines and the addition of slurried lime to the stockpile. Acceptable mix-design procedures and hot-mix-plant control should, of course, be exercised in any future use of the material.

ACKNOWLEDGMENT

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Potential Use of Incinerator Residue as Aggregate for Portland Cement Concrete

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An evaluation, as a potential source of aggregate, of the residue that remains after the incineration of municipal refuse is described. The results of American Society of Testing and Materials acceptance tests indicate that incinerator residue can potentially be used as subbase and base-course material and aggregate for asphaltic concrete, portland cement concrete, and masonry units. Concrete strengths of approximately 27.6 MPa (4000 lbf/in^2) are possible. Medium-weight concrete blocks that meet ASTM C 90 specifications have been manufactured. Alkali-aggregate reactions cause expansion, but it appears that this problem can be controlled.

A joint research project has been conducted by the Department of Civil Engineering, University of Notre Dame, and the Environmental Research Laboratory of Wheelabrator-Frye, Inc., of Mishawaka, Indiana, to develop profitable uses for incinerator residue produced by incineration of municipal refuse. The residue is discharged from grates at a temperature of about 815°C (1513°F) and passed through a grader. Residue smaller than 5.1 cm (2 in) passes through a water-sealed discharge and is quenched. Fly ash produced by the combustion of the refuse is collected in an electroprecipitator. It can be kept separate or combined with the residue in the quenching pit. In this research, the fly ash was combined with the residue. The wet-quenched material can then be dried, screened, and separated into magnetic and nonmagnetic fractions.

PROPERTIES OF NONMAGNETIC FRACTION

The nonmagnetic fraction can be described as a graded material made up of discrete particles >3.8 cm (>1.5 in) to <0.075 mm (<no. 200) in size. It is composed of glass, sand, slag, ash, and some metallic components. The coarse fraction can contain as much as 50 percent glass (1, 2). Some physical properties of the nonmagnetic fraction are given below (1 mm = 0.039 in):

Sieve Size (mm)	Percentage Retained	Specific Gravity	Percentage Absorption
25.4	5.0		
19.1	11.0		
12.7	17.8		
9.5	40.5	2.57	1.96
4.75	66.0	2.50	2.63
2.36	76.0	2.13	7.84
1.18	82.0	1.75	16.80
0.60	87.5	1.34	32.5
0.30	91.4		
0.15	94.3	2.42	
0.075	96.8		

Specific gravity was determined based on ASTM C 128, and percentage absorption was determined based on ASTM D 854.

The material was subjected to American Society of Testing and Materials (ASTM) acceptance tests to evaluate its potential as aggregate. The results were as follows (1 kg/m^3 = 0.062 lb/ft^3):

Test Property	Coarse Aggregate	Fine Aggregate
Clay lumps and friable materials (%)	0.5	
Los Angeles abrasion loss (%)	35.6	
Soundness loss ($MgSO_4$, five cycles) (%)	~5	~20
Organic impurities	OK	OK
Density (kg/m^3)	1280	960
Staining		Very light
Coating and stripping asphalt		OK
Loss on ignition (%)		

PORTLAND CEMENT CONCRETE

The potential use of residue as aggregate in portland cement concrete (PCC) is best evaluated by using the relation between water-cement ratio and strength. This relation, shown in Figure 1, includes combinations of residue with limestone and sand as replacements for coarse and fine components of aggregate. As a basis for comparison, the test results of a mix in which limestone and sand were used as aggregate are included.

It is apparent that, at higher water-cement ratios (lower strengths), the residue compares favorably with the limestone and sand. At lower water-cement ratios, the fine component of the residue appears to lower the strength of the mix. Its high porosity would tend to explain this. Replacement of the fine portion of the residue by sand appears to produce strengths that approach those of a normal concrete mix. Splitting strengths ranged from 10 to 15 percent of the compressive strength.

The use of residue as aggregate in PCC is complicated by a potential alkali-glass reaction. Johnston (3) found this to be true when he substituted waste glass for coarse aggregate in concrete. Johnston's data indicate that the detrimental expansion that occurs with significant replacements can be avoided by controlling the al-

Figure 1. Water-cement ratio versus strength.

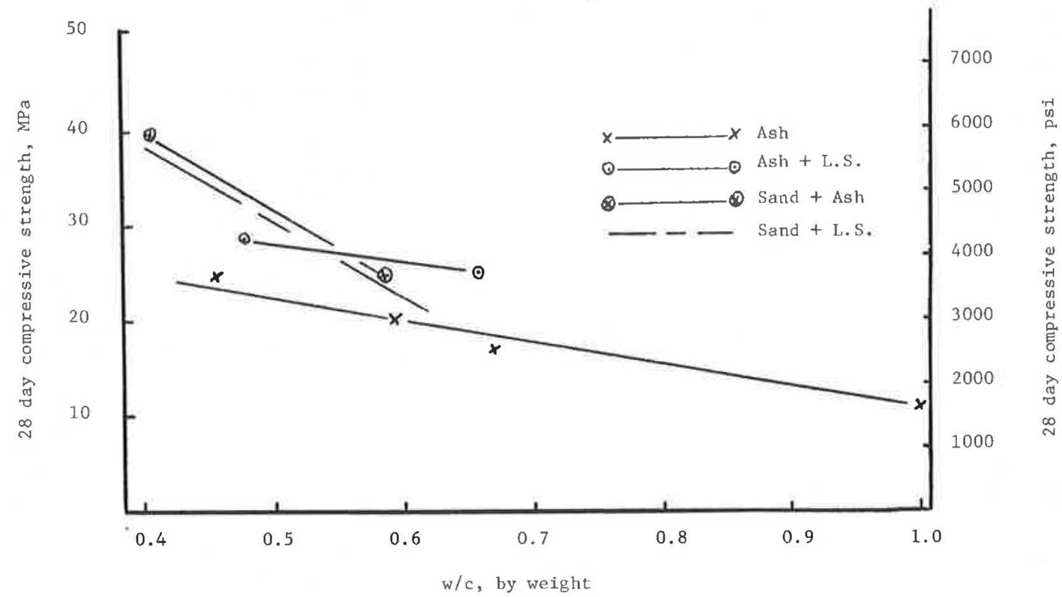


Figure 2. Expansion of concrete.

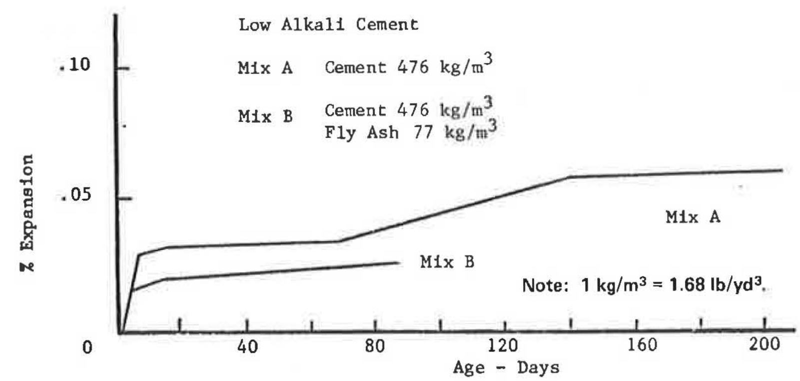
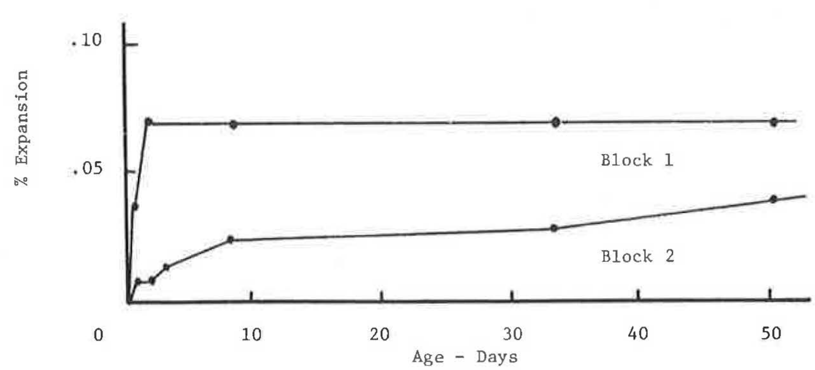


Figure 3. Expansion of concrete block.



kali content and using fly ash as a replacement for some of the cement.

Figure 2 shows available expansion data for concrete in which residue was used as aggregate. The expansion test results were obtained on concrete bars 8.9x8.9x29.2 cm (3.5x3.5x11.5 in) that were continuously moist cured at 21°C (70°F). As Johnston (3) points out, this approach is preferable to the standard mortar bar test (ASTM C 227) because it more readily identifies such variables as cement content and coarse-aggregate component.

It is interesting to note that significant expansions occur at an early age and then continue slowly over longer periods of time (Figure 2). This is similar to results obtained by Phillips and Cahn (4) in their investigation of the use of refuse glass in concrete block. The presence of additional fly ash in mix B reduces expansion by apparently dissipating the alkalies with the additional surface area available for reaction.

CONCRETE MASONRY BLOCK

The use of residue as aggregate in concrete block was investigated. The results are compared below with ASTM C 90 specifications (1 MPa = 145 lbf/in^2; 1 kg/m^3 = 0.062 lb/ft^3):

Property	ASTM C 90 Hollow Load-Bearing Units, Grade N	Blocks Made with Incinerator Residue
Compressive strength of individual units using gross area (MPa)	5.5	>5.5
Medium weight classification (kg/m^3)	1682-2002	1728
Maximum water absorption (kg/m^3)	240	232
Moisture content		
Percentage of total	35	35
Associated percentage of linear shrinkage	≤0.03	>0.3

The grading of the residue was altered to conform to that used locally in lightweight block. The blocks were fabricated in a manually operated single-block machine. Although they are not directly comparable with data for machine-manufactured units that are steam cured at low pressure, the data indicate a potential use for residue in this area. The possibility of alkali-glass reaction is also of concern in this case. Figure 3 shows the expansion characteristics of two blocks. Block 1 was water-cured at 21°C (70°F) after 24 h of exposure to a laboratory environment (~24°C and 50 percent relative humidity). After 24 h of exposure to laboratory air, block 2 was cured in water at a temperature of 82°C (182°F) for 24 h to simulate low-pressure steam curing. The block was then cured in water at 21°C. The increased initial expansion associated with block 2 is to be expected. The subsequent lack of expansion may be especially significant. The fact that these data are based on single blocks and do not include kiln curing must be kept in mind.

CONCLUSIONS

On the basis of tests carried out to date, the following conclusions appear to be warranted:

1. Incinerator residue can potentially be used as aggregate in PCC. Strengths of 27.6 MPa (4000 lbf/in^2) have been attained. The problem of alkali-aggregate expansion exists, but initial test data indicate that it can be controlled.
2. Incinerator residue can potentially be used as aggregate in masonry block. A medium-weight block that meets ASTM C 90 specifications has been manufactured in a manually operated block machine.

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Composition and Characteristics of Municipal Incinerator Residues

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The basic physical and chemical composition of municipal incinerator residues and their essential engineering properties are discussed. The analytic methods used in characterizing samples of incinerator residue from various sources are also discussed. Some unique characteristics of incinerator residue are identified, and the significance of these characteristics relative to the potential use of residue as construction material is outlined. A means for classifying the residues from municipal-scale incinerator operations is recommended.

Billions of dollars are spent each year on the collection and disposal of municipal solid waste in the United States. Although 90 percent of all solid waste is currently disposed of in landfills, incineration is the principal means of solid waste disposal in many U. S. metropolitan areas. The primary advantage of incineration is that it reduces the volume of incoming solid waste by as much as 80-90 percent, thereby extending existing landfill space, which is often scarce in densely populated areas. Furthermore, combustion of solid waste provides municipalities with an opportunity to convert such waste into a needed source of energy.

Despite such advantages, incineration produces a residue after burning that usually represents 20-30 percent by weight of the original refuse and must be disposed of in an environmentally acceptable manner. Currently, there are approximately 140 municipal incinerator plants in operation in the United States, primarily in the Northeast. It has been estimated that these plants produce a combined total of 4.5 million Mg (5 million tons) of incinerator residue annually (1).

Incinerator residue is potentially useful in a number of construction-related applications. It is necessary, however, to know something about the material and its essential properties before considering its possible utilization. This paper discusses the physical and chemical characteristics of different incinerator residues and offers a system for classifying residues from different sources.

CLASSIFICATION OF INCINERATOR RESIDUES

Incinerator residue is a heterogeneous material derived from the combustion of municipal refuse. Essentially, municipal refuse consists of a combustible fraction (including paper, wood, food wastes, textiles, and yard wastes) and a noncombustible fraction (including metals, glass, bricks, ceramics, and rocks). Although the composition and moisture content of refuse vary during different times of the year and in different parts of the country, the combustible fraction ordinarily comprises 60-80 percent by weight of the refuse. The table below gives published data on the nationwide average composition of municipal refuse (2):

Component	Percentage by Dry Weight
Paper	51.6
Food wastes	19.3
Metals	10.2
Glass	9.9
Wood	3.0
Textiles	2.7
Leather and rubber	1.9
Plastics	1.4
Total	100.0

These data assume a material free of "yard waste" and "miscellaneous" components (yard waste includes leaves, grass, and branches, and miscellaneous includes bricks, rocks, and dirt). These two fractions are highly variable and can constitute up to one-third of refuse at certain times.

The proper combustion of solid waste in a municipal incinerator plant is influenced by three basic factors: time, temperature, and turbulence. For satisfactory burning to occur, the refuse might be exposed for a sufficient amount of time to temperatures in the range of 871°C-982°C (1600°F-1800°F). Generally, the greater the agitation of the refuse is during burning, the more complete is the "degree of burnout".

Degree of burnout—the ratio of the incinerated refuse to the combustible fraction of the refuse—is directly related to the design of the incinerator furnace and the feed grates but can also be affected by differences in plant operational philosophies. In general, continuous-fed incinerators produce a better burnout than batch-fed incinerators. In addition, incinerators that have grates that agitate the refuse during the burning cycle can be expected to produce a higher degree of burnout than incinerators with grates that provide little or no agitation.

For practical purposes, municipal incinerator residues can be broadly classified into three main categories on the basis of degree of burnout:

1. Well-burned—Well-burned residues come from continuous-fed incinerators that provide a high degree of grate agitation. Residues of this type are usually produced at plants that have rotary kilns, reciprocating grates, or rocking grates. Well-burned residue should constitute approximately 10 percent by volume and from 20 to 30 percent by weight of the refuse input (3).
2. Intermediately burned—Intermediately burned residue is produced by continuous-fed incinerators with traveling grates, which do not mechanically agitate or break down the burning refuse to any great extent. These residues generally represent approximately 20 percent by volume and from 25 to 35 percent by weight of the incoming refuse (3).
3. Poorly burned—Poorly burned residues are byproducts from batch-fed incinerators or from poorly operated continuous-fed incinerators, especially plants with traveling grates. Variations in burning time in such plants significantly affect the burnout and result in a substantial amount of unburned or partially burned combustible material. Typically, poorly burned residue will be from 30 to 40 percent of the original volume and weight of the refuse (3).

Since incineration is used by cities to reduce the overall volume of solid waste for disposal, quality control of the resultant residue is of little importance in the operation of most incinerator plants. Therefore, the basic design of the furnace and the grates should not be the only criterion used to classify a source of incinerator residue. The manner in which the plant is operated should also be considered.

In addition to conventional incineration, pyrolysis is also being used in the thermal reduction of municipal solid waste (although very few municipal-scale plants are operational at this time). Pyrolysis is a process by which heat is applied, in the absence of excess air, at temperatures that range from 500° to 1000°C (932°F-1832°F). This results in the chemical decomposition of organic substances. In addition to oil and gaseous fuel by-products, the process produces a char residue. The residues from pyrolysis processes are, in most cases, extremely well burned, uniformly graded, glassy materials that represent approximately 5-10 percent by volume and 10-20 percent by weight of the original solid waste (4). Pyrolysis residue should be considered a special residue classification.

Because classification of incinerator residues according to degree of burnout depends largely on visual inspection and the judgment of the observer, a more definitive means of assigning a residue classification is needed for practical use. Recommended criteria for quantitatively classifying incinerator residue are given below:

Classification	Loss on Ignition (%)	Color of Organic Test Solution
Well burned	<5	Lighter than standard
Intermediately burned	5-10	Same as standard
Poorly burned	>10	Darker than standard

Since there is a unique similarity among the residues from pyrolysis operations, these criteria are not applicable to materials produced by pyrolysis. In classifying pyrolysis residue, it is necessary only to identify that the source of the residue was a pyrolysis process.

The two parameters used as classification criteria for incinerator residues are loss on ignition and the presence of organic impurities. Since these parameters are indicative of the relative content of carbonaceous and organic matter in the residue, they can readily be related to the degree of residue burnout, which is the basis for the classification system.

Loss on ignition is tested in accordance with a test method developed and reported by the U. S. Environmental Protection Agency (5). This procedure involves fine grinding a representative sample of the residue, selecting a 50-g sample for testing, subjecting the sample to 1-h exposure in a muffle furnace at 950°C (1742°F), and computing the loss in weight of the sample. The loss-on-ignition value is directly related to the percentage of combustible material contained in the sample. Because of the comparatively small sample size, it is recommended that a series of tests be performed so that a truly representative value is obtained.

The test for organic impurities in sands for concrete (ASTM C 40) is simple to perform. It consists of adding a 3 percent NaOH solution to a sample of material in a standard glass container, shaking, and allowing it to stand for 24 h, after which the color of the supernatant liquor in the bottle is compared with that of a standard reference color solution. If the supernatant liquor is darker, the material is considered to have a comparatively high organic content.

Classification of incinerator residues is extremely important in identifying the potential use of the material. The classification system described here is based on normal operating conditions at the incinerator plant. At the same time, it is recognized that significant variations in operating conditions over an extended period of time could result in a change in classification for a particular source of residue.

SAMPLING INCINERATOR RESIDUE

The collection of representative samples of incinerator residue is essential for proper characterization of the material from a specific source. Since different plants use different burning techniques and discharge methods, sampling procedures are extremely important in order to ensure that representative samples are obtained. It is recommended that approximately 1.8 Mg (2 tons) of residue be collected at a specific incinerator site over a relatively short time period (from 30 min to several hours) of normal plant operation. Samples should be collected in 208-L (55-gal) drums, and maximum particle size should be limited to objects 152.4 mm (6 in) in diameter. Samples can be obtained directly from the residue discharge chute, where possible, or loaded from stockpiles by a front-end loader. Samples can probably be obtained most conveniently from stockpiles.

ANALYSIS OF INCINERATOR RESIDUES

A recently completed study on the use of incinerator residue in highway construction, performed for the Federal Highway Administration (FHWA) by Valley Forge Laboratories (7), involved the analysis of residues from seven different sources, each of which represented a type of incinerator (including pyrolysis) in terms of grate design and anticipated type of residue. The findings of characterization studies performed on these samples are discussed below and are compared with data from a 1968 study by the U. S. Bureau of Mines (6) in which 0.9-Mg (1-ton) or larger samples of residues were obtained from seven incinerator plants and analyzed.

Basic Engineering Properties

Incinerator residue, as it is discharged directly from the plant, is a soaking-wet mixture of glass, metals, ash, minerals, and partially burned combustible matter. Its moisture content is highly variable and depends on the degree of burnout, the method of quenching, and the age of the residue. The moisture content of incinerator residues can range from as low as 15 percent, for well-burned residue that has been stockpiled for some time, to 60 percent or higher, for freshly quenched residue that has not been well burned. It is well known that stockpiling reduces the moisture content of fresh residue over time. In the studies already noted, the average moisture content of samples of incinerator residue was found to be approximately 30 percent but that of stockpiled, drained samples to average about 20 percent. The natural moisture content of pyrolysis residues is extremely low, usually from 1 to 2 percent, because these materials, after the separation of ferrous metal and carbon, are essentially glassy slags with very low porosity.

The unit weight of incinerator residues is directly related to the composition of the residue and the extent of burnout. As burnout improves, the unit weight of the residue increases. Samples of incinerator residue were first dried to constant weight at 105°C (220°F) and then screened to a 76.2-mm (3-in) maximum size. Dry unit weight was determined by compacting the samples in a 152.4-mm (6-in) diameter container. The procedures used were a modification of the test method outlined in ASTM C 29. Dry rodded unit weights were found to range from less than 800 kg/m^3 (50 lb/ft^3) for poorly burned residue to more than 1280 kg/m^3 (80 lb/ft^3) for well-burned material. The unit weight of the glassy fraction of pyrolysis residues is considerably higher than that of incinerator residues, usually ranging from 1600 to 1920 kg/m^3 (100-120 lb/ft^3).

Initial determinations of particle-size distribution for "as-received" incinerator residues involved the removal of oversize material by means of a 76.2-mm screen. Regardless of the degree of burnout, all residues sampled were found to be essentially well-graded materials with particle sizes that ranged from the 76.2-mm upper limit to a nominal amount passing the 0.075-mm (no. 200) screen. In general, residues from rotary-kiln incinerators are likely to be more finely graded than grate-type incinerator residues because of the constant tumbling action in the kiln, which results in more degradation of residue particles. Pyrolysis residues are, by comparison, much more finely graded than incinerator residues. Pyrolysis residues, as received from the plant, will not normally contain particles larger than 12.7 mm (0.5 in), and the majority of particle sizes were in the 4.76-mm (no. 4) to 0.42-mm (no. 40) range. Figure 1 shows the normally expected particle-size distribution of as-received incinerator and pyrolysis residues. The maximum particle size of the incinerator residues is 76.2 mm.

To more realistically evaluate incinerator residue for potential applications in highway construction, a maximum particle size of 38.1 mm (1.5 in) was selected as the upper limit for material used as structural fill or in base-course applications. Samples of various sources of incinerator residue were oven dried and passed through a 38.1-mm screen. Several sieve analyses were performed on samples from each basic type of residue. Despite the inherent variability of residues from different sources, the results of the sieve analyses of screened residues were, for the most part, comparable. Figure 2 shows the range of particle-size distribution observed for various sources of incinerator residues with a maximum size of 38.1 mm.

Physical Characterization

The physical composition of incinerator residues is

Figure 1. Particle-size distribution of as-received incinerator residues.

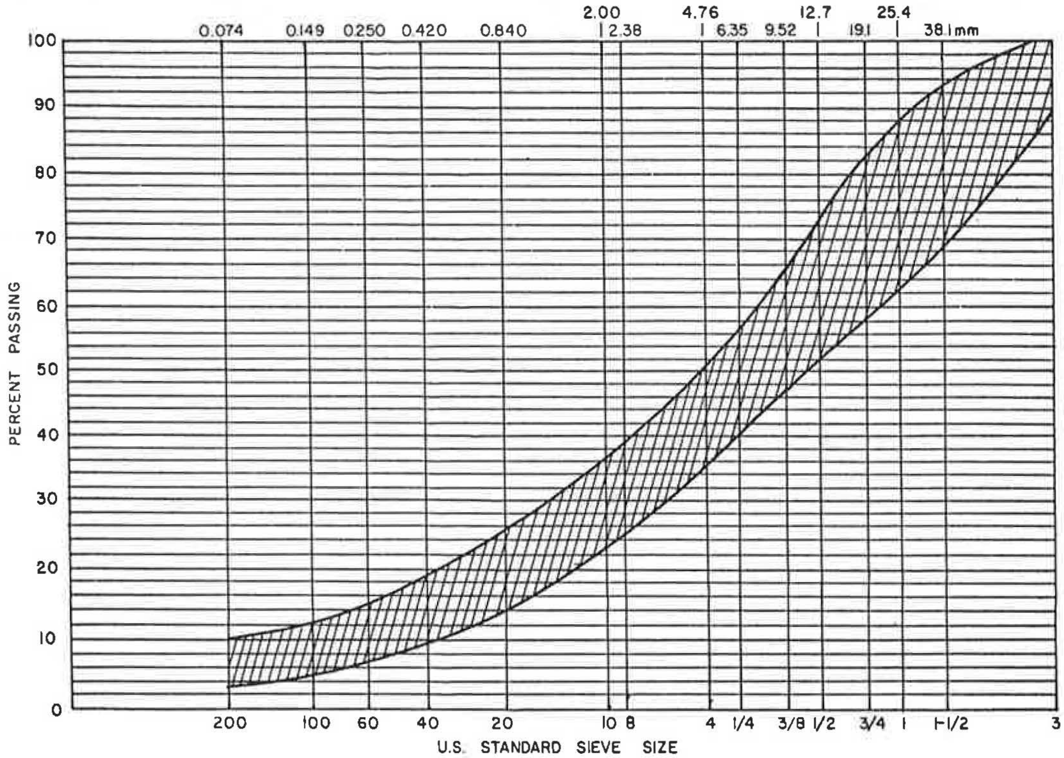
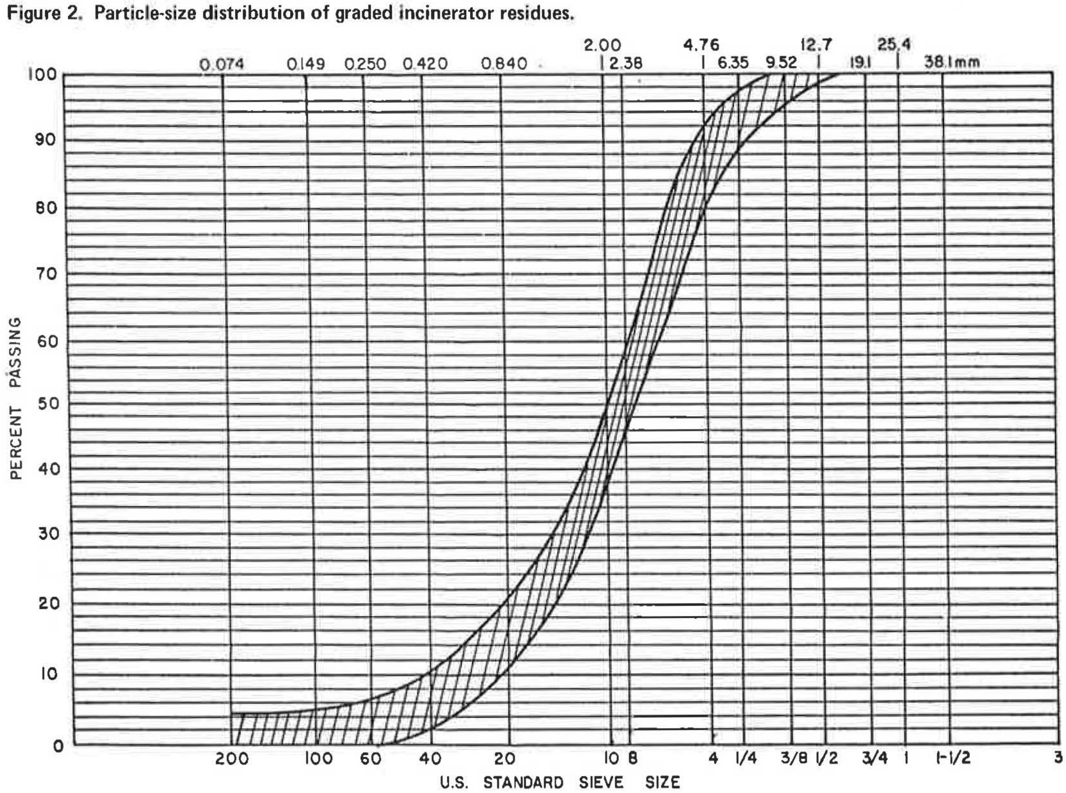


Figure 2. Particle-size distribution of graded incinerator residues.



significant because of the heterogeneity of these materials. Fluctuations in composition influence the engineering properties and behavior of residues and are thus a factor in determining the overall suitability of different residue sources in some form of highway construction use.

In extensive laboratory studies performed for FHWA by Valley Forge Laboratories, the principal components of municipal incinerator residues have been identified as glass, ferrous metal, nonferrous metal, minerals and ash, and combustible and organic matter. The components of as-received residue samples were identified by first separating each of the samples into individual sieve fractions or cuts, as follows:

Size Fraction	Sieve Cut	Sieve Size (mm) Passing	Retained
Coarse	1	76.2	25.4
	2	25.4	12.7
	3	12.7	6.35
	4	6.35	2.00
Fine	5	2.00	0.420
	6	0.420	0.074
	7	0.074	

Hand sorting and visual inspection were used to identify the principal components of the coarse fractions. For the fine cuts, visual identification proved to be extremely difficult, even under a microscope, because of the uniform color and appearance of the fine-size particles [<2 mm (<no. 10)]. The composition of the fine cuts was therefore determined chemically.

Ignition in a Leco induction furnace was used to determine the carbon content in the finer fractions. The carbon content was then used to estimate the amount of unburned combustibles, assuming an average of 86 percent fixed carbon in typical organic compounds. The iron content of the fine sieve fractions was determined by volumetric chemical analysis by using the Zimmerman-Reinhardt method with $KMnO_4$ titration. The iron identified in the finer cuts was assumed to be iron oxide.

The glass content in the fine cuts was estimated from the silicon content obtained with gravimetric chemical analysis. Standard wet silicate analysis of the solution of sodium carbonate fusion with the sample material was used. Since glass is approximately 70 percent silica and silicon makes up approximately 47 percent of silica, the silicon content was converted to percentage glass by multiplying the percentage of silicon by 3.062.

Various chemical methods were used to identify the principal nonferrous metal components in the finer fractions. Standard gravimetric analysis was used to determine the total alumina (Al_2O_3), including that present in glass and ceramics. It is unlikely that much, if any, free aluminum is present in the fine cuts since it would be easily oxidized. Atomic absorption analysis was used to identify the copper content. Other nonferrous metals are normally found in trace amounts. The identification of these trace elements will be discussed as part of the chemical composition of incinerator residues.

The overall physical characterization of each residue sample was determined by multiplying the percentage of each component in any sieve fraction by the weight distribution of that sieve fraction relative to the entire sample and then summing each of the products. Samples that represented the same residue classification were grouped together and averaged for reporting. Table 1 gives the physical composition of the as-received incinerator residue samples evaluated by Valley Forge Laboratories and also the data for average residue composition developed in 1968 by the U. S. Bureau of Mines.

The data given in Table 1 show that the average composition of incinerator residues reported by the two studies is basically in agreement, although the percentage of ferrous metal reported by Valley Forge Laboratories was considerably lower than that reported by the Bureau of Mines. This is because the maximum particle size of residue samples evaluated by Valley Forge Laboratories was 76.2 mm (3 in), whereas the Bureau of Mines exercised no size control of residue samples. The fact that most material larger than 76.2 mm is ferrous metal accounts for the apparent difference in the data.

Although the average composition of residue samples appears to be consistent between the two studies, closer inspection of Table 1 shows a significant variation in composition among residues of different classifications. Most obvious is the wide range of combustible and organic matter, which can constitute between 25 and 30 percent by weight of a poorly burned residue. The amount of minerals and ash also varies according to the degree of burnout and is highest in the well-burned residues. The deviations in composition are much less pronounced for the other component materials. In fact, the amount of nonferrous metal was found to be almost the same in all residue samples. The average physical composition of as-received incinerator residues, as determined by Valley Forge Laboratories, is shown in Figure 3.

Residues from pyrolysis processes are quite different from municipal incinerator residues. The physical composition of the "glassy aggregate" fraction of residue obtained from the Landgard pyrolysis system in Baltimore was as follows:

Component	Percentage by Weight
Glass	65
Mineral matter	28
Ferrous metal	3
Nonferrous metal	2
Carbon	2
Total	100

The composition of this material is probably representative of most sources of pyrolysis residue in which the carbon char and ferrous metal components have first been separated, as was the case here. Figure 4 shows the composition of this sample of pyrolysis residue. Although the residue is well burned, the combined carbon and ferrous components are nearly equal in weight to the glassy fraction.

Chemical Composition

The chemical composition of incinerator residues is determined by the characteristics of the incoming refuse, which fluctuate according to geography and season and the extent to which the refuse is burned at the plant. The most variable component in the basic chemistry of incinerator residues is its carbon content. The amount of carbon in incinerator residue can vary widely depending on the degree of burnout. To determine reasonably accurate figures for the chemical composition of residues from different sources, it was decided to compare the analyses of the carbon-free fractions of residues evaluated in previous studies.

In a study of high-temperature incineration, Bortz and Pincus (8) cited the chemical analysis of organic-

Table 1. Physical composition of municipal incinerator residues.

	Percentage by Weight				
	Valley Forge Laboratories				
Component	Well Burned	Intermediately Burned	Poorly Burned	Average	Bureau of Mines
Glass	39.9	51.5	45.7	48.0	44.0
Mineral matter	39.5	17.5	13.1	21.0	17.6
Ferrous metal	13.8	16.0	8.7	14.2	28.0
Nonferrous metal	3.3	4.3	4.3	4.1	1.4
Combustible and organic matter	3.5	10.7	28.2	12.7	9.0
Total	100.0	100.0	100.0	100.0	100.0

Figure 3. Physical composition of municipal incinerator residues.

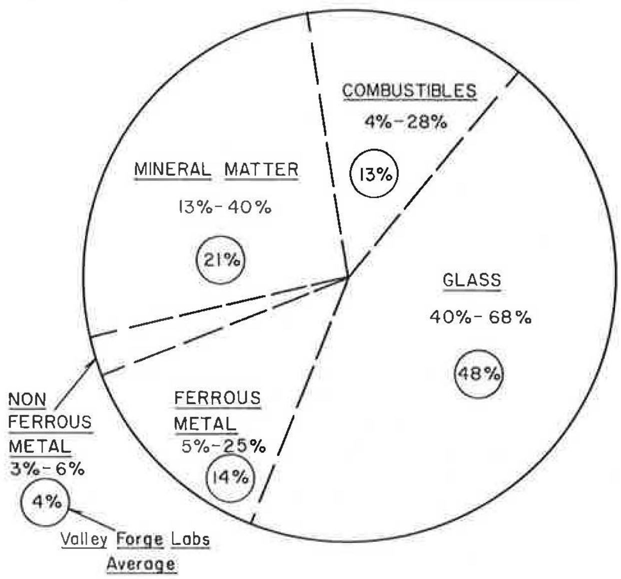


Figure 4. Physical composition of Landgard pyrolysis residue.

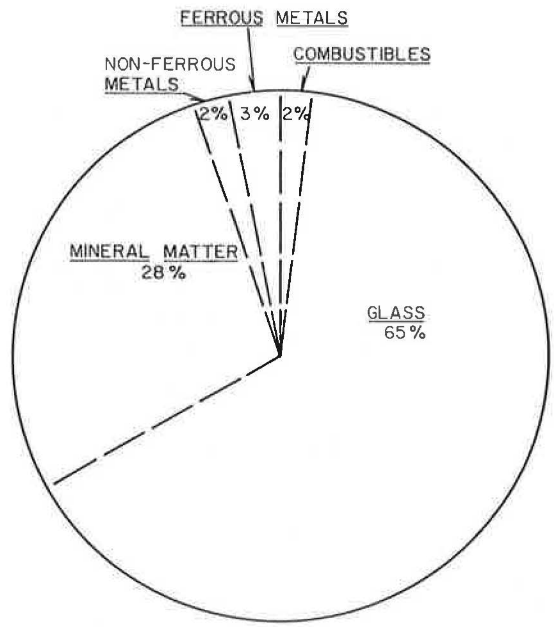


Table 2. Chemical analysis of carbon-free incinerator residue.

	Percentage by Weight				
Compound	Hartford	Brockton	Pittsburgh	St. Louis	Average
SiO	61.9	62.4	60.0	54.88	59.8
Al_2O_3	13.6	7.6	8.0	9.87	9.8
Fe_2O_3	3.7	5.2[a]	4.0	3.42	4.0
TiO_2	-	0.7	-	1.25	1.0
CaO	6.6	14.2	17.0	9.93	11.9
MgO	2.0	3.3	5.0	1.66	3.0
BaO	0.2	-	-	-	-
ZnO	1.7	-	-	-	0.4
PbO	0.5	-	-	-	0.1
CuO	0.4	-	-	-	0.1
MnO	-	0.2	1.0	-	0.3
Na_2O	9.4[b]	3.8[b]	3.0[b]	6.19	6.1
K_2O	-	-	-	1.99	0.5
SO_3	-	-	-	3.66	0.9
P_2OS	-	0.7	-	1.33	0.5
Other	-	1.9	2.0	5.82[c]	1.6
Total	100.0	100.0	100.0	100.0	100.0

[a]Percentage expressed as FeO.
[b]Percentage includes both Na_2O and K_2O.
[c]Includes 2.79 percent reported as ash.

free incinerator residues from three sources. Residues from high-temperature furnaces in Hartford, Connecticut; Brockton, Massachusetts; and Pittsburgh, Pennsylvania, were analyzed. The findings, which are given in Table 2, are considered to be closely representative of well-burned residues. Also included in Table 2 is an analysis of carbon-free residue from an installation in St. Louis (9). An average chemical composition of carbon-free incinerator residue, based on the analyses of these four sources, is also given in Table 2.

In addition to the data given in Table 2, further chemical analyses of residues from different sources have been reported by Lilge (10), Schoenberger and Purdom (11), and others. Gony and Cossais (12) have documented the chemical composition of residues from French incinerators. All of these analyses are essentially in agreement with the average chemical composition presented in this paper.

As data given in Table 2 show, the principal component of carbon-free incinerator residue is silica, which constitutes approximately 60 percent by weight of the residue. Oxides of calcium and aluminum make up approximately 22 percent and sodium, potassium, and iron oxides 10 percent. Magnesium and titanium oxides constitute 4 percent by weight of incinerator residue. The remaining 4 percent consists of sulfates and metallic oxides of copper, lead, zinc, manganese, phosphorus, and other trace elements.

Individual sources of residue may vary somewhat from this average composition. But such differences should not be significant, and the relation between principal components should be essentially the same. Although carbonaceous material is not accounted for in Table 2, allowance must be made for the amount of carbonaceous material contained in residue. The most

Table 3. Summary of results of aggregate acceptance tests for incinerator residues.

Test	ASTM Test Method	Type of Residue: Well Burned	Intermediately Burned	Poorly Burned	Pyrolysis
Specific gravity[a]	D 2041	2.53	2.43	2.27	3.18
Moisture content[b] (%)	D 698	12-18	14-20	16-24	
Dry density[b] (kg/m³)	D 698	1532-1774	1452-1693.5	1210-1452	
Los Angeles abrasion loss[c] (%)	C 131	37.1	36.6	36.4	40
Sodium sulfate soundness loss (%)	C 88				
Fine[d]		10.3	11.8	27.6	5.2
Coarse[e]		15.6	14.2	11.3	8.0

Note: 1 kg/m³ = 0.062 lb/ft³.

[a]Average specific gravity was determined by using a modified dry-back procedure.
[b]Optimum moisture content and maximum dry density were determined by using separate samples of material to develop each point on the moisture-density curve.
[c]Different charge weights were used according to residue gradations. Allowable value ranges from 40 to 50 percent, depending on use of aggregate (AASHTO specifications).
[d]Allowable value ranges from 10 to 15 percent, depending on use of aggregate (AASHTO specifications).
[e]Allowable value is 12 percent (AASHTO specifications).

practical way of measuring the amount of combustible and organic matter in incinerator residue is the loss-on-ignition test. Loss-on-ignition values from 2 to more than 15 percent can be expected. The chemical composition of a particular source of residue must, therefore, be adjusted to include the presence of carbon in the analysis.

Some analytic work has also been performed to identify the concentration of trace elements in residues that are potentially detrimental to the environment. Since leaching potential is a function of surface area and water solubility, an analysis was made of the fine fractions of residue samples from various types of incinerators to determine trace-element concentrations. Results of these tests show that, with the exception of zinc, the presence of trace elements in the finer-size fractions of incinerator residues is consistently low for each basic type of residue. This is particularly true for arsenic, cadmium, and selenium, of which concentrations less than 0.005 percent were generally noted. Furthermore, incinerator residues, and in particular the fine sieve fractions, were shown to be slightly basic, a condition that is more desirable from the standpoint of leaching than is acidity.

The combustion of municipal solid waste by pyrolysis produces gaseous fuel, tar and oils, and a char residue. The residue from the Landgard pyrolysis system in Baltimore is subjected to several levels of processing before its final use as aggregate in bituminous pavement. First, the lightweight carbon and ash fraction is separated by flotation. Ferrous metal is then reclaimed by magnetic separation. The remainder of the residue, approximately 60 percent by weight, is referred to as glassy aggregate (13). Although no chemical analysis is presented for the glassy fraction of pyrolysis residue, in all probability the material is highly siliceous with a substantial percentage of alumina. Information on the presence of trace elements in the glassy portion of the residue is not available. However, because of its slaglike structure, the material does not appear to have high potential for leaching.

Results of Aggregate Acceptance Tests

Standard soil and aggregate acceptance tests were performed to evaluate the potential of different sources of residue for applications in highway construction. Because of the heterogeneous nature of these materials and their tendency to absorb moisture, difficulties were encountered during some of these tests.

The procedures normally used in ASTM C 127 and C 128 for determination of specific gravity and absorption for conventional aggregate materials were not suitable for testing incinerator residues. Dry-back procedures similar to those described in ASTM D 2041 (Rice method) were adapted for determining the bulk specific gravity of incinerator residue samples. Because of variations in the composition of the material itself and the comparatively high percentage of lightweight particles present, a larger-size pycnometer was used in the test together with a proportionately increased sample size in order to ensure better reproducibility of test results. Although bulk specific gravities were found to vary somewhat for different samples from the same residue source, average values of specific gravity were found to increase as the degree of burnout improved.

The normal relation between moisture and density was not consistently applicable to the testing of incinerator residues. Sieve analyses performed before and after compaction clearly show that considerable particle degradation occurs when the same sample of residue is used throughout the entire moisture-density test. To counteract this, separate samples of residue should be used in determining individual moisture-density values in the test.

Besides particle degradation, the absorption and nonuniform distribution of moisture throughout the sample also contribute to the problem of determining the moisture-density relation of incinerator residues. For these reasons, small variations in dry density often occur over a wide range of moisture content values. It was ultimately concluded that a fully reliable value for the optimum moisture content of incinerator residue could not be determined solely on the basis of maximum dry density without some exercise in judgment by the operator of the test (14). Nevertheless, it is possible to attain a high degree of compaction with incinerator residue in the field despite fluctuations of moisture content in the material.

A number of other physical tests were performed on samples of incinerator residues. The most significant of these tests were the Los Angeles abrasion resistance and sodium sulfate soundness tests. Although all residue samples tested exhibited satisfactory abrasion loss, most specimens did not fully satisfy soundness criteria.

Table 3 summarizes the results of the physical tests performed on as-received samples of various types of incinerator residues. This table also includes results from tests performed on samples of pyrolysis residue. Because of the unusual behavior of incinerator residue, caution is advised in the use of data from standard test methods as evaluative criteria for determining the applicability of a particular source of residue for use in highway construction. Since the glassy fraction of pyrolysis residues is similar to a sandy slag, this material is more suited to the use of standard test methods than residues from conventional incinerator plants.

SUMMARY

Although residues from the municipal incineration of solid waste are heterogeneous materials, their physical composition and basic physical properties do fall within generally predictable limits. These characteristics are related to the degree of burnout of the residue, which can be determined to a great extent by the basic design of the incinerator plant. The degree of burnout is used as the basis for classifying residues in three categories and considering residues from pyrolysis operations as a special classification.

Well-burned and intermediately burned materials are considered acceptable for some type of use in highway construction. Poorly burned residues should not be used as received in any type of application but should be stockpiled for at least six months. Other residue materials, except pyrolysis residues, should be aged for at least one to two months after leaving the plant to reduce moisture content to an acceptable level prior to their use.

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