HIGHWAY RESEARCH RECORD

Number | Mineral Aggregates
353 | 5 Reports

HIGHWAY RESEARCH BOARD

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FOREWORD

Mineral aggregates, both naturally occurring and man-made, are of continuing interest to the highway engineer. This interest is evidenced by the 23 publications involving mineral aggregates listed in the January 1971 publications catalog of the Highway Research Board (5 of these are bibliographies).

The highway engineer must make judicious use of available aggregates to build the highways and structures required for this nation. Thus he needs to know the best way to use these materials and how to avoid problems with them. The five papers contained in this RECORD provide additional information on various mineral aggregates and how to avoid problems connected with their use.

Dolar-Mantuani discusses the late-expansion characteristics of some alkali-reactive carbonate rocks, noting that this late-expansion may not occur for some time and may be preceded by an early shrinkage in alkali solution. She points out that this late-expansion is distinctly different from the far more abundant early-expansion group of aggregates.

Larson et al. discuss the benefits of modifying the Los Angeles abrasion test by adding water to the sample during a portion of the test and then performing a sedimentation analysis (in addition to a loss determination) on the sample.

Bonifay et al. report on research involving the effects of different agglomerating and extruding aids on the properties of fly ash aggregates. They conclude that suitable aggregates can be made for use in asphaltic concrete surfaces and bases, unexposed portland cement concrete, portland cement concrete pavements, and base courses.

The potential availability of aggregate throughout the conterminous United States is assessed by Witczak et al. Aggregate availability ranging from "abundant-to-adequate" to "severe problem" was found and reported. The study showed that aggregates are in potentially short supply over about one-third of the conterminous 48 states.

The final report in this RECORD discusses the feasibility of reducing absorptive capabilities of aggregates by various chemical treatments. Results indicate the relative effectiveness of the chemicals investigated in reducing water and asphalt absorption into the aggregate.

—W. B. Ledbetter

CONTENTS

LATE-EXPANSION ALKALI-REACTIVE CARBONATE ROCKS

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Some alkali-reactive carbonate rocks of the Ordovician Gull River formation in Ontario are characterized by a shrinkage period preceding expansion when tested in 1N sodium hydroxide. These late-expansion rocks have a shrinkage period of at least 10 weeks, maximum shrinkage of at least 0.03 percent, and later minimum expansion of 0.20 percent. Petrographic examinations show the rocks to be sandy or silty argillaceous calcitic dolomites. Typical samples have 3 to 5 percent absorption, bulk specific gravity below 2.60, more than 20 percent acid-insoluble residues, and 75 to 87 percent dolomite in the carbonates.

•THIS PAPER reports on the continuation of the studies of carbonate rocks from the Ordovician Gull River formation, an important source of concrete aggregate, which outcrops in a narrow band along the southeastern border of the Canadian Shield in Ontario. Some Gull River rocks react with the alkalies in concrete and with alkali hydroxide solutions. Previous results of tests on these rocks in 1N sodium hydroxide were reported in 1964 (1). Many of the initial rock cylinders have now been tested for up to 8 years, and new samples have been added.

Many carbonate rocks shrink in alkali solutions before they start to expand (2, 3, 16). Five cylinders from one quarry in the area examined previously showed late-expansion characteristics that were distinctly different from those of the more abundant early-expansion group. Of this late-expansion group, 38 cylinders from four locations have now been tested. The expansion and petrographic features were compared with 248 other expansive and nonexpansive Gull River carbonate rocks. A few carbonate rocks from other Paleozoic formations were tested for comparison.

EXAMINATION METHODS

Expansion measurements on the rock cylinders essentially followed ASTM Designation C 586-69. Small pieces for petrographic studies were taken from the rock next to each cylinder. Thin-section analyses and acid-insoluble residue determinations using 20 percent hydrochloric acid were made on all samples. From 282 samples tested, the noncarbonate fractions of 84 were studied using X-ray diffraction by R. Laakso of the Ontario Department of Mines. Calcite and dolomite percentages were calculated for 98 samples, assuming that all of the MgO was present in dolomite. Magnesium and calcium were determined by wet chemical methods and by atomic absorption spectrophotometry. Calcite and dolomite percentages were also determined on -200 mesh material in 35 samples using evolution displacement (5) by W.O. Taylor of the Department of Mines. Differences in the calcite-to-dolomite ratio by the three methods were small. Information on porosity changes that occurred during NaOH exposure was obtained by absorption and specific gravity tests. These were made at atmospheric pressure and at 15 mm Hg (4) on small pieces of each late-expansion sample and at normal pressure on undamaged cylinders after 5 to 7 years of testing.

DISTINCTIVE CHARACTERISTICS

With the additional data obtained, a better definition of the late-expansion group can be made. The other groups have been redefined to conform with published data. In this

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paper, carbonate rocks are defined as late-expansion if they meet the following conditions: (a) the initial shrinkage lasts at least 10 weeks, (b) the maximum shrinkage is at least 0.03 percent, and (c) the later expansion is at least 0.20 percent. The 0.20 percent expansion limit is from Sherwood and Newlon (6, compare 17), but their time of 26 weeks for expansion was not adopted. A few carbonate rocks that shrank for less than 10 weeks or had minimal shrinkage were added to groups 1 or 2 listed in the following.

The few carbonate rocks were divided into four groups:

1. Early-expansion rocks (82 cylinders tested) with a maximum expansion of over 0.20 percent.
2. Rocks (35 cylinders) with insignificant expansion, up to 0.10 to 0.20 percent. These are part of the minor-expansion group defined in 1964 (1) as having maximum expansion of 0.10 to 0.38 percent. Cylinders expanding above 0.20 percent are now included in group 1. A lower limit of 0.10 percent was used by Buck (7).
3. Nonexpansive rocks (125 cylinders) with a maximum expansion below 0.10 percent.
4. Shrinking carbonate rocks (35 cylinders) with only negative values for length changes.

RESULTS OF TESTS

Out of 282 samples (315 cylinders) of the Gull River carbonate rocks from 22 locations, 34 are represented by 38 cylinders and belong to the late-expansion group. Thirty were taken from two quarries at one specific level in each quarry. Because this group was not recognized at first, tests of cylinders that did not expand were stopped after 52 or 70 weeks, and the rocks were classed as nonexpansive (1). Some may have been late-expanders, but retesting of a few after a lengthy dry period failed to show late-expansion characteristics.

Expansion Characteristics

Shrinkage measured shortly after immersion in NaOH followed by long-term expansion, characteristic of the late-expansion group, is given in Table 1. Expansion may be gradual, up to 4 years, or sudden. The lengthy delay before expansion starts concealed the reactivity of rocks tested for too short a time. The shrinkage and expansion trends of this group are given in Table 2.

The shrinkage is significant, and growth beyond the original water-stablized length may not occur for over 2 years. However, some length increase occurs sooner because the maximum shrinkage is usually passed after 6 months of testing, and then the negative values start to decrease. No relationships were found among the parameters given in Table 2, except that shrinkage values over 0.20 percent were associated with longer shrinkage periods. Petrographic characteristics help in recognition of the type.

The expansion characteristics of the late-expansion group are evident in comparison with the early-expansion group shown in Figure 1. The number of samples decreases with time because cylinders are withdrawn. The early-expansion cylinders usually expand in the first 6 months of alkali immersion and then length changes become smaller, but the late-expansion group may not start to expand in 6 months. This feature should make early distinction between the two groups possible, even though at later stages the confidence envelopes and means overlap. This is not critical, however, because the objective is to find early signs of potential alkali reactivity.

Whether a rock belongs to the late-expansion group or to one of the nonexpansive of slightly expansive groups cannot be decided early. The enlarged insert in Figure 1 shows this. The very narrow (up to ±0.02 percent) 95 percent confidence envelopes of the three nonexpansive groups are omitted, but that of the early-expansion group, which widens early and later is about the same as at the end of the first half year, is shown in the main graph.

Early recognition of the late-expansion group as being alkali-reactive is important. The difficulty in obtaining reliable results on expanding carbonate rocks at 2 and 5 weeks testing has been pointed out (Fig. 6 of 1). Late-expansion rocks are hard to identify

TABLE 1

LATE-EXPANSION CARBONATE ROCKS

Cylinder	Petrographic Data			Summary of Length Change Based on Yearly Values in Percent									Cracking[a]
	Insoluble Residue (weight percent)	Calcite/ Dolomite Ratio	Absorption (weight percent)	Immersion Time in NaOH in Years									
				$\frac{1}{4}$	$\frac{1}{2}$	1	2	3	4	5	6	7	
20	29.2	0.15	4.69	-0.142	-0.041	0.332	0.896	0.970	1.045	1.087	1.004		
21	25.9	0.22	4.66	-0.133	-0.099	0.390	1.078	1.095	1.128	1.170	1.103		
22	26.0	0.27	3.17	-0.074	-0.041	-0.042	0.788	0.987	1.144	1.235	1.211		
26B	21.1	0.30		-0.075	0.125	0.150	0.250	0.308					
112A	2.8	1.15	0.22	-0.025	-0.008	0.017	0.187	0.238	0.357	0.374	0.332	0.340	Crack
125A} 125B}	34.4	0.32	3.19}	-0.066 -0.053	-0.116 -0.080	-0.149 -0.134	0.098 -0.098	0.280 0.089	0.506 0.436	0.589 0.436	0.532	0.548	
126A} 126B}	25.9	0.19	4.30}	-0.016 -0.116	0.141 -0.099	0.656 0.357	0.856 0.839	0.856 0.922	0.914 0.972	0.955 0.956	0.764	0.764	Crack
127A} 127B}	48.5	0.20	3.57}	-0.041 -0.008	-0.107 0.076	-0.158 -0.051	1.261 0.358	1.966 0.417	2.356 0.604	2.348 0.613	2.165	2.174	Crack
181	30.1	0.22	4.55	-0.058	0.199	0.625	0.641	0.716	0.616	0.683	0.691		
182	31.7	0.25	4.87	-0.049	0.174	0.574	0.599	0.690	0.640	0.632	0.632		
183	35.2	0.19	5.20	-0.041	0.083	0.258	1.047	1.164	1.164	1.089	1.072		Crack
184	35.8	0.20	4.76	0	0.033	0.349	1.221	1.205	1.155	1.254	1.254		Crack
185	34.4	0.27	4.37	-0.016	0.348	0.581	0.573	0.698	0.540	0.615	0.615		
186A} 186B}	30.4	0.25	5.32}	0.008 -0.074	0.165 0.149	0.663 0.448	0.688 0.431	0.829 0.539	0.780 0.489	0.771 0.514	0.771 0.514		
187	33.7	0.22	4.98	0.033	0.632	1.123	1.114	1.198	1.164	1.114	1.123		
188	36.5	0.22	4.96	0.082	0.663	1.277	1.244	1.269	1.302	1.236	1.236		
189	33.5	0.22	5.28	-0.058	0.351	0.795	0.928	0.811	0.895	0.870			
195	8.2	0.40	1.52	-0.058	0	0.058	0.116	0.141	0.224	0.208			Crack
196	5.9	0.37	1.21	-0.033	0.033	0.058	0.091	0.075	0.158	0.141			
224	5.7	1.15	0.45	-0.024	0.024	0.149	0.224	0.157	0.215	0.249			
234	43.4	0.30	3.12	-0.016	-0.049	-0.041	0	0.448	0.431	0.414			Crack
235	29.4	0.27	5.20	-0.033	-0.083	-0.107	0.226	0.810	0.861	0.844			
236	26.9	0.33	4.35	-0.091	-0.108	-0.083	0.165	0.645	0.628	0.612			
237	28.9	0.25	4.96	-0.150	-0.197	-0.173	0.107	0.479	0.578	0.570			
238	35.5	0.19	3.40	-0.074	-0.074	-0.058	-0.091	0.232	0.215	0.232			
239	27.0	0.33	3.45	-0.091	-0.140	-0.108	0.091	0.563	0.505	0.497			
240	24.8	0.33	4.20	-0.132	-0.198	-0.223	0.099	0.637	0.538	0.538			
241	21.3	0.25	3.87	-0.100	-0.058	0.116	0.463	0.810	0.810	0.802			
242	28.2	0.23	4.32	0	-0.035	-0.035	0.261	0.732	0.819	0.828			
243	29.9	0.28	4.42	-0.083	-0.041	-0.091	0.298	0.967	1.108	1.100			Crack
244	29.3	0.25	4.70	-0.093	-0.126	-0.212	0.059	1.007	1.083	1.075			Crack
245	24.9	0.37	3.59	-0.036	-0.093	-0.009	0.188	0.599	0.527	0.527			
246	24.9	0.30	2.94	-0.034	-0.051	0	0.111	0.256	0.342	0.324			
247	25.8	0.35	3.33	-0.157	-0.132	-0.174	0.041	0.423	0.497	0.489			

[a]Cracking after length change test.

TABLE 2

LENGTH CHANGE PARAMETERS OF LATE-EXPANSION GROUP

Parameter	Mean	95 Percent Confidence Limit		Observed Range
		Lower	Upper	
Maximum shrinkage, percent	-0.13	-0.15	-0.11	-0.03 to -0.25
Time of maximum shrinkage, weeks	24	17	30	2 to 81
Time of first length increase from original length, weeks	48	38	58	11 to 119
Maximum expansion, percent	0.81	0.67	0.95	0.24 to 2.36
Time of maximum expansion, weeks	194	182	207	93 to 278

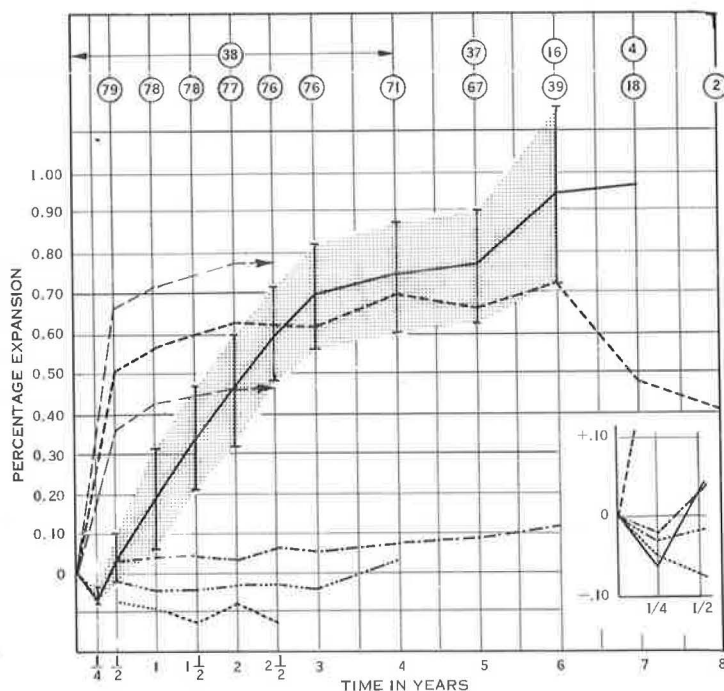Note: Data are based on maximal values.

Figure 1. Expansion versus time for Gull River carbonate rocks in NaOH immersion. Mean and 95 percent confidence for late- and early-expansion groups are indicated by stippling and dashed lines with arrows respectively. Numbers of cylinders tested at various times are indicated in circles at top (upper row is for late-expansion and lower row is for early-expansion cylinders). Enlarged area at lower right corner is for time of 0 to ½ year and expansion of -0.10 to + 0.10 percent. Mean: —— late-expansion cylinders; ----- early-expansion cylinders; _._._. cylinders suspected of being alkali-reactive; ————···nonreactive cylinders; ··········· only shrinking cylinders.

when tested at 12 or 26 weeks (6,7). Figure 2 shows the expansion after 12 and 26 weeks related to expansion after 1 year. The 12-week graph shows that none of the late-expansion cylinders expanded 0.10 percent, which is the basis for classifying rocks as suspected of being alkali-reactive, and that 23 of the 82 early-expansion cylinders might not have been recognized. The 26-week graph shows that about one-fourth (10) of the late-expansion cylinders went from shrinkage to expansion and that most of the early-expansion cylinders were expanding, with 50 of them expanding more than 0.20 percent. The points in the lower left corners of the graphs represent cylinders that were still shrinking after 1 year of immersion in NaOH.

In the 12-week graph, the plots of the late-expansion group are scattered in the shrinkage area; in the 26-week graph they begin to align in the same direction as the early-expansion group. The best-fit regression line, for the early-expansion group only, is similar in both graphs. A comparison of the data in both graphs by student's t-test shows that there is no significant difference in time between the means of the early-expansion group at the 95 percent confidence level. The 12-week results are apparently satisfactory for recognition of the early-expansion group. But significant expansion at a later stage is often not seen until after 6 months of testing.
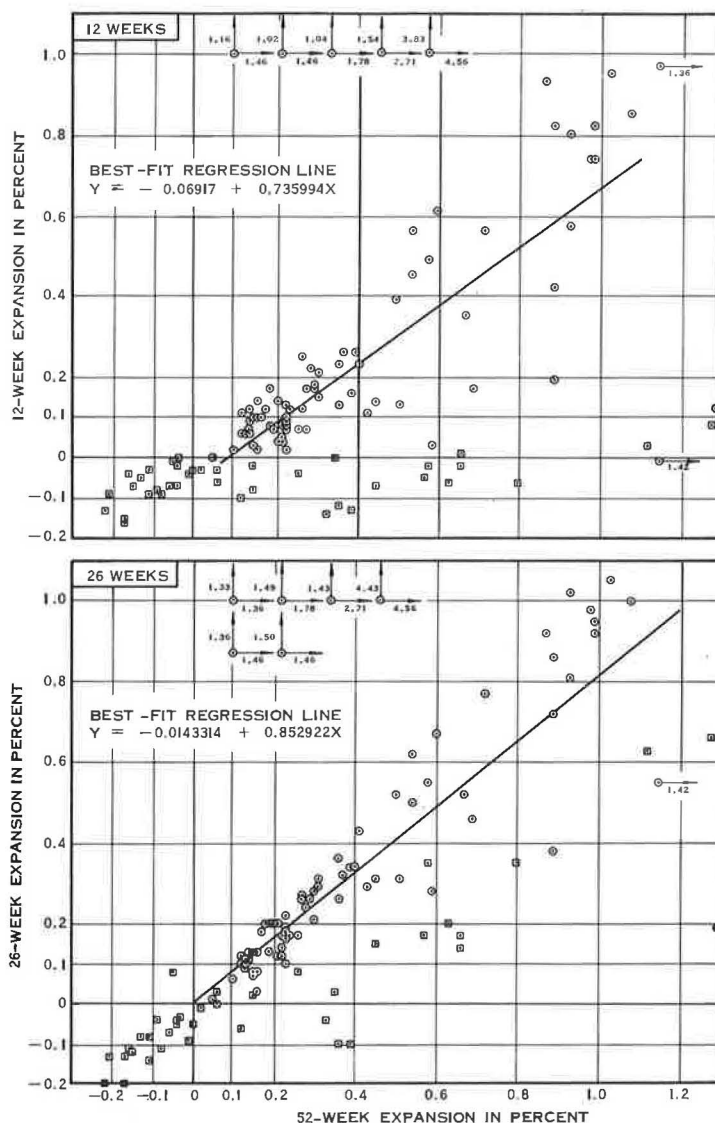
Figure 2. Expansion comparison at 12 and 26 weeks versus 52 weeks. Square denotes late-expansion cylinder; circle, early-expansion cylinder; solid line shows best-fit regression line for early-expansion cylinders only.

Influence of Cylinder Size on Expansion

Three cylinders, 145 mm in length and 50 mm in diameter, comparable in size to the prisms used by Swenson and Gillott (14), were drilled from two adjacent and parallel areas in a late-expansion layer and one from an early-expansion layer in the same quarry. Sets of four normal small-sized cylinders were drilled alongside of each of the large cylinders to compare the expansions with the expansions of the corresponding large cylinders. Figure 3 shows that the expansion of adjacent cylinders and the means of the adjacent sets were similar, but that the expansions of the large and the small cylinders were different. Small late-expansion cylinders had 22- and 15-week shrinking periods,

whereas large cylinders had only 8- and 9-week delays in expansion. The examples show the great variability in results obtained by the sodium hydroxide immersion method.

Damage During Testing

Cracks were observed on 9 out of 38 late-expansion cylinders. The cracks did not correlate with the amount or time of expansion or shrinkage, absorption characteristics, or amounts of acid-insoluble residue (Table 1). The usually shallow cracks occurred mostly in the central area of the cylinders. They could be wavy, branching, offset, or repeated at two (rarely three) levels. Only one cylinder (maximum expansion 1.35 percent) with one deep crack was bent. One cylinder had a fine network of hair cracks in a sandy portion, and another had a hair crack along three limonite concentrations, demonstrating the influence of compositional and textural weakness points.

Figure 3. Comparison of expansion versus time of three large cylinders (A,B,C) and means of small cylinders (a, b,c) drilled from rock along side the large cylinders.

Unlike the early-expansion group, of which five cylinders broke early and many were cracked at the end of testing, none of the late expansion cylinders broke, which indicates greater resistance to alkalies. This is probably due to the greater porosity or slower rate of volume changes or both during the early stages of testing.

Indentations due to leaching were visible in small areas of a few cylinders (Fig. 4). The increase in absorption after testing shows that leaching occurred during NaOH immersion. The rocks are bleached after testing because of the loss of organic matter. The originally medium light gray (N6) of Munsell (11), rarely light gray (N7) grading to greenish gray (5GY6/1), became light to very light gray (N8) with slight greenish or more grayish hues in the aphanitic areas. Some cylinders contain spots of limonite from pyrite.

PETROGRAPHIC CHARACTERISTICS

To characterize the late-expansion group, the following petrographic features of each sample were considered: specific gravities, absorption, acid-insoluble residue, calcite-to-dolomite ratio, noncarbonate minerals, texture, and petrographic classification.

Physical Properties

The specific gravities and absorption determined at atmospheric pressure on single pieces of samples before testing and on undamaged cylinders after testing are given in Table 3. Absorption and specific gravities of a few undamaged early-expansive cylinders and several nonexpansive or slightly expansive cylinders were determined for comparison.

High absorption values and, for dolomites, relatively low bulk specific gravities are typical for the late-expansion

Figure 4. Etching marks on cylinder 169 tested for 5 years (scales in millimeters).

TABLE 3

SPECIFIC GRAVITIES AND ABSORPTION OF LATE-EXPANSION GROUP

Characteristic	Mean	95 Percent Confidence Limit		Observed Range[a]
		Lower	Upper	
Atmospheric pressure, 24-hour immersion in distilled water				
Specific gravity, oven-dry	2.50	2.47	2.54	2.41 to 2.59
Specific gravity, saturated surface-dry	2.61	2.58	2.64	2.54 to 2.67
Specific gravity, apparent	2.80	2.78	2.83	2.75 to 2.85
Absorption, weight percent	4.27	3.99	4.55	2.94 to 5.32
15 mm Hg vacuum, $23\frac{3}{4}$-hour immersion in distilled water[b]				
Specific gravity, oven-dry	2.51	2.47	2.54	2.42 to 2.59
Specific gravity, saturated surface-dry	2.62	2.59	2.64	2.55 to 2.68
Specific gravity, apparent	2.82	2.80	2.85	2.78 to 2.86
Absorption, weight percent	4.52	4.21	4.83	3.08 to 5.65
Rock cylinders after test, atmospheric pressure[c]				
Specific gravity, oven-dry	2.28	2.24	2.33	2.14 to 2.55
Specific gravity, saturated surface-dry	2.41	2.37	2.45	2.30 to 2.67
Specific gravity, apparent	2.61	2.58	2.64	2.54 to 2.87
Absorption, weight percent	5.57	5.06	6.07	3.61 to 7.44

[a]Without specific gravities of sample 234 (2.82 to 3.12), which contains celestite (true specific gravity 3.95 to 3.97).
[b]29 samples.
[c]25 undamaged cylinders, 4 to 7 years tested.

rocks. This holds true for all typical samples regardless of their original locations within two quarries 140 miles apart. The 3 to 5 percent absorptions differentiate the late-expansion group from the early-expansion group (6, 8). Such high absorption values and low bulk specific gravities usually indicate that the rock is physically unsuitable for use as concrete aggregate, but if no information is available on its reactivity it is usually tolerated if it occurs only in a few layers. Low bulk specific gravities and high absorption values are not always related to high maximum-expansion values (10), but samples with less than 0.60 percent expansion have absorption below 3.6 percent.

The range in the difference between absorption under normal and low pressures is 0.04 to 0.57 percent, which is comparable with values found in fresh, vuggy, medium-grained pure dolomites. This may indicate that these porous rocks contain some fine capillaries. There is no correlation between the absorption values and these differences in absorption.

The great increase in absorption of cylinders after testing 5 to 7 years (0.75 to 2.75 percent), in comparison with the absorption of the rock as received, indicates that significant leaching has taken place during immersion in NaOH; this effect has also been reported by others (9, 10, 18). The increase is greater in cylinders with expansions over 1 percent than in the others. Two cylinders showing only shrinkage had absorptions of 0.35 percent after $1\frac{1}{2}$ years of testing.

Texture and Composition

The principal constituent of late-expansion carbonate rocks is dolomite. Calcite, quartz, and clay minerals are major or minor constituents; feldspars, pyrite, limonite, and sulfates are accessory minerals. Single scales of mica and grains of tourmaline are visible in the thin sections.

The euhedral dolomite is 0.01 to 0.20 mm in grain size. The coarser rhombohedrons are finely dispersed in the finer grained mass (Fig. 5). The variation of grain size of dolomite suggests a second generation of a few larger sized dolomite grains. The small dolomite euhedra are clear, and clayey impurities in the larger grains are usually
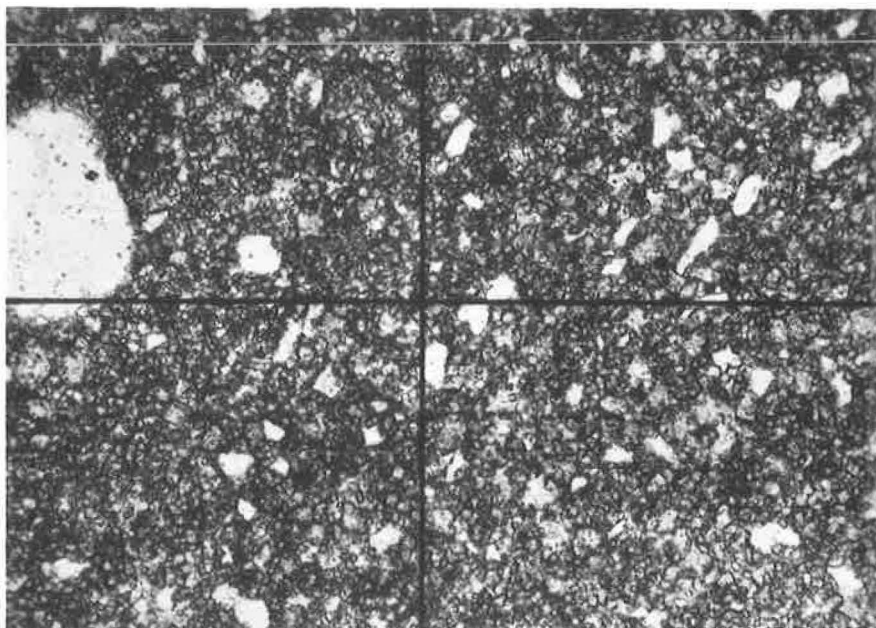
Figure 5. Large dolomite rhombohedrons and clear silt-sized detrital quartz seen in a matrix of very fine crystalline dolomite, calcite, and clay. Sand-sized grain is quartz (Sample 247; 88X).

scarce. Calcite occurs in the fine crystalline to aphanocrystalline (size less than 0.004 mm, 12) matrix together with dolomite and clay. Calcite may form larger patches of single grains typical of the poikilitic texture, with dolomite and rarely quartz as inclusions.

The rounded or angular silt to fine sand grains of quartz (0.01 to 0.10 mm) are scattered throughout the rock, concentrated in small areas (Fig. 6), or form a netlike structure. Small patches may give an impression of a dolomitic siltstone or sandstone. Quartz is always clear with uniform extinction. Fresh detrital feldspars of the same size as quartz occur in all samples in moderate or minor amounts. Potash feldspars are more abundant than plagioclases, the latter occurring as minor constituents or in traces. Most of the thin sections contain a few rounded quartz grains and, rarely single feldspar grains up to 1.0 mm in size.

Clay occurs in lenses, pockets, or interstices. The rare stylolites are usually very narrow concentrations of clay minerals and organic matter with a few detrital quartz grains. The clay minerals are represented mostly by illite and subordinately by chlorite. In three samples relatively rich in quartz, clay minerals are present in amounts not detectable by X-ray. Earlier X-ray examinations of five samples by Laakso indicated the presence of an expanding clay mineral in these samples. This is unlikely to be responsible for the expansive properties of the rock, however, because of the very small amounts involved (1).

Pyrite in single crystals or small concentrations may be partially altered to limonite; both minerals are ubiquitous. Sulfates are present in all samples, at least in traces. Five samples had more than 0.2 percent of sulfur trioxide in the filtrates from the acid-insoluble residues; one sample had 1.7 percent. Gypsum was not observed in the thin sections, but celestite was found in the 0.05- to 0.20-mm sizes of irregular grains as an accessory constituent.

The approximate amounts of quartz, illite, chlorite, potash feldspar, and plagioclases were established by Laakso by X-ray methods. There are variations in the
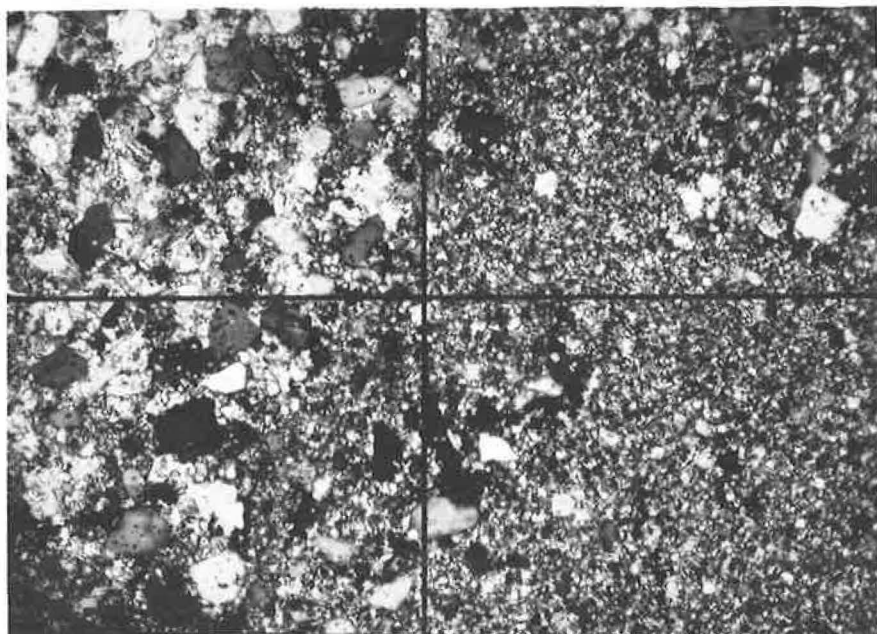
Figure 6. Very fine crystalline dolomite, calcite, and clay (right side) and accumulation of detrital silt-sized quartz (clear, uniform grains) and larger dolomite rhombohedrons in a very finely grained matrix (left side) (Sample 184; crossed nichols, 88X).

amounts of the constituents, some seemingly influenced by the source location of the samples. For example, rocks from one of the main quarries were poorer in clay minerals and contained very little chlorite and only traces of sulfates, whereas rocks from the other quarry always contained chlorite and sulfates usually in weighable amounts. One general difference between the late- and early-expansion rocks is the significant presence of quartz in the former. In agreement with the massive structure of the beds containing late-expansion rocks, bedding is usually not easily discernible in the samples.

On the basis of the calcite-to-dolomite ratio and the noncarbonate constituents, the rocks are classified as sandy or silty argillaceous calcitic dolomites. They are similar to the 24- to 30-ft bed in a Kingston quarry described in detail by Gillott (13).

Graphic Presentation

To obtain better characterization of the composition of the late-expansion rocks, the quantities of dolomite, calcite, and noncarbonates (acid-insoluble residue) have been plotted. This simplified composition is presented in a triangular plot like those used by Sherwood and Newlon (6), with corners representing 100 percent of calcite, dolomite, and acid-insoluble residue plus SO_3 calculated as gypsum (Fig. 7). Plots of the early-expansion rocks and a few nonexpansive or slightly expansive rocks are included in Figure 7.

The late-expansion group is concentrated in a small area along the low calcite side, indicating little variation in the composition of the group. Determinations of the three components of 97 Gull River samples show that only two early-expansion and two nonexpansive rocks are located in this area. The early-expansion rocks are widely scattered in the lower portion of the triangle and are concentrated at the calcitic corner. The few plots of suspected expansive (x) and of shrinking (+) carbonate rocks are located in areas of expansive rocks. They meet the main compositional characteristics of expansive rocks but not the additional complex conditions for distinct alkali reactivity.
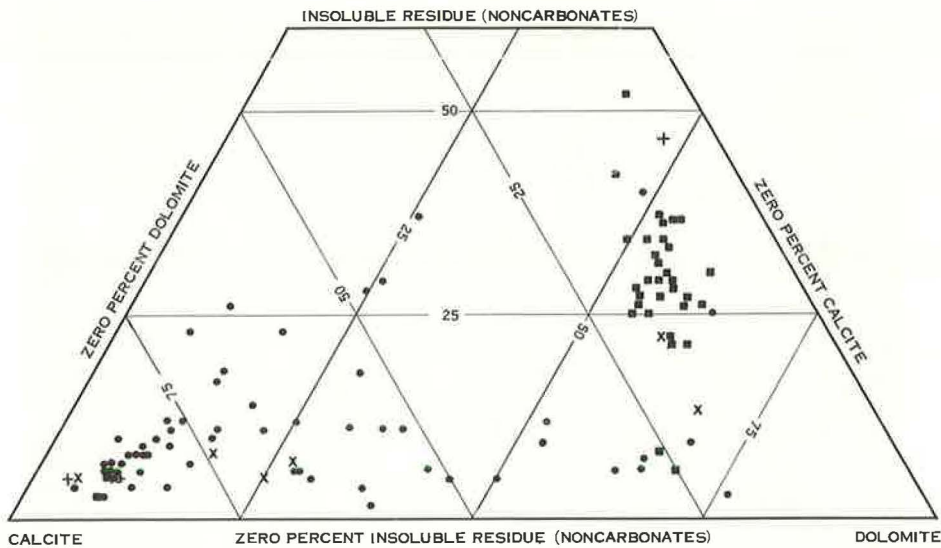
Figure 7. Ternary plot of composition of test samples. Solid square denotes late-expansion cylinder; solid circle, early-expansion cylinder; x, cylinder suspected of being expansive; +, nonexpansive cylinder.

RELATIONSHIP BETWEEN PETROGRAPHIC AND EXPANSION CHARACTERISTICS

Figures 8 and 9 show the compositional parameters related to the degrees of expansion. The amounts of acid-insoluble residue are between 21 and 49 percent. The upper limit of insoluble residue content is much higher for the megascopically uniform Gull River rocks than has been assumed (1). Most of the rocks contain from 24 to 36 percent insolubles, whereas most of the early-expansion Gull River rocks contain less than 13

TABLE 4

CHARACTERISTICS OF BORDERLINE LATE-EXPANSION ROCKS

Characteristic	Samples[a]				Late-Expansion Group[b]
	112A	195	196	224	
General properties					
Maximum shrinkage, percent	-0.03	-0.08	-0.04	-0.07	-0.03
Time of maximum shrinkage, weeks	22	9	15	9	
Duration of shrinkage, weeks	41	22	23	20	10
Maximum expansion, percent	0.41	0.24	0.26	0.29	0.20
Time of maximum expansion, weeks	236	185	185	163	—
Damage in NaOH	crack	crack	nil	nil	—
Petrographic properties					
Bulk specific gravity	2.70	2.71	2.73	2.69	2.50
Absorption at atmospheric pressure, weight percent	0.22	1.52	1.21	0.45	4.27
Absorption at 15 mm Hg, weight percent	0.24	1.59	1.29	0.49	4.52
Acid-insoluble residue, weight percent	2.8	8.2	5.9	5.7	31.5
Calcite/dolomite ratio	1.15	0.40	0.37	1.15	0.27

[a]Sample designations are as follows: 112A, patchy (biopelmicritic) limestone; 195, medium crystalline dolomite (V:D4); 196, medium crystalline dolomite (V:D4); and 224, aphanitic limestone (d:III mX:L1).
[b]Included for comparison; general properties given are minimum requirements for late-expansion group; petrographic properties given are mean values of typical late-expansion rocks (compare table 3).

percent; 25 cylinders with an expansion of more than 0.50 percent have acid-insoluble residues between 5 and 13 percent.

Figure 9 shows the restricted range for dolomite in the late-expansion group. These rocks have 75 to 87 percent dolomite in the carbonate constituents. They fall in the range of 70 to 90 percent dolomite that is reported to lack expanding representatives when rocks from Virginia were examined (6). Few expansive carbonate rocks from the Gull River formation in Ontario have between 40 and 70 percent of the mineral dolomite (14, 13, 8). There is no relationship between the amounts of expansion and the percentages of acid-insoluble residues and dolomite (16).

Some Exceptions

Late-Expansion Rocks Not Having Essential Petrographic Features—Table 4 shows that four carbonate rocks are late-expanders by their expansion characteristics but not according to their petrographic properties (Fig. 7, 8, 9). The two dolomitic rocks occur in small areas in layers of early expansion rocks, and the two limestones occur in a quarry of apparently nonexpansive rocks.

Rocks Having Several Essential Petrographic Features But Not Classified as Late Expanders—Seventeen samples have residues over 20 percent typical of late-expansion rocks, but they are early-expansive, nonexpansive, or very slightly expansive. Chemical analyses showed that five are limestones and four do not meet the defined expansion requirement or were tested for too short a time. One dolomite from the contact between early- and late-expansion rocks contains only a few lenses typical of the latter group.
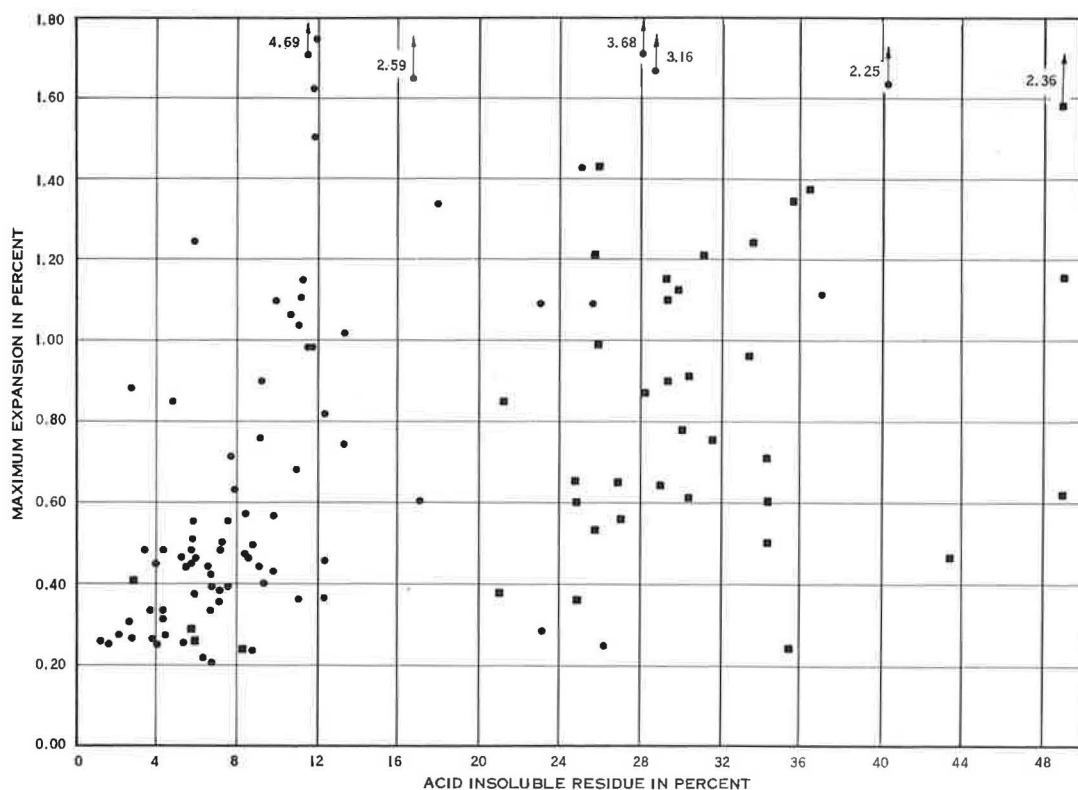
Figure 8. Acid-insoluble residue versus maximum expansion. Solid square denotes late-expansion cylinder; solid circle, early-expansion cylinder.
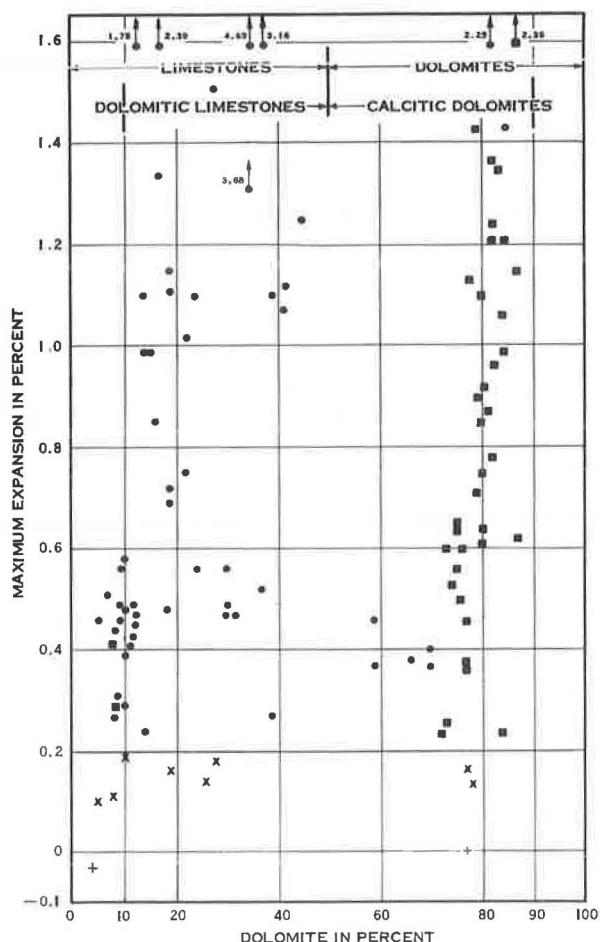
Figure 9. Dolomite in percent of carbonate portion versus maximum expansion. Solid square denotes late-expansion cylinder; solid circle, early-expansion cylinder; x, cylinder suspected of being expansive; +, nonexpansive cylinder.

The remaining seven samples do not conform to the late-expansion group on the basis of detailed thin-section examinations.

To establish minute microscopic differences or similarities, a comparison with thin sections of late-expansion rocks is essential. To establish that a rock belongs to the late-expansion group in principle may be possible, but it is impossible to predict how much it will expand, not even by means of the least-squares from petrographic parameters.

SUMMARY AND CONCLUSIONS

The late-expansion alkali-reactive carbonate rocks of the Gull River formation are well defined by their expansion and petrographic characteristics, but rocks from elsewhere may show variations. When immersed in 1N sodium hydroxide, this group is separated from the early-expansion group by a shrinkage of at least 0.03 percent for at least 10 weeks, during which the cylinders are shorter than the original water-stabilized length, and by a later expansion of a minimum of 0.20 percent. The maximum shrinkage period was more than 27 months.

Early recognition of the late-expansion group was not possible because no sample expanded enough to be classified suspect after 12 weeks of testing. Some may not expand after testing for 1 year. In contrast, the early-expansion rocks usually expand within 6 months. However, a sigmoidal curve representing the length changes of the late-expansion group, together with distinct petrographic features, should be helpful in detecting this type of rock after 6 months of testing.

The group is characterized by an absorption of up to 5 percent, with bulk specific gravity below 2.60, acid-insoluble residue from 21 to 49 percent, and a dolomite content of 75 to 87 percent carbonates. The high percentages of acid-insolubles indicate poor interlocking of carbonates and a low structural rigidity (10). The narrow range of the dolomite content is striking, but this may be because most of the late-expansion samples came from one distinct level in two quarries.

Petrographically, the rocks are fine-grained sandy or silty argillaceous calcitic dolomites. The noncarbonate minerals include quartz and clay minerals—mostly illite and subordinately chlorite, potash feldspar, and plagioclase. Pyrite, limonite, sulfates, gypsum, and, in five samples, celestite are accessory constituents or are present in traces. Organic matter may be concentrated in rare stylolites. Detrital quartz is one of the major noncarbonate constituents. Other Gull River carbonate rocks with a high content of noncarbonates do not have much detrital quartz.

Four samples showed length changes in NaOH that were characteristic of the late-expansion group, but the samples also had some petrographic features typical of early-expansion rocks. Twelve samples had most of the petrographic features typical of the late-expansion rocks, but they did not have their expansion behavior. Such borderline cases reflect the normal complexity of rocks. The thorough study necessary for finding fine differences usually cannot be performed during normal examinations of rocks intended for use in concrete.

The expansive Gull River rocks were apparently sedimented under different environmental conditions. The late-expansion rocks were deposited under conditions of higher energy than the more abundant aphanitic dense early-expansion rocks (15). The alkali reactivity in both groups, however, is comparable.

Results obtained on different sizes of rock specimens may differ—a fact established for the carbonate rock aggregates in concrete (14). Standardization of size is strongly recommended for comparison of test results from different laboratories. Results on cylinders of two sizes illustrate the difficulties in determining acceptance limits. They further support the need for testing what are suspected to be alkali-reactive carbonate rocks in concrete before accepting them for use as concrete aggregates.

ACKNOWLEDGMENTS

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MODIFICATION OF THE STANDARD LOS ANGELES ABRASION TEST

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The present Los Angeles abrasion test, commonly referred to as the Los Angeles rattler or LAR test, is satisfactory for determining the resistance of an aggregate to dry abrasion. However, some state highway departments have had to develop tests of their own for determining wearability of aggregates under wet abrasion conditions. This paper proposes a modification to the standard LAR test to include 250 revolutions with the aggregate in the dry state plus 250 revolutions after a fixed amount of water has been added. A sedimentation analysis is then run on the entire sample. If the sample passes the sedimentation analysis within specified limits, it is then oven-dried and placed in a sieve shaker. The percent loss through a No. 16 sieve is then determined. Results obtained from the testing of numerous samples, when compared with the Washington degradation test, the standard LAR test, and petrographic analysis, indicate that the proposed modification of the standard LAR test has considerable merit.

•THE NEED for quality tests as a measure of the degree of suitability of aggregates for construction purposes was first recognized in 1780 by Higgins (1). He stated, "I have thought that the small stones, which constitute the gravel chosen for our roads, could not be reduced to dust so soon as they now are by the heavy carriages...."

The Scottish engineer and road builder, John McAdam, noted in 1819 that the workmen determined the proper size of stone for road-building purposes by measuring them with their fists (2).

The Laboratoire des Ponts et Chaussees in Paris developed the first abrasion test for stone in 1870. In 1878 Deval invented a test to determine the resistance of mineral aggregates to abrasion. The American Society for Testing and Materials approved the Deval abrasion test in 1908 (3).

The first highway research to be carried out in the United States was in 1893 at Harvard University under the direction of L. W. Page. He developed a test for the cementing value of broken stone dust and a test for the toughness of aggregates by the Page impact machine (3).

As aggregate testing became standard practice in materials engineering, shortcomings of the Deval test were found. These inadequacies led to the development of the Los Angeles rattler (LAR) test in 1916 by the Los Angeles City Engineer's office (3). This test was rapid and simple and provided consistent results, as confirmed by circular track tests (4). For these reasons the American Society for Testing and Materials adopted the Los Angeles rattler test as a standard test in 1939.

Goldbeck (5) and Woolf (6) observed that there appeared to be a definite relationship between the results of the Los Angeles test and the corresponding road performance record. However, failures occurred on roads containing aggregates that had passed the LAR and other required tests.

The present Los Angeles rattler test determines the abrasive and structural characteristics of dry aggregates, but it is unable to identify some aggregates that have failed because of the large amount of plastic fines that are produced when these aggregates degrade in the presence of water. As a consequence, there have been many attempts to develop degradation tests with water added.

In 1966 Breese did extensive research for the state of Nevada in correlating existing degradation tests with a view toward possible development of a new test (7). The tests that he correlated included the elutriation test devised by Collins (8), the jar-mill test devised by Minor (9), the Washington degradation test (10), and the California aggregate durability test (11).

It is important to note that all four of these degradation tests use a sedimentation analysis of the fines produced. This analysis is based on the principle of Stokes' law (12), which states that the theoretical velocity of vertical settling for a particle can be computed by the following formula:

$$V = gd^2 \frac{(D_1 - D_2)}{18u}$$

where

g = acceleration due to gravity, cm/sec/sec;
D_1 = density of settling particle, gram/cm^3;
D_2 = density of water, gram/cm^3;
d = diameter of settling particle, cm; and
u = dynamic viscosity of water, dyne-sec/cm^2.

By analyzing Stokes' law it can be noted that the larger and denser particles have a greater settlement velocity and therefore will travel farther in a specified amount of time. For this reason, in a given time period, coarse-grained particles would provide lower sediment heights than fine-grained particles.

In all four of the degradation tests that Breese studied, a representative sample of the fine particles produced during the mechanical agitation of the aggregate was poured into a sand-equivalent test cylinder. Seven milliliters of sand-equivalent stock solution were added, and the cylinder was filled with water to the 15-in. mark. The cylinder with its contents was thoroughly mixed by 20 inversions in 35 sec and then allowed to settle for 20 min. The sediment (floc) height was then read. Poor aggregates that tend to produce plastic fines were indicated by high sediment heights.

Breese correlated the degradation factors obtained by the various methods and also the sediment heights obtained by each method. Using the linear regression method of correlation, he found that the Washington degradation test produced the highest coefficients of correlation when compared with the other tests (7).

Platts and Lloyd (13) studied six degradation tests in Alaska. From this study, the Washington degradation test was recommended for the following reasons:

1. Test results and field evaluations correlated very well.
2. The test has but one numerical value on which to base the quality of any given aggregate.
3. Although similar to the California durability test, it provides more consistent results.

In 1968 the authors developed a modified LAR test that included wet abrasion, loss determined by a sieve in the standard fine-aggregate sieve series, and a sediment-height test using a specially designed Plexiglas cylinder, 6 in. in diameter by 25 in. high. Conclusions were as follows:

1. The modified LAR test appeared to be as reliable as the Washington degradation test in determining unsatisfactory aggregates.
2. It is not necessary to sieve out the coarse particles before sedimentation, because these settle out very rapidly and have minor effects on the settlement of the fines.

3. The modified LAR test does not produce as many plastic fines as the Washington degradation test; however, the results indicated similar categorization of aggregates.

4. The modified LAR test uses the No. 16 sieve, which is in the standard fine-aggregate sieve series.

Breese's report states that there appears to be universal agreement that the best method for determining potential degradation is by petrographic analysis (7). The drawbacks to such an approach are that it is time-consuming, requires the services of a trained petrographic expert, and is, in general, not adaptable to field laboratory use.

OBJECTIVES OF INVESTIGATION

The objectives of this investigation were (a) to compare the modified LAR test with the standard LAR test, the Washington degradation test, and the petrographic analysis; (b) to determine a sediment-height correction factor for mixture temperature in the sedimentation portion of the modified LAR test; and (c) to determine the reproducibility of the modified LAR test.

AGGREGATES TESTED

In this study aggregate tests were conducted on 51 samples from Idaho, Montana, and Washington supplied by the U.S. Forest Service, Region 1, Missoula, Montana. Most of the samples were crushed basalt, siltstone, quartzite, granite, granodiorite, schist, or gneiss. The major minerals or rocks that composed each of the 51 samples were determined by petrographic analysis. Some additional crushing was necessary to obtain the B gradation used for the modified LAR test.

NEW WASHINGTON DEGRADATION TEST

The procedure used for the new Washington degradation test (10) is as follows:

1. The material to be tested shall be crushed to pass the ½-in. sieve, washed over a No. 10 sieve, and dried to constant weight. Prepare samples graded as follows: ½ in. to ¼ in., 500 grams; ¼ in. to No. 10, 500 grams.

2. Place sample in a 7.5-in. diameter by 6-in. high plastic cannister (Tupperware), add 200 cc water, cover tightly, and place in a Tyler portable sieve shaker (Soiltest C1-300 and 305, suitably motorized to provide agitation described in step 3).

3. Run shaker for 20 min at 300 ± 5 oscillations per minute with a 1.75-in. throw on the cam. At the conclusion of the shaking time, empty the cannister into nested No. 10 and No. 200 sieves placed in a funnel over a 500-ml graduate to catch all water. Wash out the cannister and continue to wash the aggregate with fresh water until the graduate is filled to the 500-ml mark. Caution: The aggregate may drain 50 to 100 ml of water after washing has been stopped.

4. Pour 7 ml of sand-equivalent stock solution into a sand-equivalent cylinder. Bring all solids in the wash water into suspension by capping the graduate with the palm of the hand and turning the cylinder upside down and right side up as rapidly as possible about 10 times. Immediately pour the liquid into the sand-equivalent cylinder to the 15-in. mark and insert the rubber stopper in the cylinder.

5. Mix the contents of the sand-equivalent cylinder by alternately turning the cylinder upside down and right side up, allowing the bubble to traverse completely from end to end. Repeat this cycle 20 times in about 35 sec.

6. At the conclusion of the mixing time, place the cylinder on the table, remove the stopper, and start the timer. After 20 min, read and record the height of the sediment column to the nearest 0.1 in.

Calculation of the new Washington degradation factor is as follows:

$$D = \frac{(15 - H)}{15 + 1.75H} \, 100$$

where D is the new Washington degradation factor and H is the height of sediment (floc) in the cylinder. In this formula the values may range from 0 to 100, with the high values being the best materials. The formula was adjusted to place doubtful materials at about the midpoint of the scale. At the present time, the minimum Washington degradation factor for surface aggregates is 25. All of the Washington degradation factors were determined at the U.S. Forest Service Materials Testing Laboratory in Missoula, Montana. Table 1 gives these data.

STANDARD LOS ANGELES RATTLER TEST

The standard LAR test procedure with B graded material was used according to the American Society for Testing and Materials. The B gradation consists of 2,500 grams

TABLE 1
DEGRADATION TEST RESULTS

Sample No.	Standard LAR No.	Modified LAR Test		Washington Degradation Test	
		Modified LAR No.	Sediment Height at 20 C (in.)	Sediment Height (in.)	Degradation Factor
1801	37	23	6.0	1.3	79
1805	26	29	13.1	9.3	18
1887	17	17	14.5	12.1	8
1909	17	16	6.2	2.7	53
1922	16	14	6.7	2.4	66
1953	18	18	13.5	12.8	6
1959	18	18	6.0	1.7	74
1960	20	17	7.3	2.2	68
2022	—[a]	19	5.7	—[a]	—[a]
2026	25	24	12.2	4.9	43
2074	13	13	7.0	1.1	82
2075	19	18	6.3	0.5	91
2076	15	15	6.1	3.8	52
2079	20	18	14.1	13.8	3
2080	17	19	9.6	2.7	62
2094	38	42	10.1	2.4	66
2096 (2583)	31[b]	28	10.8	8.7	21
2122	20	21	12.3	1.0	84
2123	42	45	7.9	2.0	70
2124	31	27	7.4	1.3	79
2125	21	19	6.1	1.8	73
2126	22	21	13.8	11.4	11
2137	34	30	7.2	1.6	75
2138	35	30	9.8	1.0	84
2139	15	17	6.8	0.4	93
2141	85	69	10.1	0.9	85
2142	60	56	7.8	1.2	81
2144	33	38	7.7	1.1	82
2145	35	33	10.5	2.7	53
2148	23	26	19.1	13.5	4
2180	26	25	15.8	0.8	87
2181	18	18	6.3	1.4	78
2182	22	20	6.7	0.7	88
2183	27	28	6.3	0.9	85
2187	17	18	14.6	11.8	9
2222	17	20	10.5	2.8	61
2516	29	30	13.7	4.2	48
2555	22	20	6.1	2.8	61
2556	28	23	8.0	1.5	77
2557	36	33	7.8	2.8	61
2561	34	30	13.8	8.2	23
2562	68	57	9.7	10.1	15
2563	25	23	10.7	1.6	75
2584	20	21	16.1	11.5	10
2635	23	20	9.7	5.9	36
2756	39	35	9.4	2.4	66
2757	36	42	11.3	5.0	42
2758	26	22	8.8	1.8	73
2759	17	15	6.8	2.0	70
2760	16	17	13.6	13.5	4
2761	28	28	13.2	13.5	4

[a]Data not available.
[b]Obtained from different sample (number in parentheses) but same source.

of material passing the $\frac{3}{4}$-in. sieve and retained on the $\frac{1}{2}$-in. sieve plus 2,500 grams passing the $\frac{1}{2}$-in. sieve and retained on the $\frac{3}{8}$-in. sieve. The abrasive charge consists of 11 steel spheres having a total weight of 4,584 ± 25 grams.

The maximum amount passing the No. 12 sieve for the standard LAR test has been specified as 40 percent for surface courses and surface treatment. A maximum passing number of 50 percent has been established for concrete, base courses for concrete pavements, and bituminous macadam pavements.

All of the standard LAR numbers given in Table 1 were determined at the Forest Service Materials Testing Laboratory in Missoula.

MODIFIED LOS ANGELES RATTLER TEST

An initial study by the authors in 1968 established a modified Los Angeles rattler test procedure as follows:

1. Prepare the sample in the same manner as required for the standard LAR test using the B grading.
2. Place the test sample and the abrasive charge into the Los Angeles testing machine. Rotate the machine for 250 revolutions with the aggregate dry. Add 1,000 ml of water and rotate for 250 additional revolutions. A flat rubber gasket cemented to the cover of the machine makes it watertight.
3. Wash the entire contents of the machine into a large pan placed beneath the machine. Then, wash the entire contents of the pan into the large specially designed graduated cylinder previously prepared with 25 ml of sand-equivalent stock solution. The specially designed cylinder is constructed of Plexiglas, 6 in. in diameter and 25 in. high (Fig. 1).
4. After initial settling of the particles, adjust the water in the cylinder to the 20-in. mark by adding or draining water. Cap the graduated cylinder and mix by inverting from end to end 20 times within 30 sec. Suspend a thermometer at the 10-in. mark in the solution and allow the floc to settle. Keep the cylinder out of direct sunlight.
5. After 20 min read the water temperature to the nearest 0.1 of a degree centigrade and record the sediment height to the nearest 0.1 in.

6. Wash the entire contents of the cylinder over a No. 30 sieve and dry the portion retained on the sieve to constant weight.

7. Sieve the dry part over a No. 16 sieve and weigh the amount retained. Subtract this weight from the original weight and calculate the percentage loss.

8. The modified Los Angeles rattler number is expressed as a fraction showing the percent loss/sediment height (for example, 34/10.6).

The maximum allowable percent loss through the No. 16 sieve for the modified LAR test is the same as that prescribed for the standard LAR test. The study indicated that the maximum passing sediment height is about 13.0 in. The results of this test are given in Table 1.

Determination of Sediment-Height Correction Curves

To determine the effect of the mixture temperature on the sediment-height portion of the modified LAR test, the sediment heights vs. the recorded mixture temperatures for 21 samples were observed. Using 20 C as a
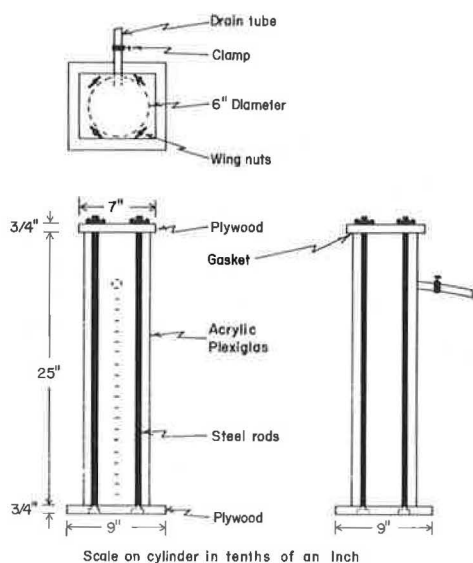

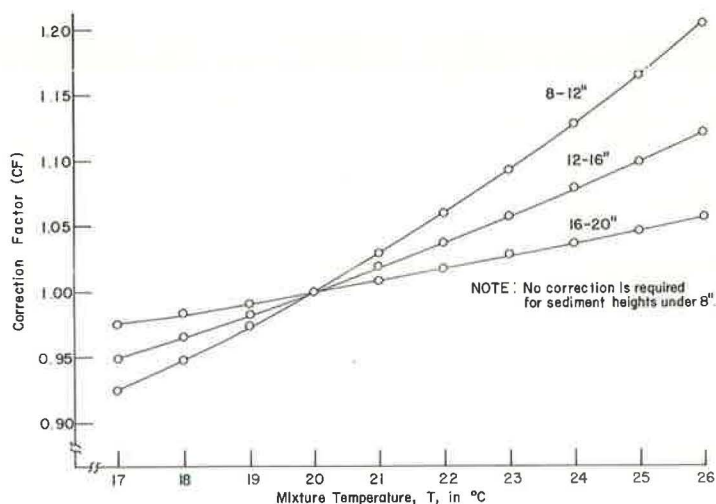Figure 1. Sediment-height cylinder for modified Los Angeles rattler test.

Figure 2. Sediment-height correction factor vs. mixture temperature for
three sediment height ranges.

standard temperature, a sediment-height correction factor, CF, was calculated for
each sample with temperatures ranging from 17 to 26 C. The average correction factor
was calculated for each degree centigrade for each of three sediment-height ranges.
These averages vs. the 10 corresponding mixture temperatures are shown in Figure 2.

To apply the sediment-height correction factor after having performed the modified
LAR test, use the following formula:

$$S_{20} = (CF)\ S_T$$

where

S_{20} = corrected sediment height at 20 C,
CF = sediment-height correction factor, and
S_T = observed sediment height at T degrees centigrade.

PETROGRAPHIC EXAMINATION OF AGGREGATES

Petrographic examination of aggregate is a visual analysis of the material in terms
of both lithology and properties of the individual particles. The procedure requires use
of a hand lens and petrographic and stereoscopic microscopes. By petrographic exam-
ination, the relative abundance of specific types of rocks and minerals is established;
the physical and chemical attributes of each, such as particle shape, surface texture,
pore characteristics, hardness, and potential alkali reactivity, are described; and the
presence of contaminating substances is determined.

Dr. Donald W. Hyndman at the University of Montana performed the petrographic
analysis on all original samples (14). The major minerals or rocks that composed
each of the 51 samples were determined. From the petrographic analysis, Hyndman
also predicted whether or not the aggregates were acceptable for road surfacing. Those
that failed his analysis are given in Table 2.

DISCUSSION OF TEST RESULTS

The primary purpose of this study was to determine if a single test—the modified
LAR test—can identify low-quality aggregates which it now takes three tests—the stan-

TABLE 2

COMPARISON OF DEGRADATION TESTS

Passed All Four Tests		Failed Standard Los Angeles Rattler Test	Failed Modified Los Angeles Rattler Test by		Failed Washington Degradation Test	Failed Petrographic Analysis
			Percent Loss	Sediment Height		
1801	2139	2123	2094	1805	1805	1805
1909	2144	2141	2123	1887[a]	1887[a]	1953
1922	2145	2142	2141	1953	1953	2076
1959	2181	2562	2142	2079	2079	2079
1960	2182		2562	2126	2096	2080
2022[b]	2183		2757	2148	2126	2094
2026	2222			2180	2148	2180
2074	2555			2187	2187	2187
2075	2556			2516	2561	2561
2122	2557			2561	2562	2756
2124	2563			2584	2584	2757
2125	2635			2760[a]	2760[a]	2761
2137	2758			2761	2761	
2138	2759					

[a]1887 and 2760—Petrographic analysis unavailable.
[b]2022—Standard Los Angeles Rattler Test and Washington Degradation Test unavailable; therefore, assumed passing.

dard LAR test, the Washington degradation test, and a petrographic analysis—to identify.

The correlation between the standard and the modified LAR percent losses was determined. Using the method of linear correlation (15), the correlation coefficient, r, was found to be 0.956 (Fig. 3), which is quite high and therefore significant. It was felt that the 1,000 ml of water used in the modified test caused enough extra degradation to offset the use of the No. 16 (1.19-mm) sieve instead of the No. 12 (1.68-mm) sieve that is prescribed for the standard test. The standard LAR test yielded 29 specimens higher and 14 specimens lower in percent losses than the modified LAR test. However, the difference between the averages for both tests was only 1.58 percent abrasion loss. This small amount is not considered important because the sample may vary this much from test to test or from sample to sample taken from the same source.

The modified LAR test was correlated with the Washington degradation test because both tests utilize a sedimentation analysis. The parabolic equation, $Y = -3.15 + 0.555X + 0.02X^2$, provided the best index of correlation, $i = 0.745$, as is shown by the solid line in Figure 4.

Sample 2180, the only lightweight aggregate tested (specific gravity = 1.76, percent absorption = 18.97), had a modified LAR test sediment height of 15.8 in. and a Washington degradation test sediment height of only 0.8 in. The reason for this extreme variance is not known; however, it is possible that the aggregates used were from different locations within the same source. For this particular sample, performance records supplied by the U. S. Forest Service indicated that this aggregate had provided 1 to 3 years of service. When this sample was not included in the computations, an index of correlation of 0.804 was found. The par-
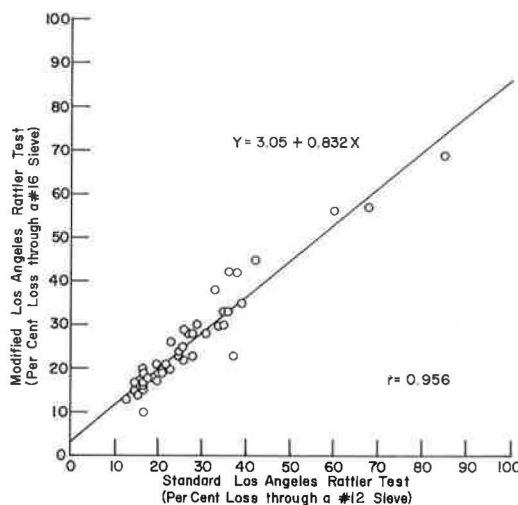

Figure 3. Modified Los Angeles rattler percent loss vs. standard Los Angeles rattler percent loss.

abolic equation for this case was $Y = -2.02 + 0.251X + 0.04X^2$, as shown by the dotted line in Figure 4.

Comparing the modified LAR test with the other tests conducted, only four specimens (2076, 2080, 2096, and 2756) would have passed the modified LAR test but failed one of the other three tests. Samples 2076, 2080, and 2756 failed only the petrographic analysis. Sample 2096 failed only the Washington degradation test. This failure may be explained by the fact that the Washington degradation test was performed on an alternate sample from the same source. The modified LAR test failed one sample, 2516, that none of the other tests termed as failing. This sample is in the "unsure" area because the modified

Figure 4. Washington degradation test sediment height vs. modified Los Angeles rattler test sediment height.

sediment height was 13.7 in. and Washington degradation factor was 48 (Table 1).

The modified Los Angeles rattler test compared with the petrographic analysis better than did the Washington degradation test. Six samples were deemed unsatisfactory by the petrographic analysis yet were approved by the Washington test compared to only three samples approved by the modified LAR test.

All of the aggregates that failed the standard Los Angeles rattler test also failed the modified Los Angeles rattler test by percent loss through the No. 12 and No. 16 sieves respectively. It is important to note that the modified test failed two additional samples. These failures may have occurred because of the water used in the modified test. The same two samples were also found to be unsuitable by the petrographic examination. Also, it is believed that if samples 1887 and 2760 had been available they would have been rejected by the petrographic analysis because both failed the modified LAR test and the Washington degradation test.

Check tests for specific gravity and absorption were made on 32 samples. No significant correlation was found when comparing these results to those of the various degradation tests conducted.

A failing percent loss in the standard and the modified LAR tests usually occurred in coarse-grained igneous or metamorphic rocks such as pegmatite, aplite, schist, granite, or gneiss. The primary minerals in these unsatisfactory aggregates were muscovite, quartz plagioclase, and orthoclase. These belong to the mica and feldspar groups of silicate minerals. The gneisses were usually "fast settling" during the sedimentation analysis.

Sample 2757, a micaceous siltstone, was the only sedimentary rock to fail by percent loss through the No. 16 sieve, whereas several samples failed the sediment-height analysis. This would seem to indicate that the degradation of sedimentary rocks normally cannot be determined without some sort of degradation in the presence of water.

The sedimentary rocks that were determined to be unsuitable were primarily siltstone, shale, sandstone, and limestone. A large amount of plastic fines was produced by these fine-grained rocks, which usually contained the minerals calcite, orthoclase, illite, and quartz. These rocks normally failed the sediment-height analysis in both the Washington degradation test and the modified LAR test.

It is of interest to note that sample 2148, a quartzite, produced extremely high sediment heights. The large quantity of plastic fines that caused these high sediment heights may have been due to a few parallel veins of shale at the sample source. The
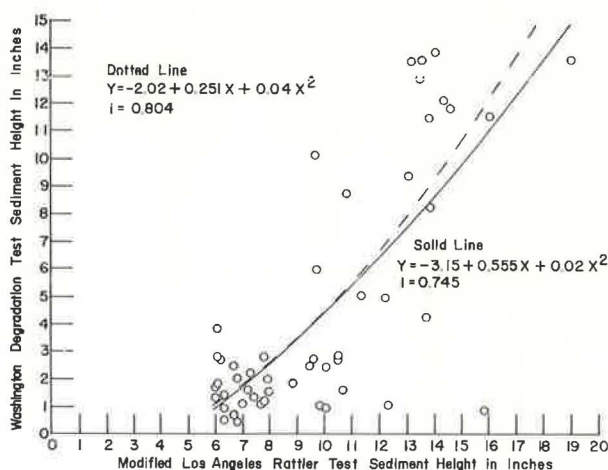
petrographic examination suggested that it was highly recommended as a road aggregate. According to available road-performance records, the aggregate has been used on a logging road for 5 years with excellent service.

To determine sediment-height correction curves, the water was changed in 21 samples. A temperature range of 17 to 26 C was chosen because the probability of a mixture temperature outside of this range was unlikely. All of the samples tested were within this range.

It was felt that applying the same correction factor for two different samples with far different sediment heights at the same temperature would not be correct. A possible solution was to make three curves, each with a different range in sediment heights. Because it was found that a change in mixture temperature had negligible effect on the sediment heights less than 8 in., no correction was needed on these fast-settling mixtures.

The corrected sediment height, S_{20}, was used to determine if an aggregate was desirable or undesirable. The failing corrected sediment height appeared to be about 13.0 in. It was found that the correction factor, CF, affected some of the observed sediment heights by more than 1 in.

To determine the reproducibility of the modified LAR test, 12 samples were tested twice. The four fast-settling samples averaged a variation in sediment height of only 0.1 in. with a range of 0 to 0.3 in. The remaining 8 samples varied by an average of 1.2 in. with a range of 0.2 to 3.0 in. The 12 samples tested varied by an average of 1.2 percent abrasion loss through the No. 16 sieve with a range of 0 to 4.0 percent. This indicates that the modified LAR test can be reasonably reproduced. Because 5,000 grams were used in the modified LAR test, compared to 1,000 grams used in the Washington degradation test, the results should be more representative of the quality of the aggregate. Better reproducibility is normally obtained using a larger, more representative sample.

CONCLUSIONS

From this study the following conclusions have been reached:

1. The modified Los Angeles rattler test is reliable for distinguishing between desirable and undesirable aggregates and, therefore, could possibly replace the standard Los Angeles rattler test, the Washington degradation test, and the petrographic examination.

2. The Washington degradation test does not appear to degrade lightweight aggregate as much as the modified Los Angeles rattler test.

3. If more samples with Washington degradation test sediment heights between 3 and 11 in. had been supplied, a higher index of correlation may have been obtained.

4. The modified Los Angeles rattler test compared with the petrographic analysis better than did the Washington degradation test.

5. The mixture temperature has pronounced effect on the sediment height in the modified Los Angeles rattler test.

6. The maximum allowable sediment height should depend on the use for which the aggregate is intended. However, aggregates producing sediment heights greater than 13.0 in. should be considered unsatisfactory for use on highway surfaces or base courses.

7. The modified Los Angeles rattler test results can be reproduced to within the tentative limits of ±1 percent abrasion loss and ± 1.25 in. in the sediment height.

8. The advantages of the modified Los Angeles rattler test are that (a) sample preparation is simple and quick; (b) the total time to perform the test is less than 1 man-hour; (c) the use of a 5,000-gram test sample increases the probability of obtaining more representative results; and (d) the use of a No. 16 sieve is more compatible with the standard fine aggregate sieve series.

ACKNOWLEDGMENTS

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ROTARY KILN-FIRED SYNTHETIC AGGREGATES MANUFACTURED FROM TEXAS LIGNITE FLY ASH

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The use of manufactured synthetic aggregates is increasing because of the depletion of natural aggregate supplies and the trend toward widespread use of lightweight construction materials. Waste materials as well as natural clays, shales, and slates are being used to manufacture these synthetic aggregates. This investigation is a study of the production of a synthetic aggregate from fly ash, a waste product of an electric power plant. The effects of several agglomerating and extrusion aids on an extruded kiln-fired fly ash aggregate are examined in this study. The physical properties of these aggregates were obtained and compared with ASTM Specification C 330 and with the recommended synthetic coarse aggregate classification system of the Texas Transportation Institute. It was concluded that five of the six kiln-fired aggregates are suitable for asphaltic concrete surfaces and bases, unexposed portland cement concrete, portland cement concrete pavements, base courses, and coverstone for seal coats and surface treatments.

•THE USE of manufactured lightweight aggregates in the construction of buildings and highways has become widespread in the last few years. The production of these lightweight aggregates has increased because of the engineering advantages of such lightweight materials as well as the depletion of locally available natural aggregates. Although most of these lightweight aggregates are produced from naturally occurring materials (clay, shale, and slate), industrial waste products such as fly ash are also being used.

Even though in 1969 only eight aggregate manufacturers in the United States were producing synthetic aggregate made from fly ash ($\underline{1}$), the utilization of fly ash in this manner is becoming increasingly accepted. Fly ash is not only used in lightweight aggregate but is also used as a cement replacement and as a soil stabilizer ($\underline{2}$). The ease of accessibility and low cost as well as its favorable properties make fly ash an excellent raw material for the production of synthetic aggregate. The fact that fly ash is an industrial waste material that presents an enormous disposal problem also enhances its appeal.

The synthetic aggregate manufacturing processes currently being used involve agglomeration of the raw materials and firing of these materials at a high temperature (2,000 F). The agglomeration phase can be accomplished by using pelletizing devices such as a revolving disk, a pelletizing drum, or a pugger-extruder. The extrusion process has several advantages over the other methods ($\underline{3}$) including (a) the elimination of the need for secondary crushing, (b) simplification of plant operation, (c) superior performance in concrete (improved workability and higher strengths), and (d) efficient mixing of additives. The firing process is accomplished either by a sintering (traveling grate) process or by a rotary kiln operation. The aggregates produced in this study were pelletized by extrusion and fired in a pilot rotary kiln.

MATERIALS

This investigation considered various combinations of fly ash, five agglomerating agents, and nine additives. The fly ash used was a lignite fly ash produced in the operation of a power plant near Rockdale, Texas. It was collected by a mechanical separator and is therefore relatively coarse. The chemical and physical properties of this Rockdale fly ash (4, 8) are given in Table 1.

Several materials were investigated to aid agglomerating and extrusion. Because the extrusion process requires a cohesive material with low interparticle friction, the pelletization of fly ash by extrusion methods often requires greater quantities of agglomerating and extrusion aids than would be necessary for pelletizing fly ash by other devices. Because of this need for cohesion and low internal friction, clays or shales are generally chosen as agglomerating agents. In this study, five such materials were studied: (a) a commercial kaolinite clay, (b) an Eagle Ford shale from near Midlothian, Texas, (c) a clay from near College Station, Texas, (d) a by-product resulting from processing bauxite referred to as "red mud", and (e) an organic clay from near Fairfield, Texas.

The study plan called for an investigation of the feasibility of using selected agglomeration agents and extrusion aids in combination with the Rockdale lignite fly ash. The additives and bloating agents used included crude oil, asphalt emulsions, soap, pine oil, commercial air-entraining agents, calcium chloride, sodium bicarbonate, vinsol resin, and corn starch. More than 50 combinations of these materials in different proportions were mixed and evaluated. The major basis of evaluation was the ease of extrusion of the fly ash mixture. The more favorable formulations chosen for further investigation included all five agglomerating agents with vinsol resin. All other additives were eliminated because of lack of cohesion or excessive internal friction developed in the extruder. After further investigation, the College Station clay and the Fairfield clay were excluded as feasible constituents because these materials did not produce pellets of sufficient strength to withstand handling operations associated with rotary kiln operations.

AGGREGATES

Production

The mixes chosen for further investigation were pelletized by means of a pugger-extruder, which extrudes the mixture through a steel die, producing pellets of various diameters ($\frac{1}{4}$, $\frac{3}{8}$, $\frac{1}{2}$, and $\frac{3}{4}$ in.). The lengths of the pellets are controlled by use of a wire cutter or other appropriate device. These pellets were then divided into three groups, each with a different drying time. One group was not allowed to dry, whereas the other groups were allowed 24 to 48 hours to dry. Each group of pellets was then

TABLE 1
ROCKDALE FLY ASH ANALYSIS

Properties[a]	Percent by Weight
Chemical	
Silicone dioxide (SiO_2)	33.9
Aluminum oxide (Al_2O_3)	18.1
Ferric oxide (Fe_2O_3)	6.6
$SiO_2 + Al_2O_3 + Fe_2O_3$	58.5
Calcium oxide (CaO)	21.9
Magnesium oxide (MgO)	1.8
Sulfur trioxide (SO_3)	2.2
Available alkali (NA_2O)	0.7
Physical	
Ignition loss	6.0
Retained on No. 325 sieve	37.0
Blaine fineness	2,115[b]
Specific gravity	2.57[c]

[a]Data taken from Manz (4, 8).
[b]In cm^2/gm.
[c]Not percentage by weight.

Figure 1. Pilot rotary kiln at Texas A&M University.

TABLE 2

PHYSICAL PROPERTIES OF THE AGGREGATES

Sample No.	Materials	Apparent Specific Gravity		Bulk Specific Gravity		Porosity (percent)	14-Day Absorption (percent)	100-Min Saturation (percent)	Recombined Unit Weight[a] (lb/cu ft)	Unit Weight as Produced (lb/cu ft)
		Average	Range of Values	Average	Range of Values					
RM5	Red mud, 5% Vinsol resin, 0.25%	2.67	2.66-2.67	2.19	2.17-2.21	18.1		7.43		48.2
K5-1	Kaolinite clay, 5% Vinsol resin, 1%	2.72	2.71-2.74	2.16	2.15-2.18	20.6	6.33	9.76	59.8	54.8
M5	Midlothian clay, 5% Vinsol resin, 0.25%	2.66	2.61-2.69	2.14	1.99-2.22	19.4	5.01	14.23	60.3	54.0
M10	Midlothian clay, 10% Vinsol resin, 0.25%	2.62	2.57-2.65	2.17	2.17-2.18	17.2	6.80	20.46	58.3	53.0
K5	Kaolinite clay, 5% Vinsol resin, 0.25%	2.78	2.70-2.80	2.19	2.18-2.20	21.2	7.75	18.51	57.4	53.8
K10	Kaolinite clay, 10% Vinsol resin, 0.25%	2.73	2.68-2.76	2.20	2.20-2.20	19.4	7.37	14.38	60.4	58.0

[a]ASTM Specification C 330 gradation (¾ in. to No. 4 sieve size).

fired in an electric muffle furnace at temperatures of 2,000, 2,050, 2,100, 2,150, 2,200, 2,250, and 2,300 F. The pellets were then compared and evaluated according to strength, external surface texture, and internal bleb structure. Relatively high-strength, well-developed bleb structure and a rough but nonporous surface texture were, in general, the qualities sought in the examination and evaluation of the pellets.

The mixes chosen for further study were extruded in the same manner as described in the preceding and fed into the pilot rotary kiln (Fig. 1) after being dried approximately 48 hours. The aggregate feed rate, kiln rotation speed, firing temperature, and slope of the kiln were set to ensure a certain retention time of the aggregate and a proper calcining of the aggregate.

Properties

The aggregates obtained from the kiln firings were subjected to selected laboratory tests and evaluations to determine their physical properties. These tests included sieve analysis, apparent and bulk specific gravity, porosity, 14-day absorption, 100-min saturation, unit weight, Los Angeles abrasion, and aggregate freeze-thaw (5). The results obtained from these tests are given in Tables 2, 3, and 4.

Evaluation

The kiln-fired aggregates were evaluated with respect to ASTM Tentative Specification C 330 and the Texas Transportation Institute's synthetic coarse aggregate classification system (6), which is given in Tables 5 and 6. This latter classification system is not a substitute for existing specifications but is intended as a supplement to them. Although the aggregates, as produced, do not meet the ASTM grading requirements for lightweight aggregates for structural concrete, they can be manipulated in either of two ways to conform to these specifications. One method is to alter the size distribution of the "green" pellets that are fed into the kiln. In this study, pellets were made in ½-, ⅜-, and ¼-in. diameter sizes, thus affording the operator three sizes to work with in altering the gradation. The other method is to crush the mate-

TABLE 3

AGGREGATE ABRASION AND FREEZE-THAW TEST RESULTS

Sample Number[a]	Los Angeles Abrasion (percent loss)	Aggregate Freeze-Thaw, 50 Cycles (percent loss)
RM5	34.7	5.77
K5-1	20.3	0.36
M5	26.8	0.03
M10	31.4	0.21
K5	39.7	0.27
K10	23.2	0.08

[a]Refer to Table 2 for description of materials of each sample.

TABLE 4

AGGREGATE GRADATIONS

Sieve Size	Cumulative Percent Retained by Weight[a]					
	RM5	K5-1	M5	M10	K5	K10
1 in.	16.8					
3/4 in.	72.8	0.63	4.65	0.98	0.0	1.45
5/8 in.		5.03	15.05	4.11	0.8	7.01
1/2 in.	88.0	31.03	54.15	44.77	33.1	35.75
3/8 in.	91.6	50.87	80.17	80.51	46.4	47.52
No. 4		99.71	98.64	99.38	97.7	97.92
No. 10	97.2	99.84	99.19	99.69	98.1	99.53
No. 16		99.85	99.43	99.78	98.4	99.70
No. 40		99.89	99.61	99.85	98.9	99.74
No. 80		99.93	99.81	99.90	99.2	99.79
No. 100		99.94	99.83	99.91	99.3	99.80
No. 200		99.97	99.95	99.96	99.7	99.88
Pan	100.0	100.00	100.00	100.00	100.0	100.00

[a]Refer to Table 2 for description of materials of each sample.

TABLE 5

TEXAS TRANSPORTATION INSTITUTE CLASSIFICATION SYSTEM FOR COARSE SYNTHETIC AGGREGATE

Class	Group	Dry Loose Unit Weight (lb/cu ft)		100-Min Saturation (percent) Max.	Aggregate Freeze-Thaw (percent loss) Max.	Pressure Slaking Value (percent) Max.	Los Angeles Abrasion (percent loss) Tex 410 A Max.
		Max.	Min.				
I (bloated)	A	55	35	15	7	6	35
	B	55	35	20	15	6	40
	C	55	35			10	45
II (nonbloated)	A		55		7	6	35
	B		55		15	6	40
	C		55			10	45

rial after kiln-firing to produce the required grading. There are advantages as well as disadvantages to either approach.

Shown in Figure 2 are the raw materials that were used to produce one of the acceptable fly ash aggregates together with samples of portland cement and asphalt concrete specimens made with these aggregates.

To be classified by ASTM as a coarse lightweight aggregate, the dry loose unit weight of the aggregate must not exceed 55 lb/cu ft. In this study, none of the aggregates met this specification and therefore are classified as nonbloated or Class II synthetic aggregates by the TTI classification system. None of these aggregates, however, fall into the Portland Cement Association's range (7) of natural, dense aggregates—75 to 100 lb/cu ft. This places the aggregates in this investigation in a range between normal and lightweight aggregates.

All six aggregates are well within the aggregate freeze-thaw specification for group A classification.

Because these aggregates are in Class II, there is no limit on the 100-min satu-

TABLE 6

FUNCTIONAL GROUPING OF SYNTHETIC COARSE AGGREGATES

Function	Permissible Aggregate Group	Additional Aggregate Requirements: Potential Reactivity (ASTM C 289)
Surface treatments[a]	IA	
Asphaltic concrete surfaces[a]	IA, IIA	
Asphaltic concrete bases[a]	IA, B, C	
	IIA, B, C	
Exposed lightweight portland cement concrete structures	IA	Innocuous
Pavements and unexposed portland cement concrete	IA, B	
	IIA, B	Innocuous
Base materials	IA, B, C	
	IIA, B, C	

[a]The 100-min saturation requirement is waived for these uses.

ration. However, one must realize that, because these aggregates have such a high saturation in the first 100 min, they may pose water-absorption problems when used to batch concrete.

The Los Angeles abrasion requirements for a group A aggregate are met by aggregates RM5, K5-1, M5, M10, and K10. Aggregate K5 falls in the B group.

From the preceding analysis, it can be seen that all the aggregates except K5 fall into the category of a Class IIA synthetic aggregate. Aggregate K5 is classified as a Class IIB aggregate. By referring to Table 6, one can see that aggregates RM5, K5-1, M5, M10, and K10 can be used in asphaltic concrete surfaces and bases, in unexposed portland cement concrete and

Figure 2. Fly ash and clay mixtures, fired pellets, and finished products using fly ash aggregate.

PCC pavements, or as base material. This classification system does not indicate that these materials can be used as coverstone for seal coats and surface treatments; however, the authors believe that these aggregates could be acceptable. Aggregate K5 is an exception and should not be used in either asphalt concrete surfaces or seal coats because of its excessive abrasion loss. This low abrasion resistance was probably caused by overheating during firing.

Visual inspection indicates that the fly ash aggregates would produce a high coefficient of friction when used as a coverstone for a seal coat or surface treatment or as an aggregate in hot-mix surfaces for highways and airfields.

CONCLUSIONS

The question of whether or not to use a lightweight synthetic aggregate in place of a natural dense aggregate is usually resolved by service demands and economic considerations. If a synthetic aggregate can effect a net savings or can be justified by providing improved physical properties, it will be used even though its unit cost may exceed that of natural aggregate. When all other variables are constant, the lighter the aggregate the greater the yield and, possibly, the lower the unit cost. There are situations, however, where weight is not an important consideration, as in highway pavements. In such cases, the lightweight or bloated synthetic aggregate may offer very little advantage over the nonbloated synthetic aggregate. In areas where natural aggregates are in short supply or have been depleted, synthetic aggregates (nonbloated as well as bloated) make attractive substitutes.

In areas where natural aggregates are scarce, therefore, kiln-fired fly ash aggregates make excellent aggregates for highway construction applications. As shown in this report, aggregates RM5, K5-1, M5, M10, and K10 are quite acceptable for use in asphaltic concrete surfaces, asphaltic concrete bases, portland cement concrete pavements, base materials, and seal coats and surface treatments. Aggregate K5 is not acceptable as an asphaltic concrete surface aggregate or a coverstone for a seal coat because of its high abrasion loss, but it is quite satisfactory in the other uses given.

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A GENERALIZED INVESTIGATION OF THE POTENTIAL AVAILABILITY OF AGGREGATE BY REGIONAL GEOMORPHIC UNITS WITHIN THE CONTERMINOUS 48 STATES

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The principal activity in this study was an examination of the potential availability of aggregate within the conterminous 48 states using an extant physiographic classification system. The classification selected was a modified version of the Woods-Lovell Physiographic-Engineering System, comprised of 97 physiographic sections. The estimated potential availability of aggregate was rated at four levels, ranging from abundant-to-adequate to severe-problem. Individual physiographic section ratings were assigned following an examination of the major aggregate type(s) used, their distribution within the section, and any quality limitations associated with their use. Information relative to aggregate types and characteristics was gathered from the literature, particularly bedrock and surficial maps, and from responses to a materials questionnaire sent to each state highway agency. The study shows that aggregates are potentially in short supply in about one-third of the conterminous 48 states, i. e., these areas have limited-to-severe-problem ratings. Such areas are primarily comprised of regional topography formed from predominantly sandstone and shale bedrock or from transported deposits deficient in either quantity or quality of sand-gravel and underlain by bedrock having little or no aggregate potential.

•ALTHOUGH engineering design and construction decisions are unique solutions to specific problems, such solutions depend strongly on the engineer's store of highly relevant experiences from which he can draw information, ideas, methodologies, etc. How does the engineer organize his experience so that the item appropriate to a particular job is retrieved? For example, are there geographic units within which general geotechnical conditions are unique and, accordingly, engineering solutions are similar?

The search for geographic units within which ground conditions, other environmental factors, engineering problems, and presumably design and construction practice demonstrate significant homogeneity has led to the study of the works of physiographers and regional geomorphologists. By using their three major classification factors of structure, process, and stage, which can be practically interpreted as parent material, origin, and age (9), physiographic units are mapped. Each unit has a unique mode of topographic expression, i.e., it differs from its neighbors in this respect. The units of greatest usefulness to engineers are, from largest to smallest, province, section, and subsection.

The principal objective of this paper is to report a study of the practicability of using physiographic units to rate the potential availability of quality aggregate resources within the conterminous 48 states. The units were defined by a slightly modified version of the Woods-Lovell Engineering-Physiographic System presented in 1960 (8). This version resulted in the establishment of 97 unique areas, termed physiographic sections, to be investigated. Table 1 gives each province and section and provides a code for their location in Figure 1.

<div align="center">METHOD OF ANALYSIS</div>

Data on the distribution of potential aggregate resources were sought in the literature and by questionnaires directed to the state highway agencies. Sources were mapped by aggregate type, and all quality problems reported for the sources were recorded.

Mapping

Aggregate sources were mapped as (a) crushed-stone sources (Fig. 2) and (b) sand-gravel sources (Fig. 4). The crushed-stone sources were subdivided into carbonate rocks, granitic and metamorphic complexes, and other igneous (primarily basaltic type) rocks. Certain references (1, 2, 4, 5, 6) were particularly valuable in providing state or regional maps of potential sources. Where such direct information was not located, the authors attempted to derive it from generalized state or regional geologic maps, origin-parent material diagrams, and aggregate production data.

In addition to regional potential aggregate areas, existing pits and quarries were mapped on a U.S. scale. These locations were grouped as sand and sand-gravel pits; crushed-stone quarries, regardless of major geologic rock type; and a miscellaneous category. Included within the latter group were slag, lightweight aggregate, caliche, clam and oyster shells, and coquina.
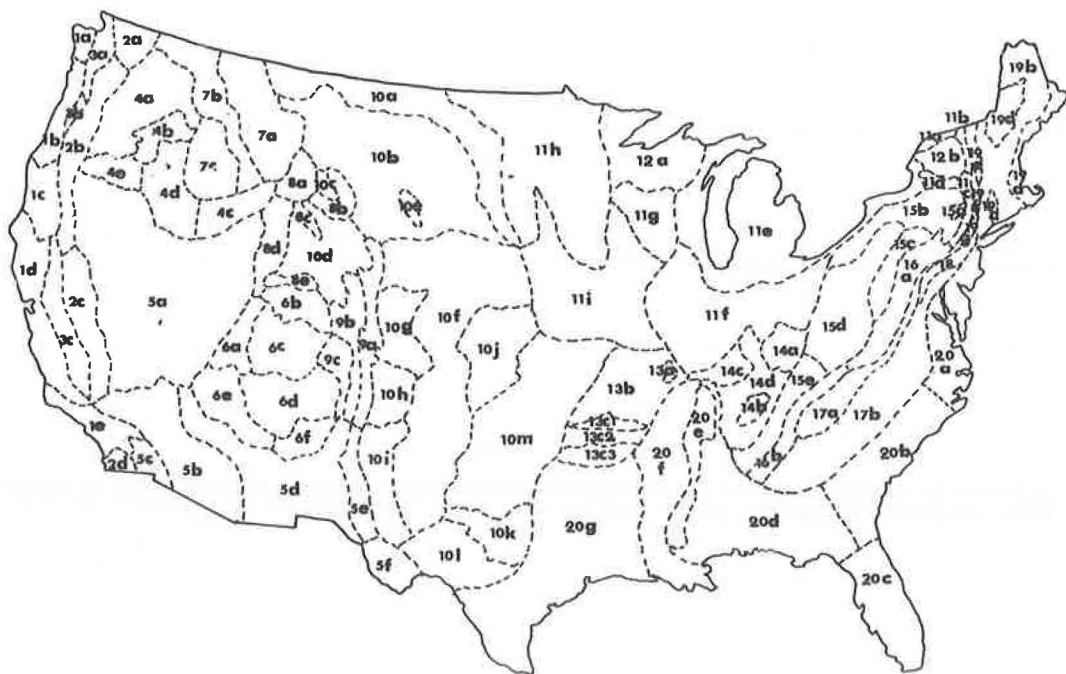

Figure 1. Physiographic diagram of the United States.

TABLE 1

PHYSIOGRAPHIC UNIT CODE

1. Western Mountains of the Pacific Coast Range
   a. Olympic Mountain
   b. Oregon Coast Range
   c. Klamath Mountain
   d. California Coast Range
   e. Los Angeles Range

2. Sierra-Cascade
   a. Northern Cascade Mountain
   b. Southern Cascade Mountain
   c. Sierra Nevada
   d. Lower California

3. Pacific Troughs
   a. Puget Sound
   b. Willamette Valley
   c. California Valley

4. Columbia Plateau
   a. Walla-Walla
   b. Blue Mountain
   c. Snake River Plains
   d. Payette
   e. Harney

5. Basin and Range
   a. Great (Closed) Basin
   b. Sonoran Desert
   c. Salton Trough
   d. Open Basin (Mexican Highland)
   e. Sacramento Highland
   f. Great Bend Highland

6. Colorado Plateau
   a. High Plateaus of Utah
   b. Uinta Basin
   c. Canyon Lands
   d. Navajo
   e. Grand Canyon
   f. Datil

7. Northern Rocky Mountain
   a. Montana
   b. Bitteroot
   c. Salmon River

8. Middle Rocky Mountain
   a. Yellowstone
   b. Bighorn Mountain
   c. Wind River Mountain
   d. Wasatch
   e. Uinta Mountain

9. Southern Rocky Mountain
   a. Front Range
   b. Western
   c. San Juan Mountain

10. Great Plains
    a. Glaciated Missouri Plateau
    b. Unglaciated Missouri Plateau
    c. Bighorn Basin
    d. Wyoming Basin
    e. Black Hills
    f. High Plains
    g. Colorado Piedmont
    h. Raton Upland
    i. Pecos Valley
    j. Plains Border
    k. Central Texas Mineral
    l. Edwards Plateau
    m. Osage Plains

11. Central and Eastern Lowlands
    a. St. Lawrence Lowland
    b. Champlain Lowland
    c. Hudson River Valley
    d. Mohawk River Valley
    e. Eastern Lakes and Lacustrine
    f. Central Till Plain
    g. Driftless
    h. Western Lakes and Lacustrine
    i. Dissected Loessial and Till Plain

12. Laurentian Upland
    a. Superior Upland
    b. Adirondack

13. Ozark and Ouachita
    a. St. Francois Mountain
    b. Springfield-Salem Plateau
    c1. Boston Mountain
    c2. Arkansas Valley
    c3. Ouachita Mountain

14. Interior Low Plateaus
    a. Blue Grass
    b. Nashville Basin
    c. Shawnee Hills
    d. Highland Rim

15. Appalachian Plateau
    a. Catskill Mountain
    b. New York Glaciated
    c. Allegheny Mountain
    d. Kanawha
    e. Cumberland

16. Newer Appalachian (Ridge and Valley)
    a. Pennsylvania-Maryland-Virginia
    b. Tennessee

17. Older Appalachian
    a. Blue Ridge
    b. Piedmont

18. Triassic Lowland

19. New England Maritime
    a. Seaboard Lowland
    b. New England Upland
    c. Connecticut Lowland
    d. White Mountain
    e. Green Mountain
    f. Taconic
    g. Reading Prong

20. Atlantic and Gulf Coastal Plain
    a. Embayed
    b. Sea Island
    c. Florida
    d. East Gulf
    e. Mississippi Loessial Upland
    f. Mississippi Alluvial Plain
    g. West Gulf

Note: Numbers represent physiographic provinces, letters represent physiographic sections.

Aggregate Problems

Many aggregate problems are identified and described in the literature (particularly 3). The distribution and magnitude of the problems were primarily defined by the responses to the materials questionnaire sent to each state highway department.

The materials questionnaire responses provided a current summary of aggregate types, uses, quality (problems), and availability for each state. The questionnaire responses were coded for convenience. Information was obtained for the specific physiographic sections within each state. Of particular importance to this report were the responses to a request to identify those physiographic sections where suitable aggregates were generally lacking.

RESULTS

Aggregate Distribution

Figures 2 through 5 partially summarize the results of the mapping effort for both the potential aggregate areas and the pit and quarry locations. Figure 2 shows the generalized potential crushed carbonate stone areas within the 48 states. [Diagrams showing the distribution of potential crushed granitic and metamorphic complexes, igneous (basaltic) rock, and the miscellaneous aggregate category are shown elsewhere (9).] It is the opinion of the authors that the mapping of the carbonate zones east of the Rocky Mountain-Great Plains border gives a relatively accurate pattern of the distribution. West of this border the carbonate sources are often quite local and scattered, which makes them very difficult to map at this scale. Figure 3 shows the location of crushed-stone quarries.

Figures 4 and 5 show the distribution pattern of potential and extant sand and sand-gravel sources. The relative accuracy of the sand-gravel map is quite variable due to the wide range of references interpreted to depict the total distribution.
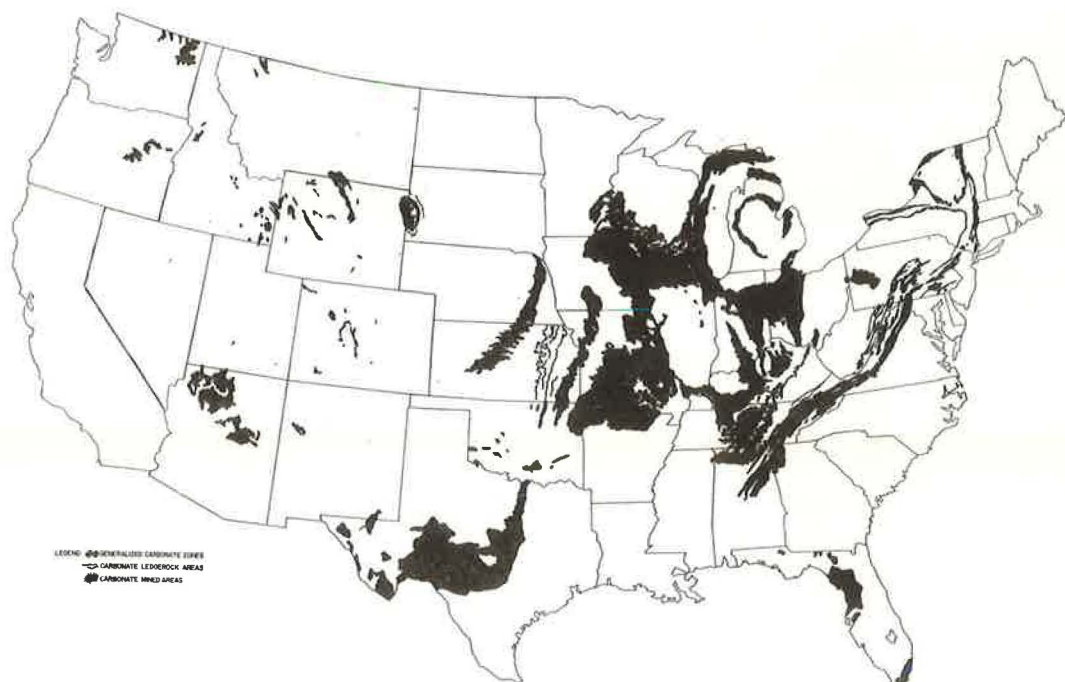

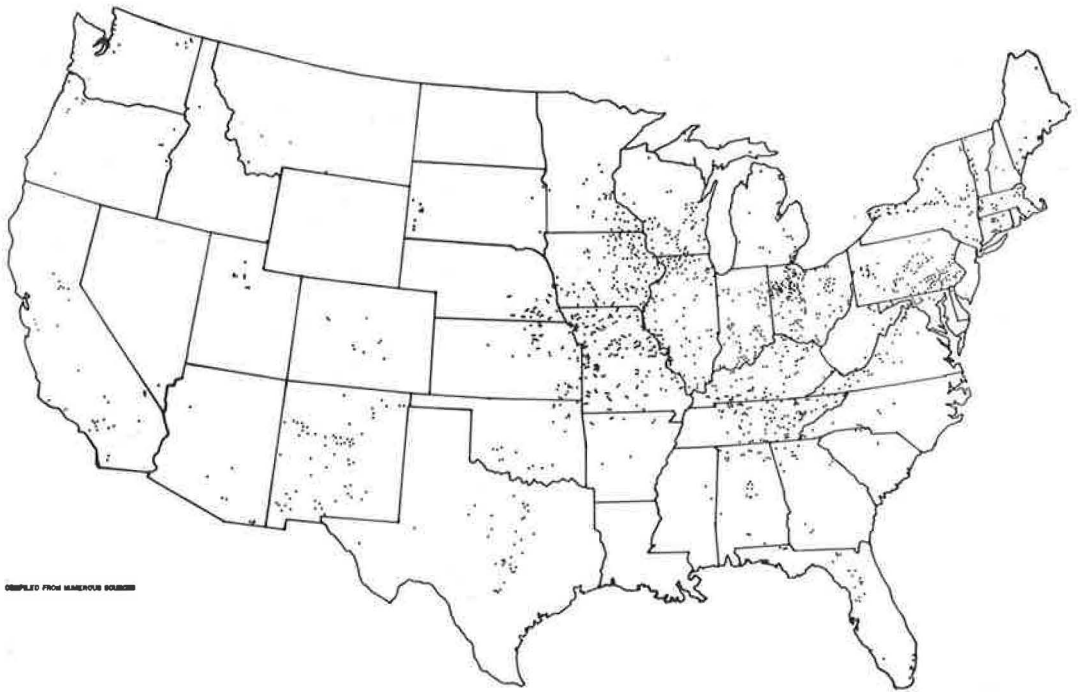Figure 2. Distribution of generalized potential crushed carbonate stone areas.

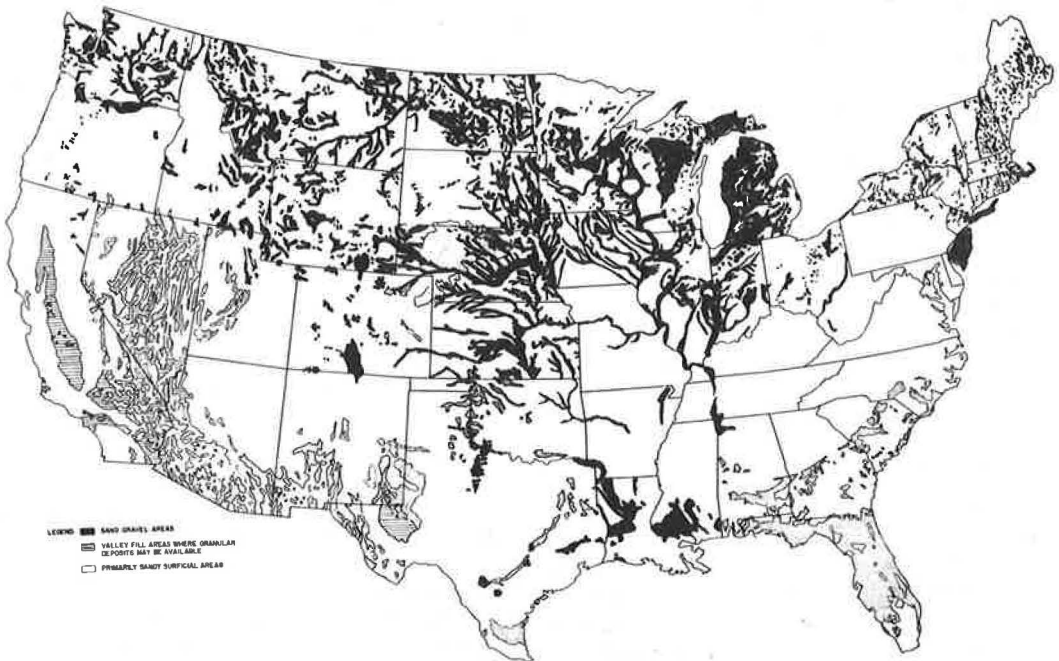Figure 3. Distribution of crushed-stone quarries in the United States.

Figure 4. Distribution of generalized potential sand-gravel areas.

Figure 5. Distribution of sand and sand-gravel pits in the United States.

Aggregate Type Use

Aggregate Type Code—A wide variety of aggregates is used for highways; e.g., a total of 34 types are recognized in this study. Table 2 keys these aggregate types to a numerical code. The major groupings are sands and gravels, crushed stone, and miscellaneous. The last group consists primarily of artificial aggregates as well as the authors' interpretation of several "localized" sources, such as coquina and clam and oyster shells. The crushed-stone group is subdivided into sedimentary, igneous (extrusive and intrusive), and metamorphic. The relationship between the coded aggregate number and the general category of aggregate should be apparent (sedimentary crushed-stone types are the 10 series, igneous crushed stones are the 20 series, etc.).

Use—A summary of the major aggregate use by state and aggregate type is given in Table 3. This summary is based primarily on the responses to the materials questionnaire. Where states did not provide such information, input was obtained from available literature (3, 5, 6, 7) and is accordingly more tentative.

Table 4 is a summary of the aggregate types used for each physiographic section. Figure 6 shows areas reported by representative state highway officials to be deficient in quality aggregate sources.

Availability Ratings of Aggregate by Physiographic Section

The estimate of the potential availability of aggregate for each physiographic section was based on a four-level rating scale: (a) abundant-to-adequate, (b) adequate-to limited, (c) limited-to-problem, and (d) severe problem.

Ratings were assigned to the physiographic units by determining the major type or types of aggregates used, examining their relative distribution within the unit, and assessing their major quality problems. It was thus possible for an area possessing an abundant distribution of a potential aggregate type to receive a "compromised" rating due to the presence of a major problem with that particular type. On the other hand,

TABLE 2

AGGREGATE TYPE CODE

Aggregate Type	Key		Aggregate Type	Key
I. Sands and gravels			C. Metamorphic	
Sand	1		1. Nonfoliated	
Gravel	2		Marble	30
Sand-gravel	3		Quartzite	31
II. Crushed stone			Serpentine	32
A. Sedimentary			2. Foliated	
Limestone	10		Gneiss	33
Dolomite	11		Schist	34
Chert	12		Amphibolite	35
Sandstone	13		III. Miscellaneous	
Novaculite	14		Limerock	40
Argillite	15		Coquina	41
B. Igneous			Clam and oyster shell	42
1. Extrusive			Scoria	43
Basalt	21		Volcanic cinders	44
Diabase	22		Pumice	45
Rhyolite	23		Expanded clay	46
Andesite	24		Slag	47
Greenstone	25		Silicified chalk	48
2. Intrusive			Caliche	49
Diorite	26			
Gabbro	27			
Granite	28			
Syenite	29			

TABLE 3

SUMMARY OF GENERAL AGGREGATE TYPES USED BY STATE

State	Aggregate Type[a]	State	Aggregate Type[a]
Alabama	2,3,10,11,13,28,30, 33,42,47	Nebraska	1,2,3,10
		Nevada*	3,10,13,21,28,30,31
Arizona	3,10,13,21,22,23, 28,31	New Hampshire	3,28,31
		New Jersey	1,3,10,11,15,21,22,
Arkansas	3,10,11,13,14,29,31		28,31,33,47
California*	3,10,13,21,28,47	New Mexico	3,10,21,23,24,26,
Colorado	3,10,13,21,28,30, 45,47		31,49
		New York	3,10,11,13,22,27,
Connecticut	3,21		28,30,31,47
Delaware	1,2,3,33	North Carolina	2,3,10,26,28,30,33
Florida	3,10,41,42	North Dakota	3,43
Georgia	3,10,12,28,30,31,33	Ohio	1,2,10,13,47
Idaho	1,2,3,10,11,21,31,47	Oklahoma	3,10,11,13,28
Illinois	3,10,11,47	Oregon	1,2,3,21
Indiana	3,10,11,47	Pennsylvania	3,10,13,22,28,47
Iowa	3,10,11	Rhode Island*	3,10,28
Kansas	3,10,11,12,13,48	South Carolina*	3,10,28
Kentucky	3,10,47	South Dakota	3,10,13,28,31
Louisiana	3,42,46	Tennessee	3,10,11,13,47
Maine	3,10,13,28,31	Texas*	3,10,13,21,28,30,
Maryland	1,2,10,27,28,30,32, 33,35,37		42,47,49
		Utah*	3,10,13,28,30
Massachusetts	3,10,21,22,23,26,33	Vermont	3,10,11,25,28,31,
Michigan	1,2,3,10,11,13,21, 23,47		32,35
		Virginia	3,10,11,13,21,22,
Minnesota*	3,11,21,28,31		28,30,33,42
Mississippi	3,42	Washington	3,10,13,21,28,31
Missouri	1,2,10,11,13,28	West Virginia	1,3,10,13,47
Montana	3,10,13,15,21,28,31, 43,47	Wisconsin	1,3,11,13,21,28,31
		Wyoming	3,10,11,13,21,28,43

*For the states marked by an asterisk (*), information has been obtained through a literature search and is to be regarded as possible aggregate types only. For the states not marked with an asterisk, information has been obtained from the materials questionnaire.

[a]See Table 2 for key to aggregate type.

TABLE 4

SUMMARY OF GENERAL AGGREGATE TYPES USED BY PHYSIOGRAPHIC SECTION

Section Code	Aggregate Type	Section Code	Aggregate Type	Section Code	Aggregate Type
1a	3, 13, 21	8a	3, 28	13a	1, 2, 10, 11, 28
b	1, 3, 13, 21	b	3, 10	b	1, 2, 3, 10, 11, 13
c	3	c	3, 10, 34	c1	3, 10, 13
d	3, 10, 11, 13	d	3, 10, 11, 13, 34, 47	c2	3, 10, 13, 29, 31
e	3, 10, 11, 28			c3	3, 10, 13, 14
2a	3, 13, 21, 28	e	3	14a	1, 2, 3, 10, 11
b	3, 10, 21	9a	3, 10, 28	b	3, 10, 47
c	3, 10, 28	b	3, 10, 28, 30	c	3, 10
d	3, 28	c	3, 21, 31	d	3, 10, 11, 13
3a	3, 13, 21	10a	3	15a	3, 13
b	3, 21	b	1, 3, 10, 13, 43	b	1, 2, 3, 10, 13, 47
c	3	c	3, 10, 11, 13, 21	c	10
4a	1, 2, 3, 21	d	3, 10, 13, 28	d	1, 2, 3, 10, 13, 47
b	1, 2, 3, 21	e	10, 28	e	1, 2, 3, 10, 11, 12, 13, 47
c	1, 2, 3, 21, 47	f	1, 2, 3, 10, 11, 13, 28, 49		
d	1, 2, 3, 21			16a	1, 2, 3, 10, 11, 13, 22, 47
e	2, 21	g	3, 10, 21, 47		
5a	1, 2, 3, 10, 11, 21, 31	h	3, 10, 21, 26	b	1, 2, 3, 10, 11, 12, 13, 30, 47
		i	3, 10, 49		
b	3, 21, 23, 28	j	1, 3, 11, 13, 48	17a	2, 3, 10, 21, 22, 28, 30, 33
c	3	k	3, 10		
d	3, 10, 13, 21-24, 28, 31, 44	l	3, 10	b	1, 2, 3, 11, 21, 22, 26-28, 30-35
		m	1, 2, 3, 10, 11, 12, 13, 28		
e	3, 10, 31, 49			18	1, 3, 10, 21, 22, 28
f	3, 10, 21	11a	3, 10, 11	19a	3, 10, 13, 23, 26, 28, 31, 33
6a	3	b	3, 10, 11, 31		
b	3	c	1, 3, 10, 11, 13, 31, 33	b	3, 10, 11, 21, 22, 25, 26, 28, 31-33, 35
c	3, 13				
d	3, 13, 21	d	3, 10, 11	c	3, 21, 22
e	3, 21	e	1, 2, 3, 10, 11, 13, 47	d	3, 28
f	3, 10, 21, 44			e	3, 25, 28, 32, 35
7a	3, 10, 13, 15, 21, 28, 31, 34, 47	f	1, 2, 3, 10, 11, 47	f	3, 10, 11, 31
		g	1, 3, 10, 11, 13, 28, 31	g	1, 3, 10, 11, 15, 22, 28, 30, 31, 33, 47
b	1, 2, 3, 10, 15, 21, 28, 31, 34	h	3, 10, 11, 28, 31		
		i	1, 2, 3, 10, 11, 28, 31, 47	20a	1, 2, 3, 28, 42, 47
c	2, 3, 21	12a	1, 2, 3, 21, 23, 30, 31	b	1, 2, 3, 42
		b	3, 27, 28	c	1, 10, 41, 42, 47
				d	1, 2, 3, 10, 11, 40, 42
				e	3
				f	1, 2, 3, 42, 46
				g	3, 42

LEGEND:
- ■ AREAS LACKING QUALITY AGGREGATES
- ▨ INDEFINITE NORTHERN BORDER
- ☐ STATES NOT REPLYING TO AVAILABILITY QUESTION OF QUESTIONNAIRE

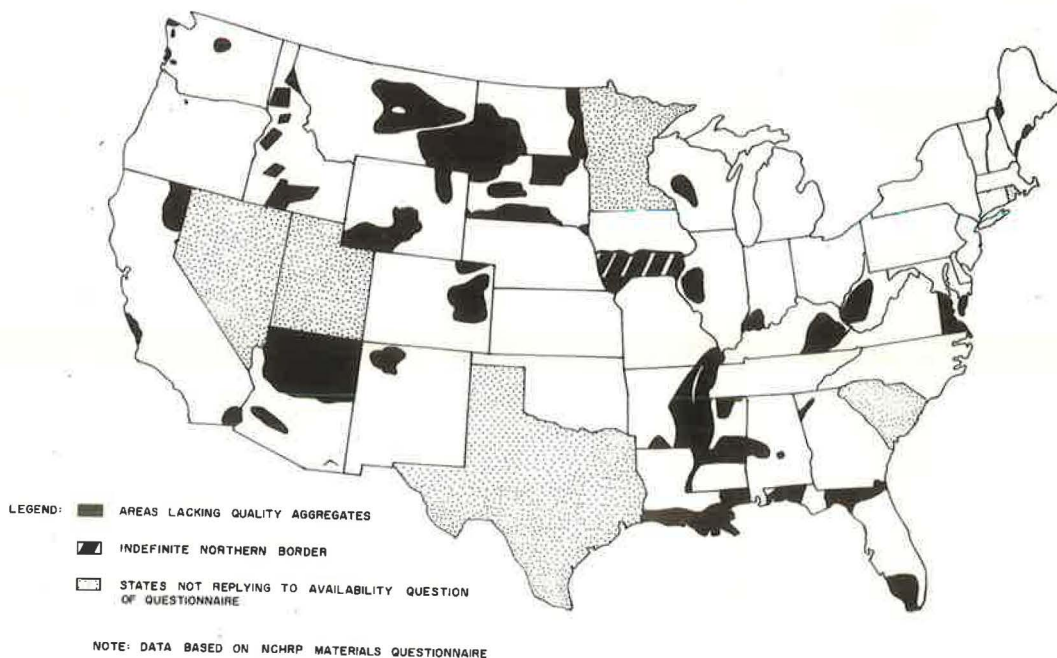NOTE: DATA BASED ON NCHRP MATERIALS QUESTIONNAIRE

Figure 6. Questionnaire summary of areas lacking quality aggregates.

TABLE 5

SUMMARY OF ESTIMATED POTENTIAL AVAILABILITY RATINGS OF QUALITY AGGREGATES WITHIN SECTIONS

Abundant-to-Adequate		Adequate-to-Limited		Limited-to-Problem		Severe Problem	
Section Code	Area (square miles)	Section Code	Area (square miles)	Section Code	Area (square miles)	Section Code	Area (square miles)
1e	21,790	1a,c	27,130	1b,d	53,840	—	—
2b,c,d	74,530	2a	16,140	—	—	—	—
3a,b,c	42,310	—	—	—	—	—	—
4a,b	61,100	4d	20,560	4c,e	33,190	—	—
5a,b,d,e,f	352,170	—	—	5c	10,520	—	—
6f	9,290	6a,e	41,520	6b,c,d	73,110	—	—
7a	43,800	7b	34,480	7c	27,500	—	—
8b,c,e	16,950	8a,d	28,390	—	—	—	—
9a,b,c	60,450	—	—	—	—	—	—
10c,e,k,l	68,560	10h,m	124,690	10a,d,f,g,i,j	336,200	10b	123,390
11a,b,c,d	11,000	11e,f,i	262,850	11g,h	119,060	—	—
12a,b	72,540	—	—	—	—	—	—
13a	3,790	13b,c3	48,610	13c1,c2	14,090	—	—
14a,b,d	34,840	—	—	14c	16,540	—	—
—	—	15a,b,c,e	56,830	15d	45,930	—	—
16a,b	45,340	—	—	—	—	—	—
17a,b	90,670	—	—	—	—	—	—
18	6,040	—	—	—	—	—	—
19b,c,e,f,g	46,440	19a,d	23,080	—	—	—	—
—	—	20a,b,d,g	319,900	20c,e	57,540	20f	45,700
Totals	1,061,610		1,004,180		787,520		169,090
Percentage of conterminous 48 states	35.1		33.2		26.1		5.6

LEGEND

ABUNDANT TO ADEQUATE
ADEQUATE TO LIMITED
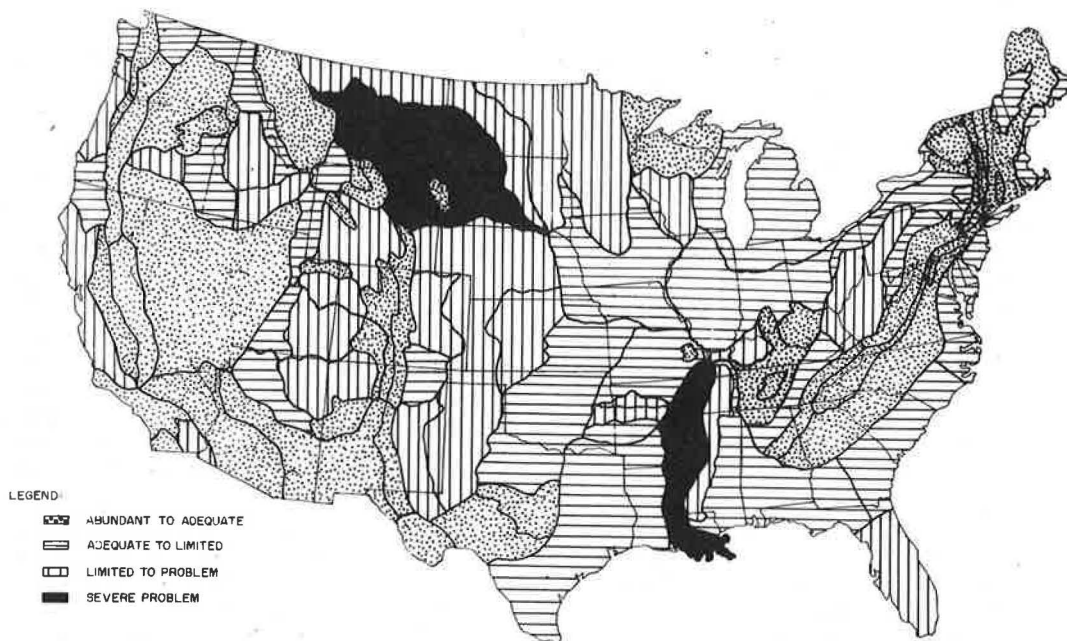LIMITED TO PROBLEM
SEVERE PROBLEM

Figure 7. Estimated potential availability rating of quality aggregate resources by physiographic unit.

a more limited supply of a good-quality aggregate type, which was well distributed within a physiographic section, could produce a high (i.e., abundant-to-adequate) aggregate availability rating.

Table 5 summarizes the estimated potential availability ratings of aggregates by section, and Figure 7 shows these regions within the conterminous 48 states.

SUMMARY AND CONCLUSIONS

Aggregate Availability

This study shows that quality natural aggregate resources are not available over sizable areas of the United States.

As noted in Table 5, section areas comprising more than 26 percent of the total estimated aggregate availability have a poor potential for aggregate resources, while sections totaling almost 6 percent have a very restricted potential aggregate supply. In addition, the "true" or "realistic" (in contrast to potential) appraisal of aggregate resources may show an even more unfavorable situation.

For example, in many Western areas good potential aggregate sources exist (particularly crushed stone), but the regions are sparsely populated and in many cases the sources are practically inaccessible due to the extremely rugged, mountainous conditions. These aggregate resources are not only less needed, they are, by virtue of their location, less available.

In contrast, in the East, many of the areas possessing potentially adequate supplies may be quite restricted for exploitation due to urban development and the associated high population densities. This would likewise tend to reduce the real availability.

Geologic Summary of Sections Having Poor Aggregate
Availability Ratings

Although each section possessing a limited-to-problem or severe-problem rating had its own unique combination of characteristics responsible for that rating, these sections can effectively be grouped into three types based on the causative geologic conditions. Table 6 gives the types and the grouping of aggregate-deficient sections within them.

One of the most significant factors, correlative with a lack of aggregate, was a widespread distribution of sandstones and shales. This, coupled with a relative lack of quality natural sand-gravel deposits, generally yields areas of extremely poor aggregate potential. From Table 6 it can also be seen that most of these areas (within the Type I grouping) have geologic ages that correspond to the Cenozoic, Mesozoic, Late Paleozoic (Permian and Pennsylvanian), and Early Paleozoic (Cambrian) eras. It is within the Paleozoic periods older than the Pennsylvanian that the majority of the crushed carbonate sources occur.

A modal bedrock pattern of sandstone and shale is not always associated with an overall poor aggregate availability rating for the section in question. Examples of sections that, in general, are characterized by relatively soft sandstone and shale bedrock but possess abundant-to-adequate potential for aggregates are the Bighorn Basin (10c), the Triassic Lowland Province (18), and the Connecticut Lowland Section (19c). Within these units, the widespread distribution of quality sand-gravels and/or crushed stone (obtained from variant bedrock types within the unit) is responsible for the rating.

Type II deficiency areas are those of poor quality igneous rocks (mostly extrusives), whereas in the Type III the bedrock is generally deeply buried by unconsolidated material deficient in the coarse sizes.

Although the examination of potential availability of aggregate in 97 physiographic sections produces a useful general perspective, the physiographic section is commonly too large and varied a region for the purpose. Subdivision of the conterminous 48 states into 242 subsections has been accomplished (9). Examination of this, or another engineering consideration, for these more homogeneous units would permit more specific regional predictions.

TABLE 6

GENERALIZED SUMMARY OF PREDOMINANT GEOLOGIC CONDITIONS EXISTENT WITHIN SECTIONS
POSSESSING AN AGGREGATE AVAILABILITY RATING LOWER THAN ADEQUATE-TO-LIMITED

Section (Code)	Remarks
Type I: Sections possessing widespread distribution of predominantly sedimentary sandstone and shale bedrock that significantly contributes to a poor aggregate availability rating	
Oregon Coast Range (1b)	Tertiary sandstones and shales.
California Coast Range (1d)	Tertiary and Mesozoic sandstones, shales, and some slates.
Uinta Basin (6b)	Tertiary sandstones and shales.
Canyon Lands (6c)	Mesozoic sandstones and shales.
Navajo (6d)	Mesozoic, Tertiary, and late Paleozoic sandstones and shales.
Unglaciated Missouri Plateau (10b)	Tertiary and Cretaceous sandstones and shales.
Wyoming Basin (10d)	Tertiary sandstones and shales.
Colorado Piedmont (10g)	Tertiary and Cretaceous sandstones and shales.
Pecos Valley (10i)	Triassic sandstones and shales; Permian sandstones, shales, limestone, and gypsum.
Plains Border (10j)	Cretaceous and Permian sandstones and shales; Cretaceous limestone.
Driftless (11g)	Cambrian sandstone and shales; Ordovician carbonates in Southwest may be used as crushed stone.
Boston Mountain (13c1)	Pennsylvanian sandstones and shales.
Arkansas Valley (13c2)	Pennsylvanian sandstones and shales.
Shawnee Hills (14c)	Pennsylvanian sandstones and shales; Mississippian limestones quarried.
Kanawha (15d)	Pennsylvanian sandstones and shales; Permian limestones and shales.
Type II: Sections possessing widespread distribution of bedrock with poor crushed-stone capability (other than that noted in Type I) that significantly contributes to a poor aggregate availability rating	
Snake River Plain (4c)	Cenozoic acidic lava plain; regional sand-gravel sources generally available only near mountain borders.
Harney (4e)	Cenozoic acidic lava plain with widespread pumice deposits and lacking regionally distributed sand-gravels.
Salmon River (7c)	Jurassic granitic rocks not suitable for use as highway aggregate.
Type III: Sections generally possessing a nonexistent-to-poor bedrock crushed-stone potential overlain by transported deposits either deficient in quantity or quality of granular materials	
Salton Trough (5c)	Widespread presence of fine-grained alluvial and lacustrine deposits characterize much of section.
Glaciated Missouri Plateau (10a)	Glaciated region possessing sand-gravel deposits of general poor quality underlain by bedrock similar to that found within unglaciated section (see 10b in Type I grouping).
High Plains (10f)	Crushed-stone potential nonexistent in section; major source of aggregates is from major rivers and tributaries. Higher concentration of rivers in northern portion of unit; however, much of the aggregate lacks coarse fraction and may be reactive with cement.
Western Lakes and Lacustrine (11h)	Western portion of unit possesses Cretaceous sandstones and shales similar in characteristics to those found in Type I grouping. Major areas void of aggregate associated with glacial lacustrine areas (Lakes Agassiz and Dakota).
Florida (20c)	Sandy unconsolidated coastal deposits veneer almost entire section. Regional carbonate zones present in portions of unit, but much of section lacks coarse aggregate.
Mississippi Loessial Upland (20e)	Widespread loessial deposits overlie areas lacking in crushed-stone potentials.
Mississippi Alluvial Plain (20f)	Widespread distribution of fine-grained alluvium throughout most of unit.

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UPGRADING ABSORPTIVE AGGREGATES
BY CHEMICAL TREATMENTS

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Dah-yinn Lee, Department of Civil Engineering, Iowa State University

ABRIDGMENT

•IN the design of bituminous paving mixtures, it has been generally recognized that absorption of bitumen by the aggregate plays an important role. It is one of the factors that determine the amount of asphalt actually present on the surface of the aggregate particles and available as a binder as well as the voids properties of the mixture. If proper correction of the asphalt absorbed is not made, the mixture may become more susceptible to weathering. For this reason, many aggregates have been classified as unsuitable for highway construction because of their high absorption characteristics.

The main objective of this study is to investigate if it is feasible to reduce the absorptive capabilities of the aggregates by various chemical treatments without detrimentally affecting the aggregates or mixtures, thus making highly absorptive aggregates suitable for use in asphalt paving mixtures.

Materials and Methods

Two Iowa aggregates—one from Menlo quarry in Adair County and one from Cook quarry in Story County—have been studied. Aggregates from Menlo quarry have been classified as magnesium limestone and those from Cook quarry as dolomitic limestone; both are considered unsuitable for asphalt paving mixtures.

The water absorption of graded Cook aggregate was 7.6 percent; the water absorption of graded Menlo aggregate was 2.6 percent. Two asphalt cements were included in this study, one 85-100 penetration and the other 120-150 penetration. The chemicals included are (a) aniline, (b) furfural, (c) Armac T, and (d) methyl methacrylate.

The treatments were made on both graded crushed aggregates and cored rock cylinders. The aggregates were treated and cured separately with the selected chemicals on a bench scale and in accordance with procedures developed during preliminary investigations. The amount of chemicals varied from $\frac{1}{2}$ percent to 9 percent by weight of the aggregate. The following tests were conducted on the treated aggregates: (a) water absorption by ASTM C 127; (b) heat stability at 400 F for 4 hours; (c) asphalt absorption by bulk-impregnated specific gravity method; and (d) mixture evaluation by Marshall method.

Results and Discussions

Aniline-Furfural Treatment—Graded crushed aggregates were treated with aniline-furfural on a 2:1 molar basis. Treated aggregates were cured at 220-230 F for 5 days before absorption tests were run. The results were generally very favorable. Sharp decreases of water absorption and asphalt absorption were found with 3 percent chemical (Figs. 1 and 2).

Rock cores were soaked in 2:1 aniline-furfural for 1 hour and cured at 220-230 F. Results of tests conducted on cores indicated that soaking was more effective than mixing.
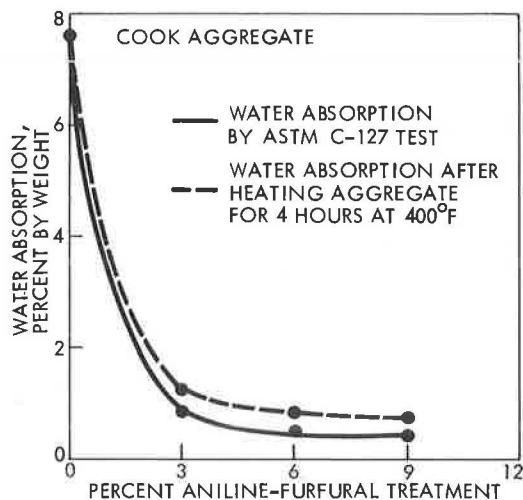
Figure 1. Water absorption of Cook aggregate versus percent aniline-furfural treatment.
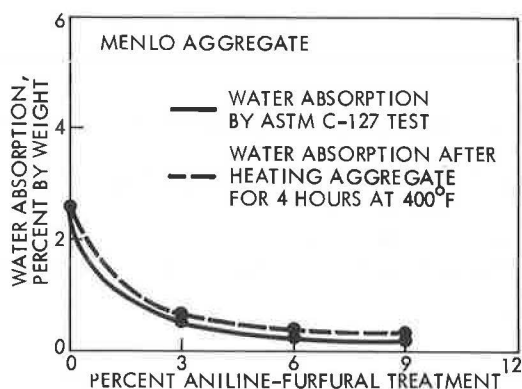

Figure 2. Water absorption of Menlo aggregate versus percent aniline-furfural treatment.

Although standard curing was done at 220-230 F for 5 days, curing curves were also established for two other temperatures, 140 F and 325 F. It was found that curing at 225 and 325 F can be considered complete after 24 hours, whereas curing at 140 F takes about 100 to 120 hours. The effectiveness of aniline-furfural treatment was also dependent on curing temperature; curing at lower temperatures was more effective than at higher temperatures.

Armac T Treatment—Results of tests conducted on the aggregates treated with different percentages of Armac T showed an appreciable reduction in both water and asphalt absorptions with increasing amount of Armac T (Fig. 3).

Curing curves for graded aggregates treated with Armac T were determined at 140 F, 225 F, and 325 F. It was found that the required curing time was dependent on curing temperature,
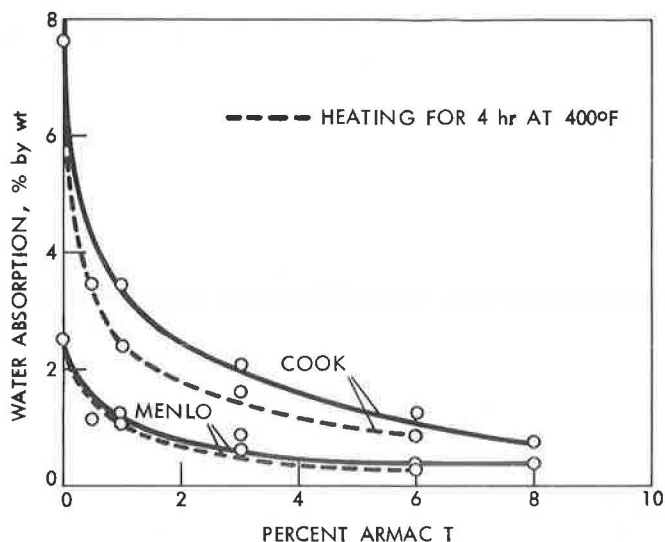

Figure 3. Water absorption versus percent Armac T treatment.

percentage of chemical, and type of aggregate. Except for 140 F curing, which required 100 to 120 hours, curing can be considered complete within 24 hours. Curing at higher temperature decreases the effectiveness of absorption reduction.

Monomer Treatment—Rock cores were first impregnated by soaking in the liquid monomer consisting of 90 percent methyl methacrylate plus 10 percent trimethylolpropane trimethacrylate. They were then polymerized by two methods, cobalt gamma radiation and thermal catalytic technique. The same degree of improvement in the reduction in water and asphalt absorption properties of the rock cores was obtained in both cases. Absorption was reduced to near zero percent for both aggregates with either polymerization technique.

Graded crushed aggregates were impregnated with the monomer and then thermally polymerized at 185 F. Drastic reduction in both asphalt and water absorption resulted. Heat stability of monomer-treated aggregates was excellent. No change in absorption values was found due to heating at 400 F. This is mainly due to the cross-linking of the monomer with trimethylolpropane trimethacrylate, one of the functions of which is to increase the softening point of the monomer solution.

Asphalt Concrete Mixture Evaluation—To determine the performance of the chemically treated aggregates in asphalt paving mixtures, asphalt concrete mixtures using treated and untreated aggregates were prepared.

Mixtures were prepared with untreated aggregates from Cook and Menlo quarries with 6 to 9 percent of 120-150 penetration asphalt cement. In both cases agricultural lime passing the No. 8 sieve was added so that the combined gradation met the Iowa Type A mix gradation. On the basis of tests conducted on Marshall specimens, it was found that a 7.5 percent asphalt by weight mix was necessary for both aggregates. It was also found that for an asphalt content of 7.5 percent the resistance of bituminous mixtures to the detrimental effect of water, as expressed by percentage of retained strength, were of the order of 74 and 77 percent for mixtures containing Cook and Menlo aggregate respectively.

Bituminous mixtures were made with aggregate treated with 2:1 aniline-furfural using the same asphalt cement and Marshall specimens. It was found that for aniline-furfural-treated aggregates, the optimum asphalt content for mixtures containing Cook aggregate was reduced to 5.5 percent and for those containing Menlo aggregate, 4.8 percent. The percentage of retained strength of the mixtures at optimum asphalt contents was 83 and 82 percent respectively (Fig. 4).
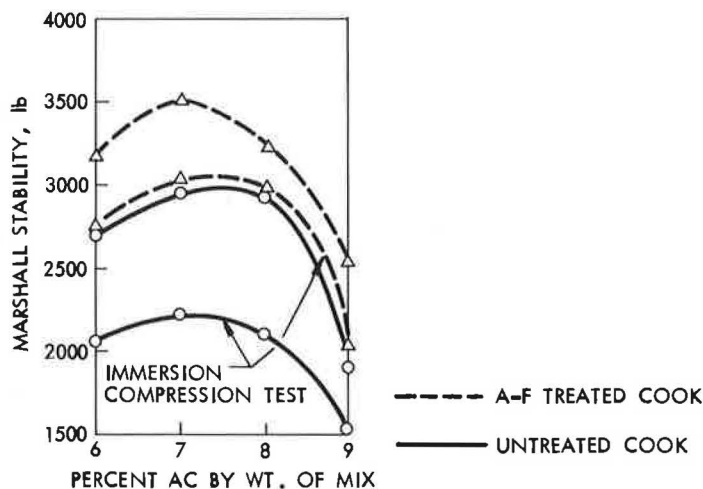
Figure 4. Marshall stability and immersion compression of mixes containing treated and untreated Cook aggregates.

Bituminous mixtures were also prepared aggregates treated with Armac T. Optimum asphalt content for mixtures containing Cook aggregate was 5.6 percent, for those containing Menlo aggregate, 5 percent. The percentage of retained strength of the mixtures at optimum asphalt content was 81 and 84 percent respectively. The lowering of the optimum asphalt content and considerable increase in the percentage of retained strength of the chemically treated aggregate bituminous mixtures can be attributed to the effective chemical treatment of these aggregates.

Conclusions

From the evaluation and analysis of tests conducted on the treated and untreated absorptive limestones, the following conclusions may be drawn:

1. Three to 6 percent of 2:1 aniline-furfural or Armac T could reduce water and asphalt absorption appreciably.

2. Impregnation of cylindrical cores and graded crushed aggregates with methyl methacrylate plus 10 percent trimethylolpropane trimethacrylate and polymerized by either radiation or thermal catalytic techniques substantially reduced water and asphalt absorptions.

3. Tests of asphalt paving mixes with treated and untreated aggregates show that a considerable saving in asphalt requirement in paving mixtures can be effected by chemical treatment.

4. A considerable increase in the percentage of retained strength of the asphalt concrete mixtures made with the chemically treated aggregates was obtained.

Acknowledgment