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Drainage and engineering soil maps prepared from aerial photographs aid in the selection of highway locations

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BUREAU OF PUBLIC ROADS
U. S. DEPARTMENT OF COMMERCE

E. A. STROMBERG, Editor

ppraisal of Soil and Terrain Conditions

r part of the Natchez Trace Parkway

ported by **PRESTON C. SMITH**[1]
pervising Highway Physical
search Engineer

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

Drainage and engineering soil maps are valuable aids in the selection of highway locations between fixed termini. They are also helpful in the location and design of drainage structures, and provide useful information on soils and other construction materials and on terrain conditions, needed by the engineer in the planning of highway design and construction.

Lacking accurate published maps, aerial photographs provide an excellent base for the preparation of the drainage and soil maps. The proper interpretation of landform, airphoto color tone, drainage characteristics, and other physical features of the aerial photographs, when studied stereoscopically, reveals a great deal about the soil and terrain conditions of the area, particularly when correlated with field investigations and study of available information on the geology and soils of the area.

The study reported in this article, of a section of the Natchez Trace Parkway, was undertaken by the Bureau both to provide information on soil and terrain conditions of the particular area, and also to investigate the technique of airphoto interpretation and to develop a symbol system for the designation of geologic material, terrain conditions, and engineering characteristics of soils.

Introduction

ORMALLY, in highway location, either cultural interests or some outstanding ographic features indicate where a high- y must be located at specific points in region to be traversed. Considerable itude may be permitted in location of highway between those points. Stand- ls of design, esthetics, and economy must considered. Several routes may satisfy design and esthetic requirements be- en any two control points, but one route l be more economical than the others. ainage and engineering soil maps pre- red from aerial photographs and other rces of information can be used as aids the selection of that route. The prepa- ion of strip soil maps for use in deter- nation of the most feasible location of a posed highway is not new. In 1925, se[2] described how such maps could be pared from soil maps published by the reau of Soils, United States Department Agriculture.

fter the location of the highway has n established, the materials engineer st obtain information on soils and other struction materials, and on terrain con- ons. The engineering soil map should serve as a guide to determine where field investigations should be concentrated. The collected information will be used in design and construction of the road. The design should be based not only on standardized charts and formulas and the information obtained in the field investigation of this particular area but also on the performance of other roads in areas having similar soil and environmental characteristics.

Although aerial photographs had been used in highway location prior to 1940, they were not used extensively for determination of soil and terrain conditions of highway and airport sites until the advent of World War II. During the last decade several organizations, including the Bureau of Public Roads, Civil Aeronautics Administration, Army Corps of Engineers, Office of Naval Research, and Purdue University, have sponsored projects wherein airphotos were used in identification of soils for engineering purposes. The Bureau of Public Roads has cooperative projects with the State highway departments of New Jersey and Maine for the preparation of soil maps on an area basis, using airphotos insofar as applicable.

In order to determine further the applicability of airphoto interpretation of soil and terrain conditions to highway location and design, the Bureau undertook an engineering soil study of a portion of the Natchez Trace Parkway.

The Natchez Trace, extending from Nashville, Tenn., to Natchez, Miss., was the overland return route, through some 460 miles of forest wilderness and Indian lands, for the crews of flatboats which floated down to New Orleans and were sold there with their cargoes. Its period of use extended from 1798 until 1817, when steamboats began to dominate the Mississippi River traffic.

Congress approved the construction of the Natchez Trace Parkway, extending along the historic route, in 1934. However, the final location of many sections of the parkway has not been made, even though the highway has been completed in some sections and construction is under way on others.

The Bureau anticipated that with the aid of airphotos a generalized engineering report and soil and drainage maps could be prepared which would (1) aid in establishment of the final location of the highway, (2) aid the materials engineers in making a detailed soil survey and locating construction materials along the established highway location, and (3) either aid in determination of a highway design standard for a particular area or, where the design standard has already been established for the area, indicate where the design might need alteration because of local soil or terrain conditions.

There were two secondary objectives. First, it was desired to obtain information concerning some of the techniques involved in the airphoto interpretation of soil and terrain conditions and the preparation of engineering soil strip maps and reports. Second, it was desired to develop a system of symbols for designation of terrain conditions and engineering characteristics of soils. It was believed that the trial area should contain a variety of soil and terrain conditions, and for that reason the proposed 62-mile section of parkway between Gravelly Springs, Ala., and U S 45, near Saltillo, Miss., was selected for study. In that region the final location of the highway had been established only at the termini and at the railroad crossing at Cherokee, Alabama.

[1] his report is a condensation of a thesis presented ornell University in June 1950 in partial fulfillment he requirements for the degree of Master of Civil ineering. Captain Smith is at present on military e, assigned to the Corps of Engineers, Fort Bel- . Va.

[2] ield methods used in subgrade surveys, by A. C. e. PUBLIC ROADS, vol. 6, No. 5, July 1925.

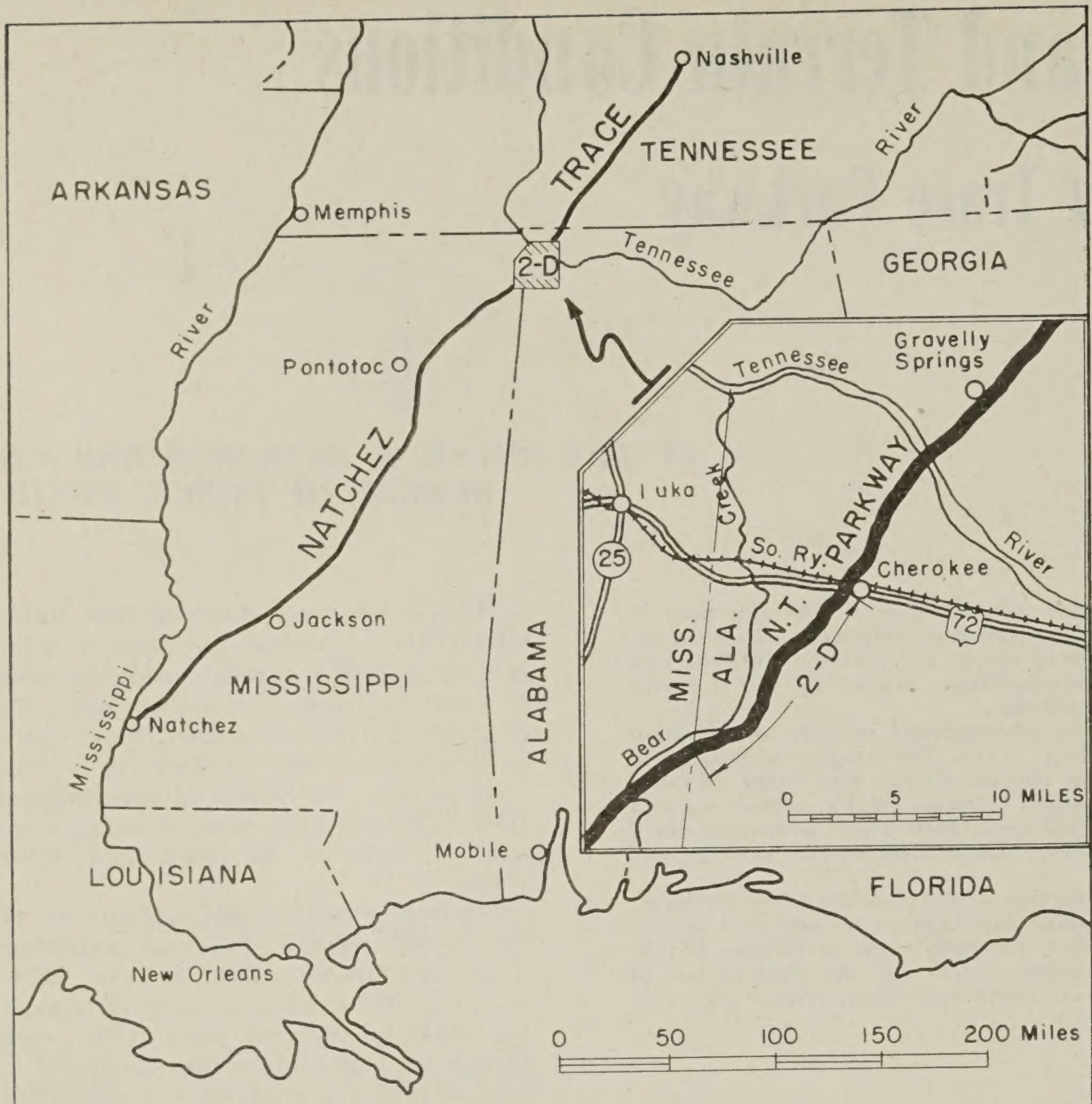

Figure 1.—Location map of the Natchez Trace Parkway and section 2-D.

While this report discusses the principles involved in preparation of drainage and soil maps from aerial photographs, detailed information is confined to section 2-D, one of the six parkway construction sections involved in the basic investigation. As shown in the inset in figure 1, section 2-D extends from U S 72 at Cherokee southwestward to the Alabama-Mississippi boundary line, a distance of approximately 12 miles. The section traverses an area which has considerable variation in topograpy and geology.

Conclusions

Although the accuracy of delineation of the map unit boundaries and description of some of the engineering characteristics of the units produced by this study will not be determined until a more detailed field investigation is made and actual construction is undertaken, it is believed that the following general statements regarding the use of the maps prepared primarily from aerial photographic information are warranted:

1. Where there are two or more possible alternate routes for the highway, soil and drainage maps will aid in determination of the best route.
2. The engineering soil map will aid the materials engineer in locating places at which an extensive soil survey should be made, probable sources of borrow, and possible sources of aggregate.
3. When the engineer has limited knowledge concerning construction materials in a region, the soil map and description of soil map units will provide information useful in the design of the highway section and pavement.
4. The drainage map can be used to determine the locations of and the areas to be served by highway drainage structures.
5. The soil map is generalized and, as a consequence, small, isolated areas of map units have not been delineated. A reasonably precise boundary sometimes cannot be established in the transition zone between some map units, particularly when material from a higher unit is washed onto a lower unit.
6. When the ground water condition or internal drainage of a unit is of particular importance to the highway engineer, he should make a field investigation of the condition. Determination of the normal internal drainage potential of a unit is difficult to determine from airphotos, particularly when there has been considerable rainfall just prior to the time the photographs were taken.

The following conclusions were reached in regard to techniques involved in analysis of soil characteristics and terrain conditions and preparation of soil and drainage maps:

1. The system of symbols proposed by Lueder and used with some modification in this report satisfactorily portrays the most important soil profile and environmental characteristics of the map units. The system of symbols can readily be mastered, for the symbols are suggestive of particular characteristics of map units.
2. By airphoto interpretation alone, it is sometimes impossible to determine in which of two similar soil groups the map unit soil should be placed. For example, differentiation cannot usually be made between A-2 and A-3 or A-6 and A-7 soils without field investigation.
3. While it is sometimes necessary to use more than one soil group symbol to indicate the variation in the soil profile, the use of the multiple numbers is an excellent indication of extreme variations in the soil strata.
4. The preparation of soil and drainage maps at airphoto scale by making an airphoto overlay, followed by photographic or some other method of reduction to a desired scale, is preferred when a base map must also be prepared. When a base map is available, however, some other method of transfer of the airphoto-delineated soil boundaries and drainage lines to the base map may be preferred.

Drainage Map

A drainage map shows the location of streams and their drainage areas, thus aiding the engineer in the determination of the proper location and design for drainage structures. Drainage channels can readily be discovered by stereoscopic examination of airphotos. In special cases, the map aids in determining the extent and characteristics of soils or geologic materials.

For the benefit of the highway engineer, streams and soil boundaries shown on drainage and soil maps should be located with respect to either adjacent cultural features or established ground control points. No large-scale maps suitable for these purposes have been published which cover all of the area under consideration. A base map was therefore prepared from airphotos, having a scale of approximately 3 inches to 1 mile, using contact prints obtained from the Production and Marketing Administration of the U. S. Department of Agriculture. These provided stereoscopic coverage for a 3-mile width of land containing the preliminary line of the parkway.

Normally, the successive airphotos in a flight strip overlap more than 50 percent, while adjacent flight strips overlap about 20 percent. In making the overlay, since the greatest distortion (perspective displacement) of features occurs at the margins of photographs, only the central portion of each photograph was used. The base map has slight variations in scale

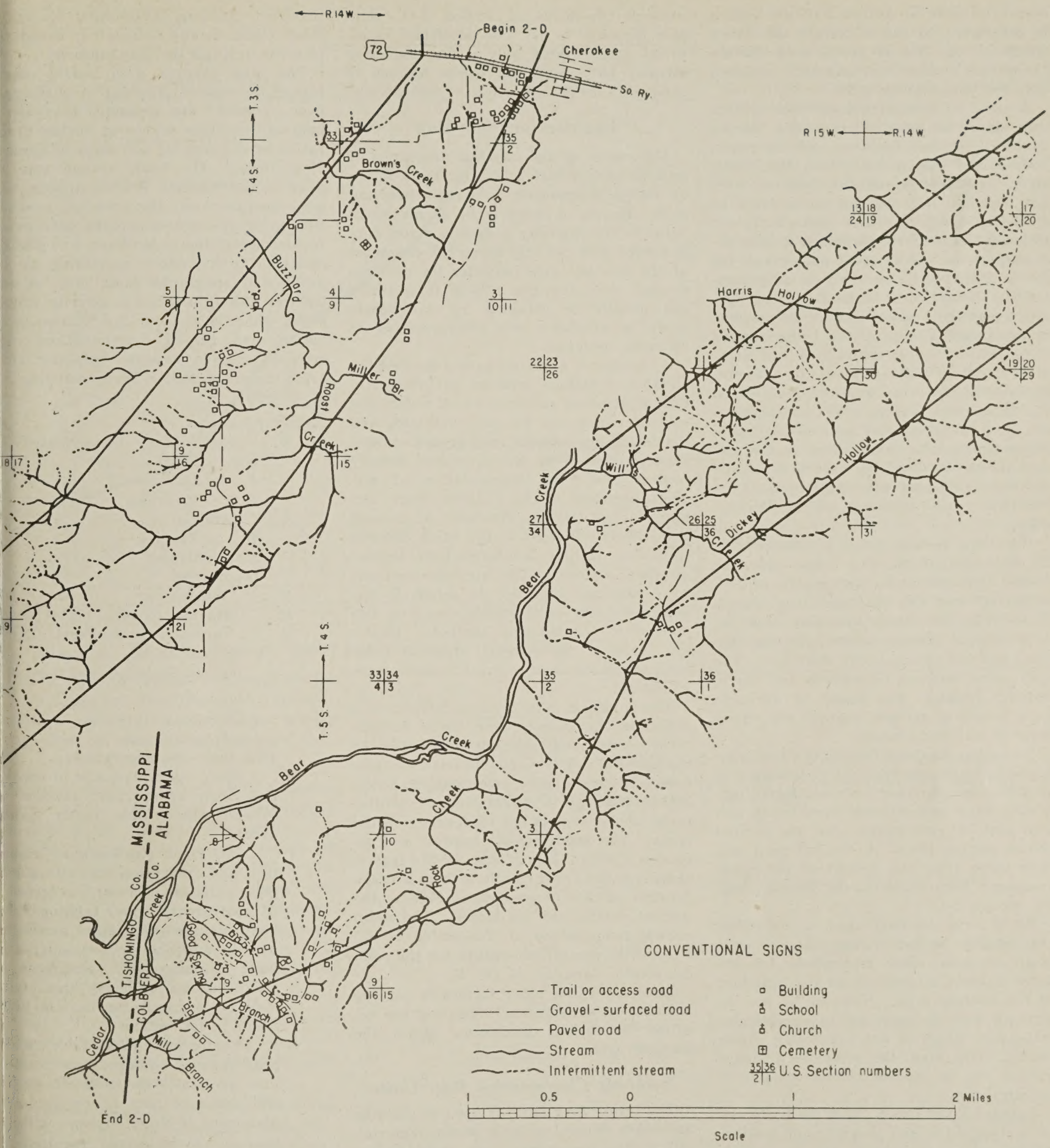

Figure 2.—Drainage map of section 2-D.

cause distortion increases with distance om the center of the airphoto and the ale at the center of one photograph may ot be exactly the same as that of another otograph. However, if an average scale used and measurements on the maps are ade from the nearest cultural feature, oints or boundaries on the maps can be cated with reasonable accuracy in the field. With a limited amount of field work and identification of cultural features on airphotos, photogrammetric methods could have been used to produce a more accurate base map. However, it was decided that the greater expenditure of effort would not be justified by the limited additional benefit derived.

The preliminary survey line (P-line) having been marked on the airphotos, approximately parallel boundary lines for mapping were established by connecting arbitrarily chosen points on each side of, and about one-half mile from the P-line. Although the final location of the highway may vary from the P-line in some areas, it will usually be contained within the mapped strip. Hereafter, in this report, when reference is made to section 2-D, the section is considered to extend across the entire width of the 1-mile strip, even though the section number was originally assigned only for the parkway width.

A sheet of transparent cellulose acetate was placed on an airphoto and a tracing made of cultural features such as roads, houses, and cemeteries within the 1-mile strip. Adjacent prominent features were also traced. Most of the roads could be traced by use of the base photograph, but to locate some of the buildings it was necessary to use the adjacent photograph (in line of flight) and view the pair stereoscopically. Using United States Geological Survey topographic maps insofar as possible, and soil survey maps of the United States Department of Agriculture or State Highway Planning Survey maps for the remainder of the area, sufficient features on the maps were identified on the airphotos to permit the location of most of the United States public land section lines on the acetate sheet. Some of the field lines on the airphotos coincided with section lines, but "bridging" was necessary in uncultivated areas.

With the acetate sheet still secured to the base photographs, the 1-mile strip was viewed stereoscopically (using pairs of photographs) and the drainage lines traced. A solid or continuous line was used for a permanent stream, while a broken line was used for intermittent drainage. The distinction between permanent and intermittent drainage was based on drainage area, length of stream, channel size, character of soil, and geology.

When the tracings from individual photographs were completed for the entire construction section, they were assembled into a strip map and oriented north-south and east-west in conformity with the United States section lines. A finished map was then made from the assembled individual tracings. Figure 2 shows the drainage map for section 2-D.

On a strip drainage map, a watershed can usually be determined only for the small streams, since watersheds for the large streams extend beyond the boundary of the mapped area. However, if the design engineer is interested in a particular watershed which is not completely shown on the strip map, the complete drainage system can be studied stereoscopically on the airphotos.

Buzzard Roost Creek (near the north end of section 2-D) and downstream sections of its major tributaries are on soluble limestone; hence, there is but little surface drainage. However, dendritic drainage patterns have developed near the source of the major southern tributaries of the creek where the soils have developed from either unconsolidated coastal plain material or calcareous shale.

Except for areas along Bear Creek (near the south end of the section) and near the mouths of its major tributaries, the dendritic pattern is continued through the southwestern two-thirds of section 2-D. Surface drainage is not well developed on alluvial areas. Also, only a few tributary streams have developed where bedrock is exposed in the lower portions of valley walls.

Engineering Soil Map

Knowledge of the physical properties of soils alone is not sufficient for the solution of many engineering problems involving soils. Hence, in order to be of maximum value to the highway engineer, a soil map is needed that not only shows the character of the soil but also indicates its environmental characteristics. Such information can usually be obtained by stereoscopic study of airphotos and interpretation of airphoto patterns.

A number of authors[3] have described the significant factors involved in interpretation of airphoto patterns which reflect soil characteristics and terrain conditions, and the details are therefore not presented here. In accordance with the procedure usually recommended, the interpretation of airphoto patterns on the Natchez Trace section started with the fundamental concept that, when subjected to the same climatic conditions, similar landforms have similar airphoto patterns. The airphoto patterns were analyzed in order to subdivide the landforms into units which would be significant to the highway engineer. Drainage, erosion, topography, airphoto color tone, vegetation, and cultural features were considered.

The landform of a geologic formation is influenced by climatic and biotic factors, relief, time, and inherent properties of the material. The climate is relatively uniform in the area under investigation; hence, little difference in weathering of similar materials having similar topography should occur. The mean annual rainfall at Tuscumbia, Colbert County, Ala., is 49.4 inches. October is the driest month, with an average rainfall of 2.8 inches, while March, the wettest month, has 5.8 inches. The mean annual temperature at Tuscumbia is 61° F., and the mean temperature for the coldest month, January, is 42° F.

Interplay of the other factors in varying amounts on each parent formation has resulted in various landforms within the mapped area.

Symbols Characterize Map Units

The term "map unit" is used to describe an area in which landform, parent material, soil profile, topography, and drainage are relatively uniform. Soil mapping, then, involves obtaining information concernin those elements and delineating boundari of areas in which they are uniform.

The map symbols used should readi convey the desired information to the eng neer. Lueder[4] has presented a system map-unit symbols developed during prep ration of engineering soil maps of areas New Jersey. His basic system was a plied in the Natchez Trace mapping, wi some modifications; the principal modific tion being the use of a dash to separa factors pertaining to landform and geolog material from factors pertaining to so profile and topographic conditions. A syr bol was also introduced to describe grou slope, which is of value for location, exc vation, and run-off determinations. Th symbols used to describe map-unit cha acteristics in Natchez Trace mapping a as follows:

Parent formation:

- *S* . . . Sedimentary (consolidated)
  - *h* . . . Shale
  - *l* . . . Limestone
  - *s* . . . Sandstone
- *A* . . . Alluvial
  - *T* . . . Terrace
  - *R* . . . Recent
  - *O* . . . Old
- *W* . . . Windblown
- *M* . . . Marine (coastal plain)
- *V* . . . Variation (in thickness, densit or composition of strata; us only on the left side of the dash.

Internal drainage:

- *e* . . . Excellent (granular materia ground water table at such dep that it is not significant).
- *g* . . . Good (permits traffic or excav tion soon after rain; position ground water table rarely signi cant).
- *i* . . . Imperfect (trafficability usua poor and excavation impractical du ing significant periods; has occasio al high ground water table, partic larly in alluvium; raised grade li usually desirable in lowlands; ar ficial drainage and placement either subbase or heavy base cour may be required for flexible pav ments in cut sections).
- *p* . . . Poor (ground water table usua near ground surface; raised gra line necessary in lowlands; arti cial drainage may be required a placement of either subbase or hea base course is usually required f flexible pavements in cut section excavation difficult during the wint season and for a considerable peri after a rain).

Ground slope:

- *l* . . . Level (range in slope from 0 3 percent).
- *u* . . . Undulating (usually between and 10 percent but may have so

[3] *The engineering significance of soil patterns,* by D. J. Belcher. Proceedings of the Highway Research Board, vol. 23, 1943, p. 569. *The origin, distribution, and airphoto identification of United States soils,* by D. J. Belcher, D. S. Jenkins, L. E. Gregg, and K. B. Woods. Technical Development Report No. 52, Civil Aeronautics Administration, 1946. *The engineering significance of landforms,* by D. J. Belcher, Bulletin No. 13, Highway Research Board, 1947, p. 9. *Aerial photographs: their use and interpretation,* by A. J. Eardley. Harper & Brothers, 1942. *Identification of granular deposits by aerial photography,* by R. E. Frost. Proceedings of the Highway Research Board, vol. 25, 1945, p. 116. *Aerial photographs and their applications,* by H. T. U. Smith. D. Appleton-Century Company, Inc., 1943.

[4] *A system for designating map-units on engineer soil-maps,* by D. R. Lueder. Bulletin No. 28, High Research Board, 1950, p. 17.

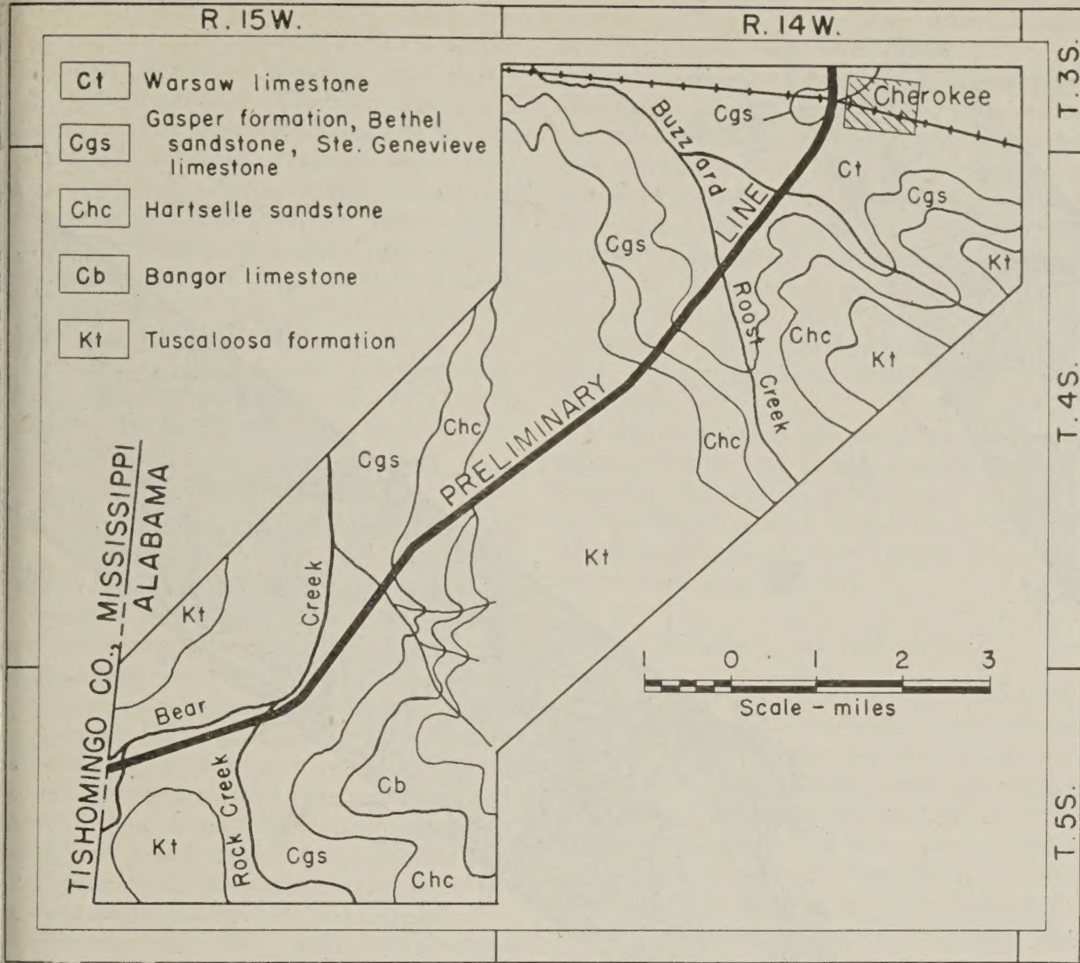

Figure 3.—Geologic map of area traversed by section 2-D.

flatter or steeper slopes; steeper slopes are short).

s . . . Steep (most of slopes steeper than 10 percent, but may have small areas with flatter slopes).

Highway Research Board soil classification.[5] Group number is indicated by a numeral, 1 to 7. Only predominant groups, referring to material within significant depth, are shown in map unit designation. Highway Research Board subgroup numbers, such as 7–6, are not used. Combined numerals, such as 24, indicate that the soils of the map unit vary, either laterally or vertically, but are principally in A-2 and A-4 groups.

Horizon: The Symbol *C* indicates significant difference between B and C horizons and is placed on the right side of the dash since it is a soil profile symbol.

When the mapping is so generalized that one symbol will not adequately describe a particular characteristic of the map unit, a combination of symbols may be used. For example, *us* on the right side of the dash indicates that ground slopes of the map unit vary from undulating to steep. The combination *ls* on the left side of the dash indicates alternate beds of limestone and sandstone, neither of which has been separated into an individual map unit. If the limestone overlies the sandstone, the combination should be *l/s*.

The internal drainage symbol indicates the drainage potential of the soil due, primarily, to its textural and structural properties. However, in landforms for which the ground water table is a factor in highway location, design, or construction, the symbol also reflects the approximate depth to the water table during prolonged wet periods. By reference to the other symbols of the map unit, it is usually possible to determine if a particular internal drainage symbol denotes a drainage condition which is due primarily to soil properties. By use of airphotos, without field investigation, it is impossible to estimate the precise depth to the ground water table for most of the map units, since an area may have been photographed after a considerable dry period when the water table is low.

An important advantage of the developed system is that the symbols are applicable to certain engineering factors which may be ascertained from geologic reports, topographic maps, and agricultural soil bulletins. Thus, published material can be used to supplement the information obtained from aerial photographs. The type of parent material and some idea concerning the

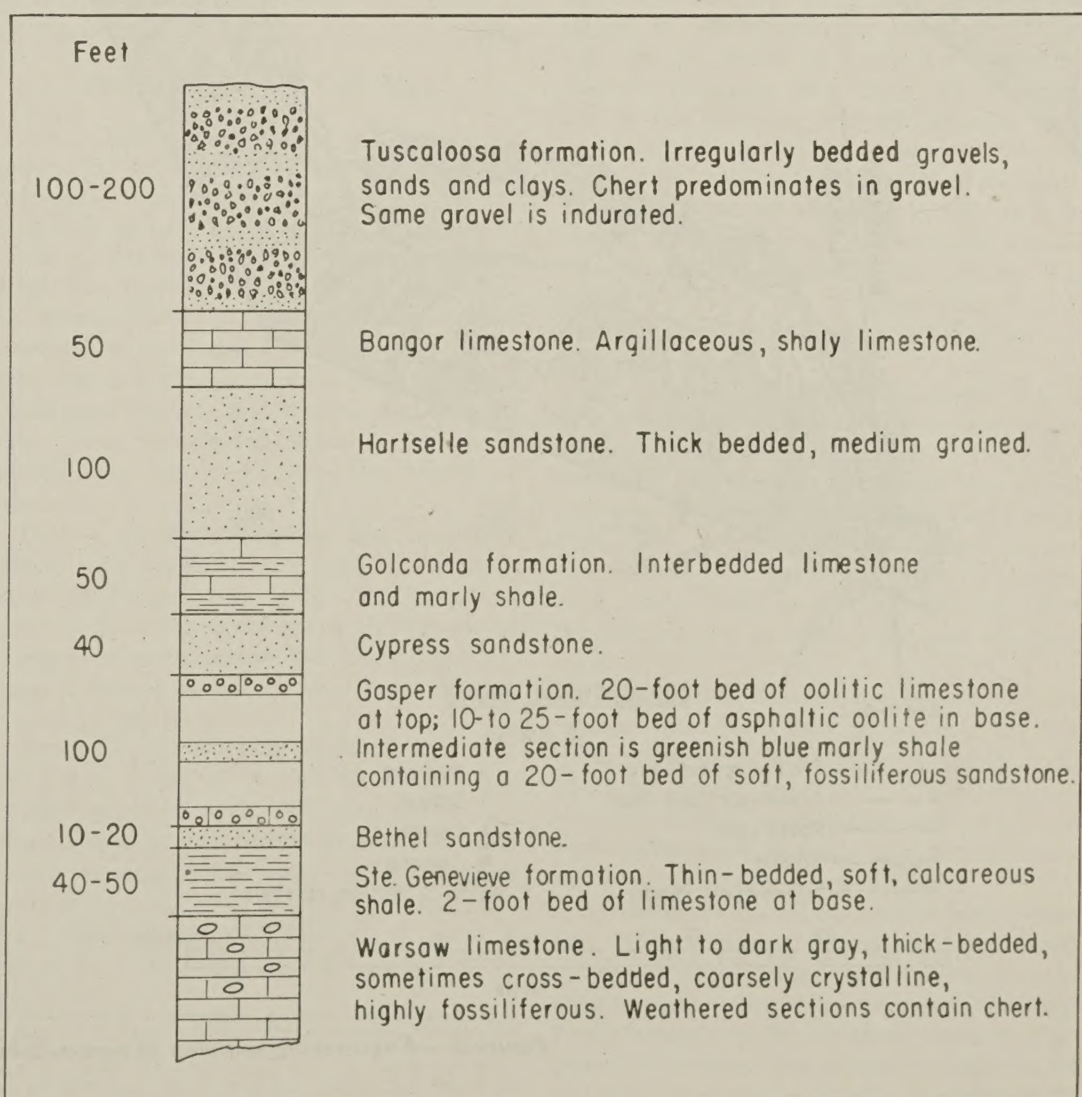

***Figure 4** (at right).—Approximate geologic columnar section through highlands southwest of Cherokee, Ala.*

[5] *Classification of highway subgrade materials.* Report of Committee on Classification of Materials for Subgrades and Granular Type Roads, Proceedings of the Highway Research Board, vol. 25, 1945, p. 375.

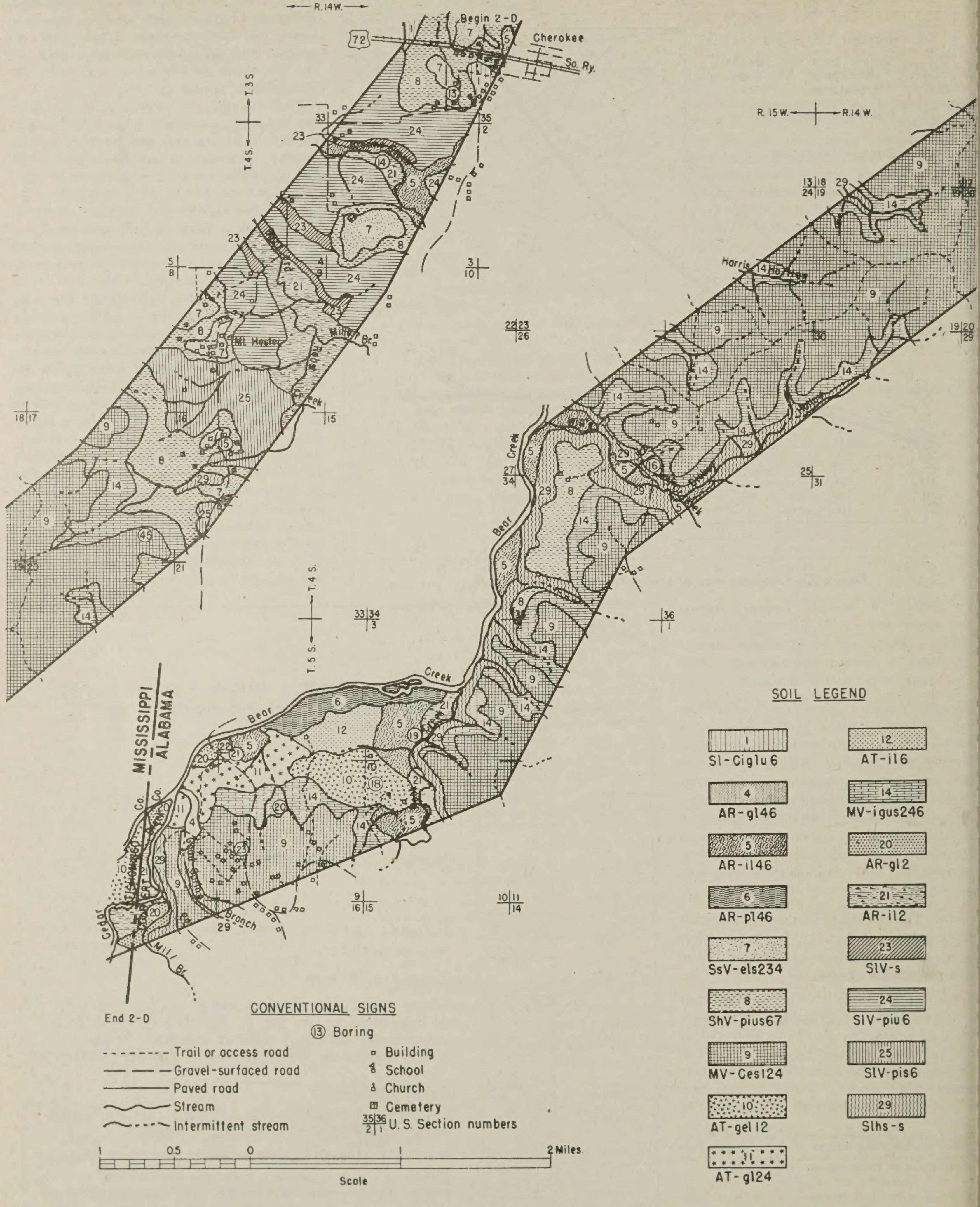

Figure 5.—Engineering soil map of section 2-D.

lerived soils can be determined from geologic reports and maps. Ground slopes and major drainage channels are shown on topographic maps. The agricultural soil bulletins give a description of soil profile, topography, internal drainage, and generalized geology. Some of the engineering characteristics indicated by other basic sources of information may be precisely determined by the use of aerial photographs. The remainder of the factors can be inferred by study of airphoto patterns. Use of geologic and agricultural reference material reduces the amount of field work necessary to verify the inferences.

Mapping Procedure

The initial office study consisted of two parts. First, geologic and soil[6] reports concerning the area to be traversed by the Parkway were studied. Second, a preliminary stereoscopic study of airphotos for the area was made, including a generalized correlation of airphoto patterns with various parent materials, soils, and terrain conditions likely to be encountered in parkway construction. Road cuts and other exposures in the vicinity of the proposed parkway where a soil profile for a typical airphoto pattern or parent material could be observed were marked on the airphotos.

Following the office study, the areas marked on the airphotos were inspected in the field. If the soil profile or ground condition was that anticipated from airphoto study, or was in conformity with that exhibited for a similar airphoto pattern already investigated in another area, no further study of that area was necessary. However, if a soil profile or condition differed from that anticipated from the airphoto pattern, further study was undertaken in order to determine the features of the airphoto pattern which would depict the soil profile and ground condition. The field reconnaissance resulted in correlation of airphoto patterns with profile and other ground conditions to such an extent that a tentative definition of soil map units for the entire section could be made.

Using a red China-marking pencil, boundaries of soil map units were marked on the same photographs from which the base map was prepared. Each soil area was indicated by a number which corresponded to a definite map unit. An area in which there was some question about either the map unit or its boundaries was marked on the adjacent photograph, in order that it could be given further office or field study. By additional study of geologic and soil reports, the map unit boundaries were established for some of the questionable areas.

Each of the remaining unmapped areas was studied in the field. Soil profiles were examined in road cuts where possible. Otherwise, borings were made. Boundaries were either marked on the airphotos at that time or sufficient information was obtained to permit office delineation.

Typical map unit locations were also examined in the field. For map units having appreciable depth of soil, a sample was usually taken from each significant soil stratum below the A-horizon. Samples were also taken to show the variations in physical properties of soils within the map unit. If borings or field observations showed that map unit boundaries or descriptions were inaccurate, corrections were made on the airphotos in the field or in the office.

A final definition of map units was made after the soils had been tested in the laboratory. The soil map was then prepared as described below. Work was done on the detailed report concurrently with soil testing and map drafting.

Geology of the Area

The topography of the area traversed by section 2-D is quite variable. The ground surface ranges from relatively narrow ridges with steep slopes, some elevations in excess of 700 feet, and local relief of as much as 300 feet, to low rounded hills separated by relatively broad valleys. Consequently, a variety of geologic formations of Mississippian and Upper Cretaceous age are exposed in the area.[7] Figure 3 is a geologic map of section 2-D, and a columnar section through the formations is shown in figure 4.

Preparation of Soil Map

After field investigation and laboratory testing of soils verified that the airphoto-delineated map units were correct, a soil map was prepared. A tracing was made of cultural features and principal streams from the drainage map already completed. Locations of borings from which soil samples were taken were also shown on the tracing. The marked airphotos were then properly positioned beneath the tracing and the soil map unit boundaries traced. The completed map for section 2-D is shown in figure 5.

On the soil map a number and a conventional map pattern are used to identify each delineated map unit. The numbers have no inherent significance. They merely serve as a quick means for identifying the map patterns, as will be seen from the soil legend in figure 5.

The scale of the map depends upon the use to be made of it and the complexity of the mapped area. If map units are large and only a few units are encountered, relatively small-scale maps—even less than an inch equals 1 mile or smaller—may suffice. Maps having a scale of at least 2 inches to 1 mile are usually preferred by the field engineer in order that he may record reconnaissance notes on the map. However, since the detail on the map cannot be greater than that of the airphotos, the maximum practical scale is that of the latter (approximately 3 inches to 1 mile for this study).

Characteristics of Map Units

It is understood, of course, that on a soil map prepared primarily by airphoto interpretation the information is generalized. While the map and description of map units for section 2-D may be sufficiently detailed for use in preliminary location and estimates, some portions of the final location will require more detailed information. The soil map will usually indicate where the field investigation of soil and ground conditions must be concentrated.

The engineering soil map units delineated in section 2-D differ in some characteristics which are of importance to the highway engineer. Even though some of the landforms were divided into several maps units, it was revealed by borings, field observation of soil profiles in road cuts, and laboratory soil test results that considerable variation both laterally and vertically exists within some units. Some of the variations are described in the discussion of the map units which follows. More detailed information concerning the variations in soil profiles within the map units may be obtained by reference to the summary of soil test results shown in table 1. Since the original mapping was for a 62-mile portion of the parkway and soil samples were taken which would be representative of map units within that length, some map-unit soils were not sampled in section 2-D. Hence, table 1 shows some soil test results for map units occurring in section 2-D but for which the representative soil samples were taken from another construction section.

The probable Highway Research Board group number was used in the description of the soil for those map units having appreciable soil mantle. This was considered permissible in such highly generalized mapping, even though the stated soil groups were based on a limited number of laboratory tests and field observations. The test data given in table 1 support the estimates.

The map unit descriptions do not indicate whether materials may be suitable for use as aggregates in bituminous mixtures or in portland cement concrete. The determination of quality, availability, and extent of aggregates must be made in a detailed survey by the materials engineer.

Although some of the engineering characteristics of map units may be ascertained by direct stereoscopic observation of airphotos, other characteristics are deduced from features of the airphoto pattern. Figures 6–14 show the airphoto patterns of the most extensive map units in section 2-D.

[6] *Soil survey, Colbert County, Alabama.* U. S. Department of Agriculture, Bureau of Chemistry and Soils. Series 1933, Number 22, issued February 1939.

[7] Information concerning geology of the area traversed by Section 2-D was obtained from *Geology of Alabama*, by G. I. Adams, C. Butts, L. W. Stephenson, and W. Cook. Special Report No. 14, Geological Survey of Alabama, 1926. Figures 3 and 4 are based on this report.

Table 1.—*Representative soil test data for section 2-D map units*

Map unit designation	Boring number[1]	Depth of sample below surface	Soil horizon	Mechanical analysis: cumulative percentage passing— ¾-inch	⅜-inch	No. 4	No. 10	No. 40	No. 200	Smaller than 0.005 mm. (clay)	Physical test constants: Liquid limit	Plasticity index	Shrinkage limit	Shrinkage ratio	Moisture-density: Maximum dry density	Optimum moisture	Highway Research Board classification: Group index	Group
		Inches													*Lb./cu. ft.*	*Percent*		
Sl-Ciglu6	2	48-72	C	100	99	99	98	96	83	37	38	15	16	1.9	107	18	10	A-6
Do	8	15-28	B	-----	-----	-----	100	99	78	39	40	16	15	1.9	110	17	10	A-6
SsV-els 234	11	18-24	B	-----	-----	-----	100	99	71	29	28	11	15	1.9	-----	-----	7	A-6
Do	11	42-60	C	-----	-----	-----	100	99	69	35	34	14	13	1.9	-----	-----	8	A-6
Do	11	72-90	D	-----	-----	-----	100	99	59	36	35	13	14	1.9	115	15	6	A-6
Do	11	96-108	E	-----	-----	-----	-----	100	37	23	25	8	19	1.8	-----	-----	0	A-4
Do	11	114-132	F	-----	-----	-----	-----	100	32	13	19	NP	18	1.8	117	12	0	A-2-4
ShV-pius 67	13	28-36	B	100	95	93	91	89	49	32	33	15	16	1.9	114	14	4	A-6
Do	13	56-66	C	-----	-----	-----	100	99	77	37	38	16	15	1.9	111	18	10	A-6
Do	13	84-96	D	-----	-----	-----	100	99	73	29	31	11	16	1.8	-----	-----	8	A-6
Do	15	8-15	B	-----	-----	-----	100	99	95	81	77	47	14	1.9	-----	-----	20	A-7-5
Do	15	20-28	C	-----	-----	-----	100	99	90	64	64	36	12	1.9	-----	-----	20	A-7-6
MV-Ces 124	23	10-30	B	99	98	97	96	91	81	35	36	15	17	1.8	116	16	10	A-6
Do	23	36-60	C	85	55	37	30	25	11	28	32	11	25	1.6	112	16	0	A-2-6
Do	45	10-30	B	96	87	73	63	54	45	32	42	21	16	1.8	-----	-----	5	A-7-6
Do	45	36-60	C	100	98	96	95	71	26	22	29	11	18	1.8	-----	-----	0	A-2-6
Do	45	96-120	D	87	71	54	43	18	15	31	47	23	21	1.7	122	12	0	A-2-7
MV-igus 246	20	10-18	B	-----	-----	-----	100	98	85	34	35	13	17	1.8	-----	-----	9	A-6
Do	20	24-36	C	-----	100	99	97	93	75	28	29	10	16	1.8	120	14	8	A-4
Do	20	48-60	D	92	81	73	66	58	42	30	32	13	16	1.9	115	14	2	A-6
Do	20	80-96	E	99	96	93	89	85	72	45	51	30	13	1.9	111	18	17	A-7-6
Do	20	108-120	F	-----	-----	-----	100	97	89	56	50	30	11	2.0	-----	-----	18	A-7-6
AT-gel 12	18	10-18	B	96	93	90	86	76	55	30	34	16	15	1.9	-----	-----	6	A-6
Do	18	20-36	C	99	85	73	63	47	31	33	42	19	17	1.8	-----	-----	1	A-2-7
AT-gel 24	21	8-15	B	-----	-----	-----	100	98	64	29	32	13	13	1.9	-----	-----	7	A-6
Do	21	36-48	C	-----	-----	-----	100	98	48	26	27	10	15	1.9	122	13	3	A-4
Do	21	84-108	D	-----	-----	-----	100	99	45	23	24	8	17	1.8	122	12	2	A-4
AT-il 6	34	12-18	B	-----	-----	-----	100	99	80	32	38	17	18	1.8	-----	-----	11	A-6
Do	34	36-48	C	-----	-----	-----	100	99	67	27	31	13	15	1.9	-----	-----	7	A-6
AR-gl 2	22	12-18	B	-----	-----	-----	100	84	16	6	NP	NP	-----	-----	-----	-----	0	A-2-4
Do	22	36-48	C	-----	-----	-----	100	84	15	5	NP	NP	-----	-----	111	12	0	A-2-4
AR-il 2	14	24-30	B	-----	-----	-----	-----	100	63	35	34	16	14	1.9	-----	-----	8	A-6
Do	14	48-54	C	-----	-----	-----	-----	100	34	19	22	7	16	1.9	-----	-----	0	A-2-4
AR-il 46	19	8-18	B	-----	-----	-----	-----	100	95	46	42	19	19	1.8	-----	-----	12	A-7-6
Do	19	48-72	C	-----	-----	-----	-----	100	80	29	28	10	18	1.8	118	14	8	A-4

[1] Approximate locations of borings are shown on figure 5. Borings Nos. 2, 8, 11, and 34 were made in other sections of the parkway.

In the following descriptions of the map units encountered in section 2-D, some units are grouped on the basis of geologic material while others are grouped according to landform.

Units on cherty limestone

Sl–Ciglu 6.—The unit has level to undulating topography underlain primarily by slightly friable A-6 soil to a depth greater than 15 feet and containing angular chert, which ranges up to 8 inches in size and to 25 percent of total material, in the subsoil. Soil from the B-horizon, which extends to a depth of 3 to 8 feet, is more plastic than from the C-horizon and soils from both horizons may be A-7. Much of the surface water is collected in depressions or sinkholes and removed through subterranean channels. In undisturbed condition the soil has good internal drainage; remolding destroys the soil structure and decreases the permeability. Bedrock occurs at such great depth that it is not a source of aggregate. Airphotos show the mottled sinkhole pattern and general lack of surface drainage development. The unit is usually cultivated.

SlV–piu 6.—The airphoto pattern of this unit is illustrated in figure 6. The topography is generally undulating, with some short steep slopes, some limestone depressions, and underlain by bouldery, plastic A-6 soil; bedrock normally occurs at a depth of less than 10 feet. Limestone boulders on or near the ground surface will interfere with earth work. Depressed areas have a high ground-water table during substantial periods. Much of the surface water is collected in depressions and discharged through subterranean channels; hence, some faintly discernible gullies disappear at the edge of depressions. Although limestone might be quarried from this unit and used for road surfacing, the quarry opening will normally be made in the adjacent unit, *SlV–s*. The airphoto color tone is medium gray mottled with darker gray. On nearly level areas the dark areas are connected to form a phantom drainage pattern.

SlV–pis 6.—The airphoto pattern of this unit is also shown in figure 6. The topography is rolling and underlain by bouldery, plastic A-6 soil, with limestone bedroc normally at a depth of less than 10 fee Limestone boulders, exposed in some area will interfere with earthwork. Althoug the internal drainage is poor to imperfec surface drainage is good. Extensive sur face drainage channels have been forme because of the steep ground slopes.

SlV–s.—The unit is composed of valle walls in which limestone is either expose or near the ground surface; slopes var from 10 percent in shallow soil areas t nearly vertical in rock exposures. Whe there is a soil mantle it is shallow and co tains angular chert or cherty limestone fra ments. Bedrock exposures may be sourc of aggregate. On airphotos the identity

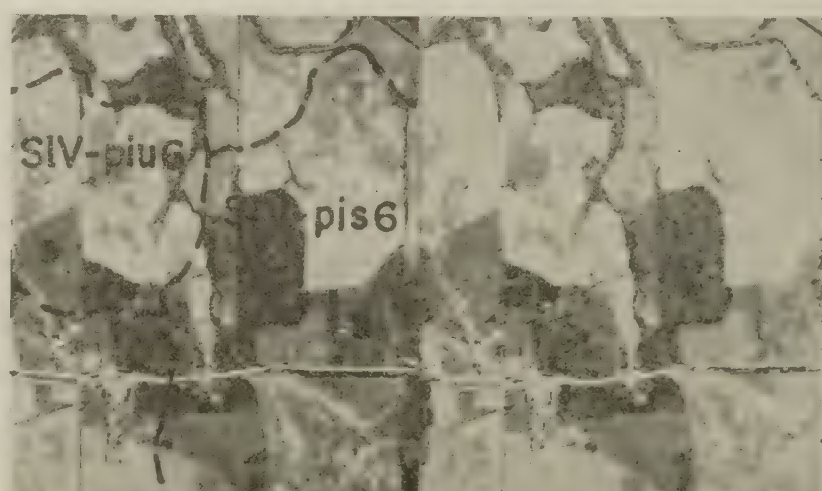

Figure 6.—Airphoto stereo-pair illustrating map units SlV-piu 6 and SlV-pis 6. Boulders are exposed in some areas as at B.

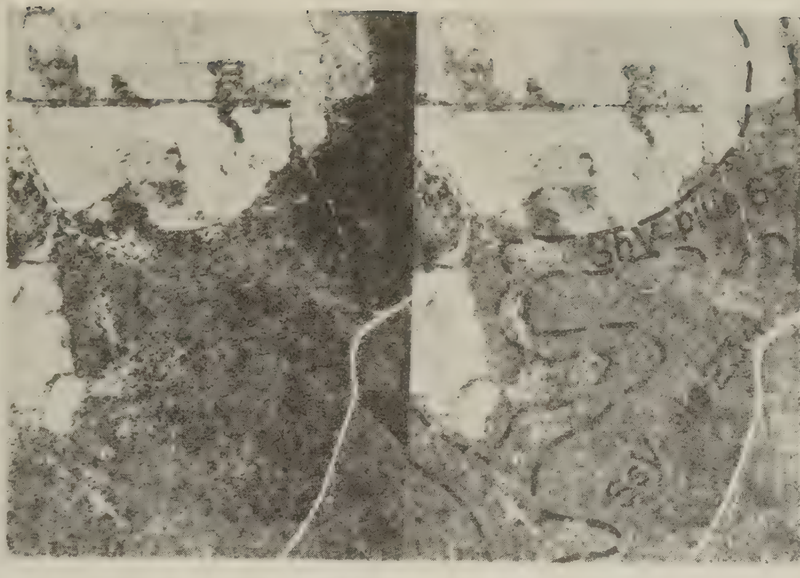

Figure 7.—Airphoto illustrating map units **SsV-els 234** *and* **ShV-pius 67.**

s unit is established by determination it adjacent units are derived from limene.

its on sandstone, calcareous shale, and interbedded sandstone, shale, and limestone.

SsV–els 234.—The airphoto pattern of s unit is illustrated in figure 7. The it occupies tops of hills and ridges, usuy wooded; ridge tops may be more than) yards wide and are nearly flat, but e slopes are steep. Normally, weakly nented, weathered sandstone occurs at depth of 2 to 4 feet, the C-horizon soil A-2 or A-3, and the B-horizon soil is able A-4. However, some hills and ges of this unit are capped with coastal in soils, and weathered bedrock may be encountered within a depth of 10 t. The A-2 or A-3 soil and weathered idstone may be used in a subbase but the idstone is usually not of such quality it it can be used for base course or suring material. However, if an exposure asphaltic sandstone is located, its suitability as aggregate should be investigated. Neither internal nor external drainage should be a problem in this unit. Watersheds are small and most of the rainfall is absorbed by the porous soil; hence drainage channels are not well developed. A landscape view is shown in figure 8.

ShV–pius 67.—The airphoto pattern of this unit is also shown in figure 7. The unit is either woodland or pasture. It occupies 5- to 20-percent slopes and normally has a mantle of elastic A-6 or A-7 soil, 2 to 10 feet in depth, with poor or imperfect internal drainage. Erosion has exposed the weathered shale in some areas. Within the usual depth of cuts the weathered shale is easily excavated; blasting will be required in deep cuts. Some areas of this unit are underlain by interbedded shale and limestone and excavation in the limestone will require blasting. The limestone, particularly that containing asphalt, may be suitable base course or surfacing material.

Slhs-s.—The airphoto pattern of this unit is shown in figure 9. The unit has slopes which usually range from 20 to 90 percent and are underlain by undifferentiated limestone, shale, or sandstone formations. The materials are exposed as nearly vertical cliffs in valley walls or occur on such steep slopes that bedrock will be encountered in shallow cuts. The flatter slopes probably have some soil mantle as well as a variable depth of weathered rock; delineation of each material cannot be made on the airphoto. Well-defined stream channels have been cut in the rock slopes, while the small tributary streams at higher elevation have no well-defined channels where there are porous, coastal-plain soils. Unweathered sandstone or limestone, particularly asphaltic limestone, may be suitable base course or surfacing material. Most of the areas are wooded.

Units on coastal plain deposits

MV-Ces 124. — The airphoto pattern of this unit is shown in figure 10. The unit occupies dissected upland areas underlain by stratified A-1, A-2, and occasional A-4 or even A-6 soil. Chert gravel predominates and usually occurs in strata having a thickness of 5 to 20 feet or more, with maximum particle size of about 4 inches. The unit contains the most extensive deposits of possible base course gravel in the region, although excessive clay, induration of gravel, and poor gradation precludes the use of many exposed deposits. Both surface and internal drainage of the unit are usually good but seepage may occur at the top of plastic soil strata. The ridge tops have undulating topography but the side slopes are steep and relatively straight. Local roads and trails on the ridges are quite circuitous. Gullies and minor tributary streams pursue relatively straight courses.

MV-igus 246.—The airphoto pattern of this unit is also shown in figure 10. The unit occurs downslope from *MV-Ces 124* and consists of material washed from the

ure 8. — Easily weathered calcareous shales are usually exposed below a more resistant formation. Here, the steep area in the distance is map unit SsV-els 234 *(soil formed from sandstone) while the flatter slope in the foreground has developed on the shale unit* ShV-pius 67.

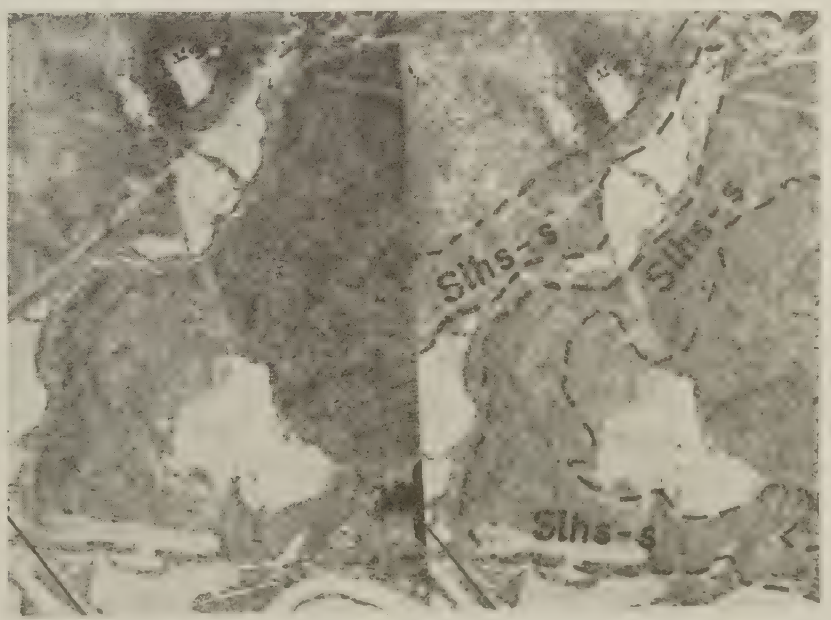

Figure 9.—Airphoto illustrating map unit Slhs-s.

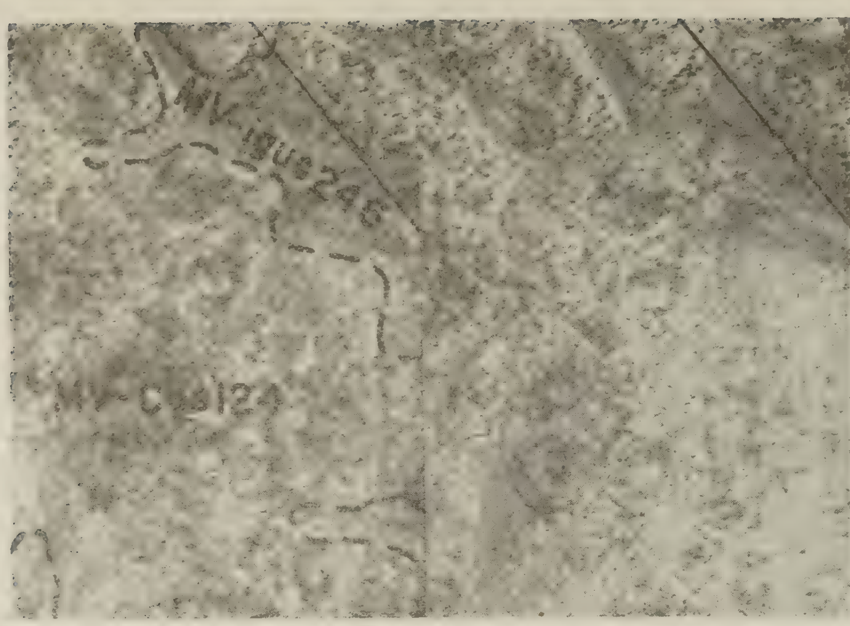

Figure 10.—Airphoto illustrating map units MV-Ces 124 *and* MV-igus 246.

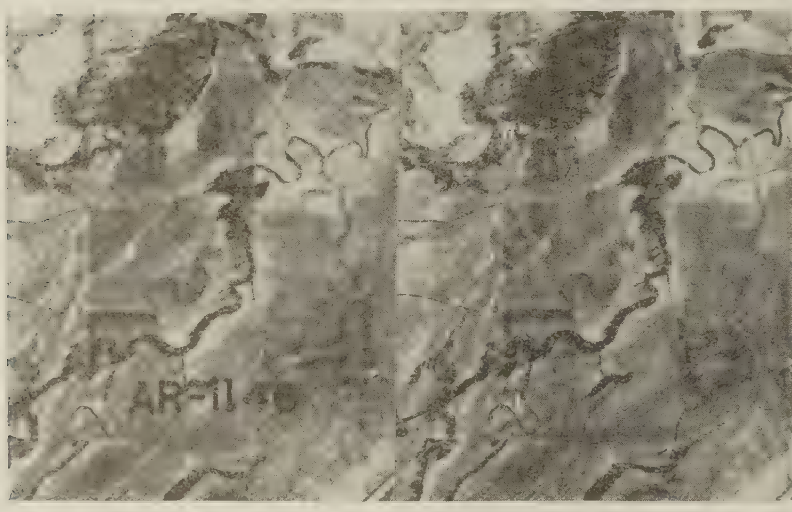

Figure 11.—Airphoto illustrating map unit AR-il 46.

higher unit onto residual soil derived from limestone, shale, or sandstone. Slopes usually range from 10 to 30 percent. The wash material varies from a few feet to about 12 feet in depth and may be A-2, A-4, or A-6 soil. Underlying the wash material is usually A-6 soil but it may be A-7. Internal drainage is usually good in the wash material, but downward movement of ground water will be retarded by the plastic residual soil. In some areas the unit is deeply dissected. Deep cuts involve some excavation in the plastic residual soil.

Units on recent alluvium

AR-gl 46.—This inextensive unit occupies flood plains which are rarely actually flooded. The soils are well-drained, friable, A-4 or A-6, and may contain some sand or gravel, particularly along existing or abandoned stream channels. The airphoto color tone is uniform dull gray.

AR-il 46.—The airphoto pattern of this unit is shown in figure 11. The unit occupies flood plains of which portions are normally flooded each year and other portions less frequently; all portions have a high ground-water table during other significant periods. Soils are plastic A-4 or A-6 but the profile may contain some strata of fine silty sand. Some areas have abandoned channels which contain organic clay. Airphoto color tone is dull gray mottled with darker gray, indicating silt or silty clay.

AR-pl 46.—The unit may be flooded each year and has a permanent high ground-water table. The soil is normally a plastic A-4 or A-6 soil but may have strata of silty sand. Depressed, wooded areas and abandoned channels contain organic clay. The airphoto color tone is medium to dark gray mottled with darker gray or black.

AR-gl 2.—The airphoto pattern of this unit is shown in figure 12. The unit occurs primarily as a natural levee adjacent to a stream and has a high water table only during flood stages, but is normally well-drained. Although the material is usually A-2 soil, there may be considerable variation in texture both horizontally and vertically, and some strata may contain gravelly sand or sandy gravel. The topography is nearly level but may have some shallow stream markings.

AR-il 2.—The unit occurs in either a natural levee of a stream or on the flatter portion of the flood plain, may be flooded more than once each year, and has a relatively high ground-water table during significant portions of each year. In general, the soil is A-2, but there may be thin strata of either A-4 or A-6 soil. Some areas have abandoned channels which may contain organic clay. Higher areas of the unit have a light gray to white airphoto color tone.

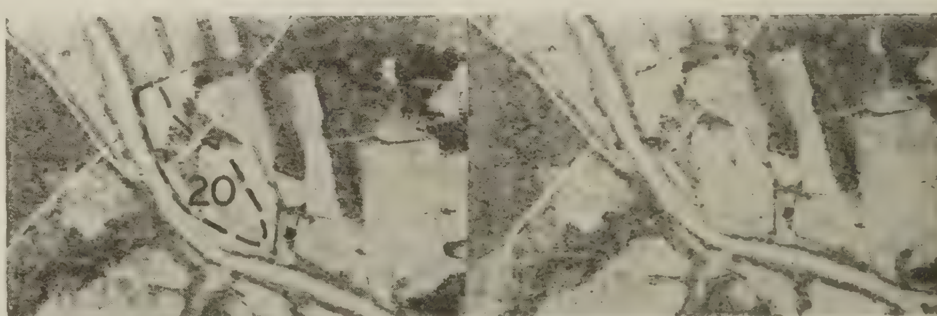

Figure 12.—Airphoto illustrating map unit AR-gl 2.

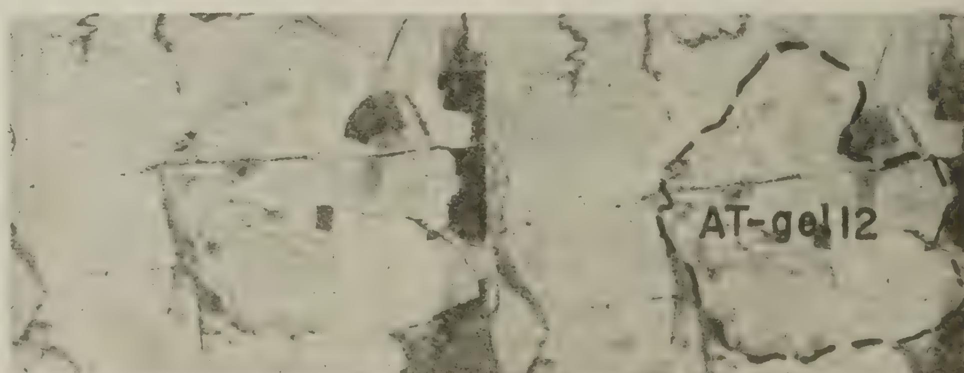

Figure 13.—Airphoto illustrating map unit AT-gel 12.

AT-gel 12.—The airphoto pattern of this it is shown in figure 13. The unit occupies slightly elevated positions along major streams and is rarely flooded; its ground surface varies from flat to gently undulating. There is little surface drainage; the outer terrace wall is steep, dissected by short, steep-walled gullies. The horizon soil may be A-4 or A-6 with some gravel, but the underlying soil is usually 1 or A-2 with considerable clay and silt in the upper portion. Extreme vertical variation in textural gradation should be anticipated. Use as base course material is questionable, but some deposits should be good sources of borrow for use in adjacent embankments crossing poorly drained alluvial soils. The over-all medium gray color tone indicates that the surface soil may be gravelly clay, while the gray mottled areas have a thicker surfacing of clayey soil.

AT-gl 24.—The airphoto pattern of this unit is shown in figure 14. The unit occupies a position about 15 to 30 feet higher than the adjacent flood plain and the soils are usually friable, well-drained A-2 or A-4. Streams from the upland continue across the unit but surface drainage does not usually develop on the terrace. The light gray color tone with only minor mottling indicates that the material is sand. It may be a good source of borrow.

AT-il 6.—The unit occupies level to gently undulating areas 20 to 75 feet above the adjacent flood plain. The soil is usually 6 but in some areas the B-horizon has 7 soil and substrata are A-4. Internal drainage is imperfect and extensive surface drainage channels have developed in some areas. It may have a hardpan at a depth 2 to 5 feet. In addition to the foregoing features, airphotos show the terrace has a gently sloping outer terrace wall (except where there has been recent stream cutting) and the color tone is dull gray mottled with darker gray.

Adequacy of Symbol System

It is believed that the system of map unit symbols proposed by Lueder and modified in this report adequately describe the most important soil profile and environmental characteristics of the area investigated, and that the coding system can readily be mastered since most of the symbols are suggestive of particular characteristics of map units. In preliminary location work the highway engineer is primarily interested in topographic details, with less consideration being given to soil conditions. In design and construction phases, the engineer is interested in topographic details, but soil characteristics, geology, and drainage conditions are also of primary importance. The symbols denote these characteristics in a generalized manner, while the detailed descriptions of map units enlarge upon the characteristics.

Figure 14.—Airphoto illustrating map unit AT-gl 24.

Adequacy of the mapping for the Natchez Trace Parkway will only be determined when the maps are used. If it is determined that more generalized mapping will suffice for future projects, the generalization can be done either by combining two or more symbols used to describe a particular characteristic of a unit or by eliminating one or more of the characteristics from the map unit designation. More detailed mapping could be accomplished by delineating smaller areas as map units and by using additional symbols to describe other characteristics of the map unit.

Geologic and agricultural soil survey units corresponding to the engineering soil map units delineated in section 2-D are listed in table 2. In general, the engineering map is more detailed than the geologic map. In agricultural soil mapping, the surface soil is usually of the greatest importance, whereas the engineer is also interested in lower soil horizons. An engineering map unit may be either a subdivision or combination of agricultural soil types.

Design and Construction Problems

Many problems of highway design and construction can only be solved by field study or application of previously obtained knowledge concerning a similar problem. Information relating to problems which are characteristic of individual map units has been noted in the descriptions of the engineering characteristics of the units. Engineeding information concerning characteristics common to several map units is presented in table 3.

Frost does not penetrate the soils to a great depth, and below-freezing periods are of relatively short duration, in the area under consideration; hence, frozen soil is not a major deterrent to earthwork. However, there is considerable rainfall during winter and early spring months, which delays construction in granular soils and prevents grading operations in fine-grained soils for substantial periods.

In some map units, particularly in limestone areas, the depth to ground-water table at any time varies considerably over relatively small areas. When the water table is deep it usually has no influence on highway construction, and when at a moderate depth it is of consequence only in deep cuts. The designation "shallow" in table 3 indicates that the water table may be at or near the ground surface for significant periods; "high" indicates those areas which may be inundated for significant periods, with the water table near the ground surface during the remainder of the year.

In any map unit the road surface should be at such an elevation that the pavement will not be adversely affected by the ground water table. In alluvial map units, the

Table 2.—*Correlation of engineering map units with geologic materials and agricultural soils*

Engineering map unit	Geologic material	Agricultural soil type
Sl-Ciglu 6	Cherty limestone	Dewey loam.
SlV-piu 6	do	Do.
SlV-pis 6	do	Do.
SlV-s	do	Rough stony land.
ShV-pius 67	Limestone, calcareous shale, and sandstone	Colbert clay or silt loam.
Slhs-s	do	Colbert clay.
SsV-els 234	Sandstone	Atwood gravelly loam and Dickson silt loam.
Mv-Ces 124	Tuscaloosa formation (gravel, sand, and clay)	Atwood gravelly loam and Guin undifferentiated.
MV-igus 246	do	Guin undifferentiated.
AR-gl 46	Recent alluvium	Ochlockonee fine sandy loam.
AR-il 46	do	Ochlockonee fine sandy loam or silt loam and Huntington silt loam.
AR-pl 46	do	Huntington silt loam.
AR-gl 2	do	Ochlockonee fine sandy loam.
AR-il 2	do	Do.
AT-gel 12	do	Cahaba gravelly fine sandy loam.
AT-gl 24	do	Kalmia fine sandy loam.
AT-il 6	do	Kalmia loamy fine sand.

Table 3.—*Engineering data and recommendations for map units*

Map unit designation	Adapted to winter grading	Normal position of water table	Location of grade line with respect to ground surface	Erosion resistance	Possible source of— Borrow	Granular subbase material	Base course material
Sl-Ciglu 6	No	Variable	Shallow cuts in ground swells; fills in depressions.	Good	Limited	No	No
SlV-piu 6	No	Shallow	do	do	No	No	No
SlV-pis 6	No	Variable	Influenced by bedrock	do	No	No	No
SlV-s	Limited	Deep	do	do	No	No	Yes
SsV-els 234	Yes	do	Anywhere	do	Yes	Limited	No
ShV-pius 67	No	Variable	do	Fair to poor	No	No	No
Slhs-s	Limited	do	Influenced by bedrock	Good to fair	No	No	Yes
MV-Ces 124	Yes	Deep	Anywhere	Good	Yes	Yes	Yes
MV-igus 246	Limited	Variable	do	Fair	Yes	Limited	No
AR-gl 46	No	Moderate	Above high water		Limited	No	No
AR-il 46	No	Shallow	do		No	No	No
AR-pl 46	No	High	do		No	No	No
AR-gl 2	Yes	Moderate	do		Yes	Yes	No
AR-il 2	No	Shallow	do		Limited	Limited	No
AT-gel 12	Yes	Moderate to deep.	Anywhere, if above high water.	Good	Yes	Yes	Yes
AT-gl 24	Yes	do	Anywhere	do	Yes	Limited	No
AT-il 6	No	do	do	do	No	No	No

grade line should be sufficiently above high water elevation to prevent damage to the pavement.

In those units designated as limited sources of borrow or subbase material, the soil is either of such poor quality that it should be used only when better material cannot be obtained, or it is difficult to excavate because of high ground-water table during significant periods.

Since there are extensive sources of gravel in the mapped area, it is probable that a pavement section composed of gravel base course and bituminous concrete surface course will be adopted for section 2-D. However, the normal base course thickness might be reduced in those areas where the subgrade is composed of granular soils, whereas an above-normal thickness of pavement may be required in areas where the subgrade is composed of very plastic soils. In some areas the necessity for a thicker base course may be alleviated by topping the subgrade with one of the materials designated as granular subbase material in table 3.

Parent formations, as well as unconsolidated materials, are represented in table 3 as sources of base course material, although it is probable that gravel will be used exclusively. Map unit *MV-Ces 124* will probably be the principal source of gravel for section 2-D, but considerable exploration may be required in order to find a deposit of gravel suitable for a base course. The terrace deposit *AT-gel 12* may have some suitable gravel, but it is limited in extent.

Time Required for Preparation of Report

So that the information obtained on the Natchez Trace study may be used in planning other mapping projects, the estimated time required, in 8-hour man-days per square mile of mapped area, for various phases of the investigation of section 2-D is given in table 4. The table does not show the amount of time required for map drafting, stenographic work, and preparation of the report, because each of those items will depend upon the use to be made of the report.

It is believed that the developed system of map-unit symbols is adequate for describing the major soil and terrain characteristics of an area; hence, any future mapping will not require any time for that phase of the investigation.

The time required for the office study of published material, correlation of airphoto patterns with soils and geology, and airphoto delineation of map unit boundaries depends upon (1) the amount of published material which must be read in order to obtain the basic information on soils and geology, (2) complexity of mapping, and (3) ability of the airphoto interpreter. Complexity of mapping depends not only upon the variations in geologic materials and environmental factors but also the purpose of the mapping. A highly generalized map may suffice for the initial highway reconnaissance survey, whereas a more detailed map will be required in a later stage of highway location.

The skill of the airphoto interpreter depends not only upon his ability to associate various features of airphoto patterns but also his experience in mapping similar areas. After the airphoto pattern for a particular map unit has been defined, airphoto delineation of the boundary between an area of that map unit and adjacent map units is usually not difficult. Mapping of a strip adjacent to a previously mapped area might require only half as much time per unit of area, because most of the map units would be repeated in the second strip.

The time required for the field survey and soil testing depends upon (1) number of map units, (2) stratification of the parent material, (3) topography, (4) amount of information previously obtained for the same map units, and (5) ability of the surveyor to select sampling sites which are representative of the particular map unit. Ordinarily, for any coastal plain map unit several strata will be encountered within the normal depth of highway excavati hence several samples will be required order to determine the characteristics the various soils. On the other hand, a residual soil map unit, the parent m terial is not usually stratified, and l sampling will be required. Borings in t areas having steep slopes or irregular top raphy must usually be deeper than in f areas; hence, more time will be requi in the field soil survey of the steep are It is probable that, because of the vari of parent materials encountered both lat ally and vertically and the irregular top raphy, the 35 samples obtained in secti 2-D are more than would normally be quired in an area of that size. If the o tests required are those for the Highw Research Board classification, the time quired for soil testing will be less than 1.0 man-day per square mile estimated section 2-D.

Frequently, the information regardi map units in one area can be applied either the same or similar map units another area, thus reducing the amount field work in the second area.

The time required for airphoto delin tion of drainage lines varies according the intricacy of dissection of the area, it is probable that less than 0.05 man-d would be required to delineate the drain in a square mile of a slightly or moderat dissected area. A highly trained airph interpreter is not required for airph delineation of drainage lines.

The primary aim of the engineering port is to present information which be used by the highway engineer; hence considerable portion of the total time quirement should be allotted for map u description and engineering recommen tions. The engineer who makes the r ommendations should have a broad kno edge of the factors involved in highw location, design, and construction. should be familiar with construction pr tices in either adjacent areas or other ar having similar soil and environmen characteristics. For each map unit he m consider the possible uses to be made of mapping, and then present engineering

Table 4.—*Time required for preparation of engineering report*

Phase of investigation	Time required, man-days per square mi... Engineer-interpreter	Other personnel
Office study of soil bulletins and geologic reports	0.15	0.[illegible]
Development of system of map symbols	0.05	0.[illegible]
Preliminary study of airphotos and correlation of airphoto patterns with soils and geology	0.25	0.[illegible]
Airphoto delineation of map unit boundaries	0.20	0.[illegible]
Field survey	0.15	0.[illegible]
Soil testing	0.00	1.[illegible]
Drainage	0.05	0.[illegible]
Map unit description and engineering recommendations	0.15	0.[illegible]
Total	1.00	1.[illegible]

›rmation which will be of value to the user.

The total of 2.25 man-days per square ›ile required for the mapping and report›ıg of section 2-D, as shown in table 4, is ›reater than for adjacent construction sec›ons of the parkway because section 2-D ›as a greater variation in geologic forma›ons and topography. Complexity of geo›gic formations and irregular topography require the expenditure of a great amount of time for mapping, particularly when the mapped strip is alined approximately in the direction of dip, as it is for section 2-D. It is probable that not more than 1.5 man-days per square mile would be required to map a mile-wide strip in a slightly to moderately dissected area having extensive exposures of relatively uniform geologic formations, and that time might be considerably reduced if the airphoto interpreter had mapped other areas having the same or similar airphoto patterns. The time would be about equally divided between the engineer-interpreter and other personnel. For a wider strip the average time required for mapping a square mile might be reduced to 1.0 man-day.

Inter-American Highway Film

Inter-American Highway Report—Part I, Mexico, a motion picture produced by the ›ureau of Public Roads, is now available ›n loan to interested organizations. The ›6-millimeter sound and color film, with a ›unning time of 55 minutes, shows the ›resent condition of the Mexican portion ›f the Inter-American Highway from Lare›o, Texas, to the Guatemalan border— ›,700 miles of excellent roads built by the ›epublic of Mexico with its own funds and ›ersonnel.

The scenes picture a realm of contrasts in a setting of magnificent scenery. There are thriving cities and relics of antiquity, primitive and modern means of transportation, native handicraft and up-to-date industrial plants, antiquated and scientifically operated farms, and many other colorful and interesting sights to capture the attention of the traveller. Typical hotel, garage, and filling-station services are shown. Animated maps represent the locations of many of the scenes depicted.

Inter-American Highway Report—Part I, Mexico may be borrowed by any responsible organization, without cost except for nominal transportation charges, by writing to the Visual Education Branch, Bureau of Public Roads, Washington 25, D. C.

Work on a companion film is now in progress. This second installment will show the condition of the continuation of the Inter-American Highway through the six Central American republics, terminating at Panama City.

Freezing and Thawing Tests of Concrete Containing Oregon and Washington Aggregates

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Senior Materials Engine

BY THE PHYSICAL RESEARCH BRANCH
BUREAU OF PUBLIC ROADS

In this study, plain and air-entrained concretes made with certain Oregon and Washington aggregates, and with Potomac River aggregates as a control, were subjected to alternate freezing and thawing after preliminary moist curing of both short and prolonged periods. Some of the concretes had poor resistance to freezing and thawing while others stood up well. There was no evidence in most cases of any alkali-aggregate reaction.

Prolonged initial curing improved the resistance to freezing and thawing of all but one of the non-air-entrained concretes. This was not true for the air-entrained concretes, and in many cases the additional curing was actually harmful. However, the air-entrained concretes with prolonged initial curing generally had better resistance to freezing and thawing than the comparable non-air-entrained concretes cured for only a short time.

In all cases, when the preliminary curing period was short, the use of an air-entraining agent produced concrete with greater resistance to freezing and thawing. With the prolonged preliminary curing, air entrainment was not beneficial to three of the five aggregate combinations.

In general, the tests indicated that the durability of air-entrained concrete may be adversely affected by conditions which involve long-continued exposure to moisture. It was also found that the results of freezing and thawing tests made after a short period of curing in moist air may not be sufficient to evaluate properly the durability of concrete containing certain cement-aggregate combinations.

IN a paper published in 1944 [1] Bailey Tremper, Materials Engineer of the Washington State Department of Highways, presented data showing that concrete containing aggregates from certain sources in eastern Washington, in combination with cements high in alkalies, when tested in freezing and thawing after prolonged preliminary moist curing (7 months), broke down quite rapidly as compared to similar concretes containing either the same aggregates in combination with low-alkali cements or neutral aggregates in combination with high-alkali cements. However, he found that when these same combinations were subjected to relatively short preliminary moist curing (28 days), all of them developed about the same resistance to freezing and thawing. He also noted that 1- by 1- by 10-inch mortar bars containing these combinations, when exposed in a saturated atmosphere at 70° F., failed to develop any significant expansion at ages up to 4 years. On the other hand, when these mortar bars were subsequently frozen and thawed, the order of deterioration followed in general the same pattern as noted for the concrete.

Tremper concluded from these data that the lack of durability observed in his tests was primarily a function of the alkali content of the cement. However, he believed that the type of reaction was distinctly different from that which had previously been reported by Stanton and other investigators of the so-called "alkali-aggregate" reaction.

The tests reported here were made to obtain additional information on this particular point as well as to study the durability of concrete made with certain other cement-aggregate combinations commonly used in Oregon and Washington.

Seventeen combinations of cements and aggregates in concrete were investigated. The materials included four fine aggregates, four coarse aggregates, and three cements from the Washington-Oregon area, Potomac River sand and gravel from the vicinity of Washington, D. C., and two cements manufactured in the Southeast, one having a high and the other a low alkali content. The combinations involving Oregon and Washington materials were selected on the basis of recommendations by Tremp and by G. S. Paxson, Bridge Engine, Oregon Highway Commission. The combinations of Potomac River sand and grav with the various cements were tested provide a comparison between the Oreg and Washington materials and combinations in which the same cements were us with an eastern aggregate. Concrete specimens containing the 17 combinations materials were tested for resistance alternate freezing and thawing in wat. Two series of tests were made—one which the freezing cycle was started the expiration of 28 days of prelimina moist curing, and the other in which freezing was started at the conclusion of 1 days of moist curing. These periods we selected in order to bring out any differen in behavior which might be due to alka action under long-continued moist curin, as observed by Tremper.

Two parallel series of tests were made—one in which the plain portland cemen were used, and the other in which sufficie neutralized Vinsol resin was added at t mixer to give an air content of approximately 4 percent in the plastic concrete.

Conclusions

The following conclusions appear warranted from the data:

1. In only two of the seventeen combinations was there evidence of a probab alkali-aggregate reaction. These were t combinations involving the two relative high-alkali cements which were used wi the sand and gravel from Rock Islan, Wash. This tendency was noted in t case of both the non-air-entrained and t air-entrained concrete.

2. Non-air-entrained concrete containing the sand from Mt. Hebron, Calif., combination with the crushed stone fro Klamath Falls, Oreg., developed very po resistance to the action of alternate freezing and thawing with all three of the cements used with these aggregates. However, no tests were made to determine t contributing factor of either aggrega, when used separately, to the poor resistance of the combination.

[1] *The effect of alkalies in portland cement on the durability of concrete*, by Bailey Tremper. Journal of the American Concrete Institute, November 1944.

Table 1.—*Properties of the cements*

	Cement				
	1	2	3	4	5
Physical properties:					
Apparent specific gravity	3.17	3.13	3.11	3.14	3.12
Specific surface ....cm.[2] per gm..	1,810	1,810	1,760	1,880	1,840
Autoclave expansion ....percent..	0.03	0.04	0	0.01	0.21
Normal consistency ....percent..	24.5	24.5	24.5	25.5	25.0
Time of set:					
Initial ....hr: min..	4:30	4:10	3:00	5:10	3:45
Final ....hr: min..	6:00	6:00	5:45	8:15	6:15
Tensile strength:					
At 3 days ....lb. per sq. in..	280	275	270	280	280
At 7 days ....lb. per sq. in..	365	365	310	355	375
At 28 days ....lb. per sq. in..	445	450	430	485	435
Sugar test:					
Neutral point ....ml..	4.5	28.7	43.3	2.8	3.9
Clear point ....ml..	4.7	38.9	60.1	2.8	4.0
Air in mortar [1] ....percent..	4.5	4.7	4.6	2.5	7.6
Chemical analyses (percent):					
Silica (SiO_2)	20.90	21.25	22.30	22.60	21.75
Alumina (Al_2O_3)	4.33	5.30	4.38	4.30	4.79
Ferric oxide (Fe_2O_3)	5.12	3.60	3.52	4.40	2.96
Calcium oxide (CaO)	64.45	63.70	65.30	64.05	63.20
Magnesia (MgO)	2.25	1.76	0.91	1.25	2.48
Sulfuric anhydride (SO_3)	1.65	1.66	1.73	1.68	1.84
Sodium oxide (Na_2O)	0.35	0.20	0.08	0.31	0.05
Potassium oxide (K_2O)	0.17	0.47	0.15	0.19	0.93
Loss on ignition	0.90	2.04	1.62	1.17	2.11
Chloroform soluble material [2]	0.005	0.013	0.006	0.015	0.004
Na_2O equivalent	0.46	0.51	0.18	0.44	0.66
Computed compound composition (percent): [3]					
Tricalcium silicate (C_3S)	61	52	57	49	50
Dicalcium silicate (C_2S)	14	21	21	28	25
Tricalcium aluminate (C_3A)	3	8	6	4	8
Tetracalcium alumino-ferrite (C_4AF)	16	11	11	13	9
Calcium sulfate ($CaSO_4$)	3	3	3	3	3

[1] A.S.T.M. method C 185-44T [2] A.S.T.M. method C 114-42.
[3] The compound compositions given are in "shorthand" form.

and the Potomac River sand and gravel. Under these conditions (180 days of curing), air entrainment did not appreciably improve the resistance of combinations containing the other aggregates. In general, improvement due to air entrainment was much less marked after 180 days of curing than after 28 days.

In general, the tests indicated that the durability of air-entrained concrete may be adversely affected by conditions which involve long-continued exposure to moisture. The results of freezing and thawing tests made after 28 days of curing in moist air may not be sufficient to evaluate properly the durability of concrete containing certain cement-aggregate combinations.

Sources of Materials

Five portland cements were used in this investigation, three from the West Coast and two from the Southeast. One of the southeastern cements was selected for its low total alkali content, the other because it contained a relatively high percentage of alkali, mostly potash. The physical properties, chemical analyses, and computed compound compositions are given in table 1. All cements complied with the standard

3. Both non-air-entrained and air-entrained concrete containing the sand and gravel from Umatilla, Oreg., and the sand and gravel from the Willamette River, Oreg., had good resistance to alternate freezing and thawing with the same three cements as were used with the Mt. Hebron-Klamath Falls aggregate combination.

4. Non-air-entrained concrete made with Potomac River sand and gravel had very poor resistance to freezing and thawing with all five cements, but there was no apparent indication of an alkali-aggregate reaction.

5. In the case of the non-air-entrained concrete, increasing the time of initial curing improved resistance to freezing and thawing for all combinations except that involving the Rock Island aggregates and the cement with the highest alkali content. In the case of the air-entrained concrete, increasing the curing time did not in general increase resistance to alternate freezing and thawing and in many cases the additional curing was harmful. However, in only one case (the Rock Island aggregate with cement 5) was the resistance of air-entrained concrete, cured for 180 days, less than that of the comparable non-air-entrained concrete cured for 28 days.

6. In all cases, for concrete cured 28 days, the use of an air-entraining agent was beneficial in producing concrete with greater resistance to freezing and thawing than was developed by the corresponding concrete without air entrainment. For concrete cured 180 days, air entrainment was definitely beneficial only in the case of combinations involving the Mt. Hebron sand with the Klamath Falls coarse aggregate

Table 2.—*Properties of the fine aggregates*

	Mt. Hebron, Calif.	Umatilla, Oreg.	Willamette River, Oreg.	Potomac River, Md.	Rock Island, Wash.
Grading: percentage passing—					
No. 4 sieve	96	95	91	98	98
No. 8 sieve	77	86	71	85	84
No. 16 sieve	60	79	59	71	67
No. 30 sieve	47	50	42	49	41
No. 50 sieve	30	10	14	18	10
No. 100 sieve	10	2	2	4	2
No. 200 sieve	3.3	0.8	0.6	1.8	0.6
Fineness modulus	2.80	2.78	3.21	2.75	2.98
Bulk specific gravity:					
Dry	2.42	2.67	2.51	2.59	2.63
Saturated surface-dry	2.51	2.73	2.58	2.63	2.66
24-hour absorption ....percent..	3.9	2.2	2.9	1.6	1.0
Sodium sulfate test: loss at 5 cycles ....percent..	10.5	5.2	6.5	....	4.1
Compressive strength ratio: [1]					
At 7 days	1.18	1.04	.87	1.13	9.8
At 28 days	.98	1.09	1.01	1.16	9.7
Tensile strength ratio: [1]					
At 7 days	.97	1.13	1.09	1.17	1.18
At 28 days	.88	1.15	1.04	1.14	1.21

[1] A.A.S.H.O. T 35-35.

Table 3.—*Properties of the coarse aggregates*

	Klamath Falls, Oreg., crushed basalt	Umatilla, Oreg., gravel	Willamette River, Oreg., gravel	Potomac River, Md., gravel	Rock Island, Wash., gravel
Grading: percentage passing—					
1-inch sieve	100	100	100	100	100
¾-inch sieve	94	64	83	75	69
½-inch sieve	50	28	44	40	42
⅜-inch sieve	28	11	13	20	10
No. 4 sieve	0	1	0	0	0
Fineness modulus	6.78	7.25	7.04	7.05	7.21
Bulk specific gravity:					
Dry	2.71	2.72	2.58	2.56	2.68
Saturated surface-dry	2.75	2.74	2.63	2.59	2.69
24-hour absorption ....percent..	1.3	0.9	2.1	1.1	0.7
Sodium sulfate test: loss at 5 cycles ....percent..	2.5	1.3	8.8	0.9	1.8
Los Angeles abrasion loss [1] ....percent..	17.6	14.6	16.4	30.3	14.6
Weight per cu. ft., dry rodded ....lb..	92	104	105	107	111

[1] Grading B.

Table 4.—*Expansion of 1:2 mortar bars, arranged in ascending order of values of alkali content of cements*

Aggregates	Expansion of mortar bars (in percent) after—									
	Moist storage at 120° F. for 3 years, using cement— [1]					Moist storage at 120° F. for 12 weeks, using cement— [1] (1 percent [2] NaOH added)				
	3 (0.18)	4 (0.44)	1 (0.46)	2 (0.51)	5 (0.66)	3 (0.18)	4 (0.44)	1 (0.46)	2 (0.51)	5 (0.66)
Mt. Hebron sand	−0.01	----	0.01	0.01	----	0.01	----	0.03	0.04	----
Klamath Falls basalt	.02	---	.13	.13	---	.04	---	.17	.13	---
Umatilla gravel	.01	---	.04	.03	---	.03	---	.09	.10	---
Umatilla sand	.01	---	.02	.05	---	.02	---	.07	.08	---
Willamette River gravel	−.01	---	.04	.02	---	.00	---	.06	.12	---
Willamette River sand	−.01	---	.02	.01	---	.04	---	.16	.12	---
Rock Island gravel	.02	0.08	---	---	0.12	.04	0.09	---	---	0.10
Rock Island sand	.00	.05	---	---	.08	.03	.19	---	---	.18
Potomac River gravel	.01	.02	.02	.03	.05	.03	.05	.06	.06	.09
Potomac River sand	.01	.02	.03	.03	.06	.02	.05	.08	.10	.09

[1] Alkali content, indicated in parentheses under the cement numbers, = percentage Na_2O + 0.658 K_2O.
[2] By weight of cement.

Table 5.—*Cement-aggregate combinations*

Aggregate combination No.	Cements	Fine aggregate source	Coarse aggregate source
I	1, 2, 3	Mt. Hebron, Calif.	Klamath Falls, Oreg.
II	1, 2, 3	Umatilla, Oreg.	Umatilla, Oreg.
III	1, 2, 3	Willamette River, Oreg.	Willamette River, Oreg.
IV	1, 2, 3, 4, 5	Potomac River, Md.	Potomac River, Md.
V	3, 4, 5	Rock Island, Wash.	Rock Island, Wash.

specifications for type I portland cement of the American Society for Testing Materials. In addition, cements 4 and 5 also met the requirements of type II cements. Cements 1, 2, and 3, while complying with type II requirements with respect to the percentage of tricalcium aluminate, cannot be classified as type II because, in each case, the amount of tricalcium silicate exceeded 50 percent.

Five fine aggregates and five coarse aggregates were used. Four of each were from the Washington-Oregon area, and Potomac River sand and gravel were used as a basis of comparison. The sources as well as the gradings and other physical properties of the fine and coarse aggregates are given in tables 2 and 3 respectively. Nothing unusual in the properties of the aggregates is apparent from a study of the values in these tables, except that the sand from Mt. Hebron, Calif., appeared to be of relatively poor structural quality, as revealed by the comparatively high sodium sulfate soundness loss and the low tensile strength of the mortar.

The petrographic composition of the fine and coarse aggregates is as follows:

Mt. Hebron sand. — Rounded rhyolite, basalt, andesite, and felsite with quartz and magnetite.

Klamath Falls basalt.—Olivine basalt.

Umatilla gravel.—Angular and subangular granite, basalt, rhyolite, quartz, and quartzite.

Umatilla sand.—Angular and subangular basalt, rhyolite, and felsite, with some granite, quartz, and limestone.

Willamette River gravel. — Rounded basalt, andesite, rhyolite, and felsite.

Williamette River sand. — Angular and subangular rhyolite and felsite, with some basalt, granite, and quartz.

Rock Island gravel. — Rounded granit syenite, rhyolite, and basalt.

Rock Island sand. — Angular and sul angular granite, rhyolite, felsite, and ba salt, with some feldspar and quartz.

Potomac River gravel.—Rounded quart quartzite, chert, and sandstone, with son granite, gneiss, and schist.

Potomac River sand.—Subangular quart quartzite, and chert, with some sandston schist, granite, and magnetite.

Mortar-Bar Tests

The results of expansion tests of 1 mortar bars are given in table 4. In ma ing the mortar bars, the coarse aggregat were crushed so that 100 percent passe the No. 4 sieve and 90 percent passed th No. 8 sieve. Only those particles of th sands passing the No. 4 sieve were use in the test. The coarse aggregates and th sands were each tested separately. Suffi cient water was used with the 1:2 morta to give a plastic consistency. The spec mens were 1- by 1- by 11½-inches in size.

One group of specimens was prepare with plain mortar, and a second group wi

Table 6.—*Summary of mix data for non-air-entrained concrete*[1]

Aggregate combination and cement	Proportions of mix, by oven-dry weight	Cement content	Net water content	Slump	Weight of fresh concrete	Calculated air content
	Pounds	*Sacks per cu. yd.*	*Gal. per sack*	*Inches*	*Lb. per cu. ft.*	*Percent*
I-1	94:212:241	6.4	6.4	3.2	145.7	1.0
I-2	94:212:241	6.4	6.4	2.9	145.4	1.1
I-3	94:212:241	6.4	6.4	3.3	145.3	1.0
II-1	94:188:300	6.5	5.4	3.2	152.9	1.3
II-2	94:188:300	6.5	5.4	2.9	152.4	1.3
II-3	94:188:300	6.5	5.4	2.8	152.5	1.4
III-1	94:198:272	6.5	5.4	3.0	148.4	0.7
III-2	94:198:272	6.4	5.3	3.0	148.1	0.9
III-3	94:198:272	6.4	5.4	3.0	148.1	0.6
IV-1	94:192:278	6.4	5.5	3.1	146.9	1.1
IV-2	94:192:278	6.4	5.6	3.0	146.5	1.2
IV-3	94:192:278	6.4	5.6	2.9	146.3	1.2
IV-4	94:192:278	6.4	5.6	2.8	146.8	1.1
IV-5	94:192:278	6.4	5.5	3.2	146.7	1.2
V-3	94:215:264	6.5	5.3	2.8	149.3	2.0
V-4	94:215:264	6.5	5.3	3.0	150.0	1.4
V-5	94:215:264	6.5	5.3	3.3	149.5	1.8

[1] Each value is average of three tests.

Table 7.—*Summary of mix data for air-entrained concrete*[1]

Aggregate combination and cement	Proportions of mix, by oven-dry weight	Vinsol-resin added [2]	Cement content	Net water content	Slump	Weight of fresh concrete	Calculated air content
	Pounds	*Percent*	*Sacks per cu. yd.*	*Gal. per sack*	*Inches*	*Lb. per cu. ft.*	*Percent*
I-1	94:198:241	0.0087	6.5	6.0	2.9	142.1	3.9
I-2	94:198:241	.0095	6.4	6.0	3.0	141.3	4.3
I-3	94:198:241	.0112	6.4	6.0	3.0	141.5	4.1
II-1	94:173:300	.0078	6.5	5.0	2.8	149.1	4.3
II-2	94:173:300	.0085	6.5	5.1	3.0	148.2	4.6
II-3	94:173:300	.0103	6.5	5.1	3.0	148.6	4.3
III-1	94:184:272	.0080	6.5	5.0	3.2	145.3	3.3
III-2	94:184:272	.0092	6.5	5.0	2.8	144.7	3.6
III-3	94:184:272	.0105	6.4	5.0	3.0	144.1	3.9
IV-1	94:178:278	.0072	6.5	5.2	3.0	143.3	4.1
IV-2	94:178:278	.0075	6.4	5.2	3.0	142.5	4.4
IV-3	94:178:278	.0098	6.4	5.2	3.1	142.6	4.3
IV-4	94:178:278	.0097	6.4	5.3	3.1	143.1	4.0
IV-5	94:178:278	.0075	6.5	5.0	3.2	143.3	4.3
V-3	94:201:264	.0102	6.5	5.0	3.2	145.5	4.8
V-4	94:201:264	.0098	6.5	5.0	3.2	146.3	4.4
V-5	94:201:264	.0072	6.5	4.8	3.2	145.5	5.2

[1] Each value is average of three tests.
[2] Percentage by weight of cement, added as a sodium hydroxide solution.

rtar to which was added 1 percent of ·ium hydroxide, by weight of cement. T· addition of sodium hydroxide was for l purpose of accelerating the action and ·ather common practice in work of this ·e. The first group was stored at 120° for 3 years, and the second group at 120° for 12 weeks. For each aggregate the ·ansion data are arranged in order of ·ending values of alkali of the cements, ·ressed as sodium oxide.

Jone of the plain mortar bars developed ·ansions which would be indicative of a ·nite alkali-aggregate reaction. How- ·r, the specimens containing the Klamath ·ls basalt with cements 1 and 2 and ·se containing the aggregates from Rock ·and with cements 4 and 5 developed ·newhat higher expansions at 3 years than ·any of the others. The same general ·nds were indicated also in the case of ·specimens containing 1 percent added ·ium hydroxide and stored 12 weeks. ·e results of these tests would classify ·Klamath Falls basalt and the Rock ·and materials as mildly alkali reactive. ·wever, as will be seen later, freezing ·l thawing tests on the concrete would ·icate that the rapid failure of the con- ·te containing the basalt was due pri- ·rily to some other factor.

Test Procedures

The materials used in each of the 17 ·nent-aggregate combinations are shown ·table 5. The Potomac River sand and ·avel were the only aggregates used with ·five cements. In general, each coarse ·gregate was tested with the fine aggre- ·te from the same general locality. As ·eviously stated, the combinations involv- ·g Oregon and Washington materials were ·ected on the basis of recommendations ·G. S. Paxson, Bridge Engineer of Oregon, ·d Bailey Tremper, Materials Engineer ·Washington, respectively; the other com- ·nations were for purposes of comparison. Summaries of the concrete mix data for ·combinations are given in tables 6 and ·The nominal cement content was 6.5 ·cks per cubic yard, and a slump of ap- ·oximately 3 inches was obtained. The ·ater content varied from a low of 4.8 ·llons per sack of cement in the case of ·e air-entrained mixes containing Rock ·land sand and gravel to 6.4 gallons for ·e three non-air-entrained mixes contain- ·g the crushed stone from Klamath Falls. Two initial moist air curing periods at ·° F., of 28 and 180 days, were employed ·ior to starting the freezing and thawing ·sts. In the case of the 28-day curing ·riod, the freezing tests were discontinued ·ter 120 cycles. In the case of the 180- ·ly curing period, they were continued ·rough 270 cycles. All control specimens ·r strength determinations were stored ·ntinuously moist until tested at the same ·e as the companion specimens which were ·bjected to freezing and thawing.

Specimens for the freezing and thawing ·sts were 3- by 4- by 16-inch beams. They were frozen in water at approximately 0° F., followed by thawing in water at 70° F., using a 24-hour cycle. Stainless steel gage points were set in the ends of the specimens during molding so that measurements of length change could be made at periodic intervals.

Deterioration of the specimens was measured by the sonic test.[2] When the average of the three specimens showed a drop in sonic modulus of elasticity of approximately 40 percent, they were tested in flexure and one of each of the broken halves was tested in compression as a modified cube. At the same time the corresponding control specimens were tested in flexure and one of each of the broken halves was tested as a modified cube.

In the sonic test, changes in the elastic properties of the specimen are determined by measuring the change in its natural frequency of vibration. Assuming that the weights and dimensions of the specimens do not change during the test, the decrease in the square of this value, expressed as N^2, may be used in place of the computed modulus of elasticity to indicate deterioration. This common practice is followed in this report. The values for percentage reduction in N^2 are shown in tables 8 and 9.

As a convenience in studying relative durability as measured by the freezing and thawing test, Walker in 1944 proposed the use of a single value called for convenience the "durability factor."[3] This value is a function of the area under the curve which results when the dynamic modulus of elasticity N^2, expressed as a percentage of the value at zero cycles, is plotted in relation to the number of cycles. Wuerpel[4] has suggested a modification of Walker's formula based on the assumption that this curve is a straight line and that the degree of deterioration at a given end point of time or cycles is a more important factor than the shape of the curve.

The modification proposed by Wuerpel has been adopted by the American Society

Table 8.—*Decrease in resistance of concrete to freezing and thawing, after initial moist curing of 28 days, as measured by decrease in N^2*

Aggregate combination and cement	Percentage decrease in N^2 after — cycles of freezing and thawing [1]															Durability factor[2]
	8	16	24	32	40	48	56	64	72	80	88	96	104	112	120	
Non-Air-Entrained Concrete																
I-1	10	22	[3]42	----	----	----	----	----	----	----	----	----	----	----	----	14
I-2	22	41	[4]51	----	----	----	----	----	----	----	----	----	----	----	----	8
I-3	25	38	[4]45	----	----	----	----	----	----	----	----	----	----	----	----	9
II-1	6	6	7	8	9	9	9	10	12	14	15	16	16	13	14	86
II-2	6	6	7	9	12	14	14	14	16	18	20	21	22	20	20	80
II-3	5	6	6	9	10	12	15	16	20	24	26	26	29	31	39	61
III-1	4	3	3	5	5	5	6	7	9	9	9	11	12	9	9	91
III-2	4	4	6	8	9	9	12	12	15	16	16	18	20	17	18	82
III-3	4	4	6	8	12	15	21	23	25	27	29	31	32	33	33	67
IV-1	9	12	16	25	32	37	[5]40	----	----	----	----	----	----	----	----	26
IV-2	11	14	18	26	31	36	[5]40	----	----	----	----	----	----	----	----	26
IV-3	14	23	30	[6]46	----	----	----	----	----	----	----	----	----	----	----	14
IV-4	14	27	[7]40	----	----	----	----	----	----	----	----	----	----	----	----	12
IV-5	10	12	14	18	21	25	30	34	[8]44	----	----	----	----	----	----	35
V-3	5	5	4	5	5	5	5	5	7	9	9	13	17	14	14	86
V-4	6	6	7	8	9	11	12	13	20	22	24	27	32	33	33	67
V-5	8	8	8	9	8	9	9	10	11	12	13	13	15	14	15	85
Air-Entrained Concrete																
I-1	3	3	4	9	10	12	12	13	17	17	16	17	17	16	14	86
I-2	2	5	6	10	15	20	23	27	30	31	32	43	44	44	44	47
I-3	1	1	2	2	4	6	8	9	12	12	10	13	11	11	9	91
II-1	2	3	3	3	3	4	4	5	7	6	6	5	5	4	2	98
II-2	2	4	5	4	4	5	6	6	9	10	12	11	10	9	7	93
II-3	2	2	3	2	2	4	6	4	6	6	6	8	5	5	4	96
III-1	0	1	3	0	1	2	2	3	3	2	3	3	3	2	+1	100
III-2	1	2	3	2	3	3	4	6	7	7	7	7	7	7	6	94
III-3	0	0	0	+1	+1	0	+1	0	0	0	+1	1	+1	+1	3	97
IV-1	5	5	6	4	5	6	5	6	6	6	5	6	6	3	4	96
IV-2	5	5	6	5	7	8	8	11	11	12	12	14	13	11	10	90
IV-3	4	5	4	3	2	4	3	5	6	7	6	8	6	6	3	97
IV-4	4	4	5	4	5	5	5	6	7	7	7	7	7	5	4	96
IV-5	5	5	5	4	3	6	4	6	7	7	5	5	5	4	2	98
V-3	3	3	2	1	0	2	1	2	2	2	2	1	1	2	1	99
V-4	3	3	3	3	3	3	3	5	5	6	5	4	4	3	1	99
V-5	4	5	5	4	3	4	4	6	5	6	4	4	4	2	1	99

[1] Each value is the average of tests on three beams except for II-2 in the non-air-entrained group, and I-3 and II-3 in the air-entrained group, for which two beams were tested, and III-1 in the air-entrained group, for which one beam was tested.
[2] Method of determination similar to that described in Tests for Air-Entraining Admixtures for Concrete, A.S.T.M. designation C 233-49T.
[3] Final reading at 29 cycles. [4] Final reading at 21 cycles. [5] Final reading at 53 cycles.
[6] Final reading at 32 cycles. [7] Final reading at 24 cycles. [8] Final reading at 75 cycles.

[2]*Application of sonic method to freezing and thawing studies of concrete*, by F. B. Hornibrook. American Society for Testing Materials, Bulletin No. 101, December 1939, p. 5.

[3]*Freezing and thawing tests of concrete made with different aggregates*, by Stanton Walker. Journal of the American Concrete Institute, June 1944, p. 573.

[4]*Discussion* by C. E. Wuerpel of Walker's paper (see footnote 3). Journal of the American Concrete Institute, November 1944, p. 580.

Table 9.—*Decrease in resistance of concrete to freezing and thawing, after initial moist curing of 180 days, as measured by decrease in N^2*

Aggregate combination and cement	Percentage decrease in N^2 after—cycles of freezing and thawing[1]														Durability factor[2]	
	20	40	60	80	100	120	140	160	180	200	220	240	250	270	At 120 cycles	At 270 cycles
Non-Air-Entrained Concrete																
I-1	4	8	9	10	15	19	26	31	37	[3]40	—	—	—	—	81	43
I-2	4	7	8	12	26	28	34	35	39	[3]40	—	—	—	—	72	43
I-3	4	13	15	19	24	27	30	34	35	35	37	37	[4]49	—	73	54
II-1	4	5	4	5	6	6	6	7	5	6	6	5	6	7	94	93
II-2	4	6	4	6	6	6	6	8	6	7	6	5	7	7	94	93
II-3	4	5	4	5	5	5	6	6	6	5	5	4	5	5	95	95
III-1	3	5	4	4	6	5	6	7	6	8	7	6	7	8	95	92
III-2	2	6	4	4	6	6	6	8	6	6	8	7	8	8	94	92
III-3	2	5	4	4	6	6	6	7	7	8	7	7	7	8	94	92
IV-1	12	18	21	27	34	[5]42	—	—	—	—	—	—	—	—	58	26
IV-2	14	20	20	21	25	26	32	35	36	[3]40	—	—	—	—	74	43
IV-3	16	21	24	29	33	35	[6]40	—	—	—	—	—	—	—	65	33
IV-4	16	20	24	28	32	38	[7]40	—	—	—	—	—	—	—	62	29
IV-5	10	12	12	14	16	19	23	28	28	32	37	39	[4]40	—	81	57
V-3	4	6	4	5	6	5	6	6	5	9	10	8	9	12	95	88
V-4	8	10	10	13	15	18	20	24	26	27	28	29	30	35	82	65
V-5	9	12	12	14	18	20	24	26	30	32	[8]40	—	—	—	80	49
Air-Entrained Concrete																
I-1	4	4	5	6	8	6	8	8	6	6	20	19	16	22	94	78
I-2	6	7	8	9	12	12	13	14	12	12	12	14	13	16	88	84
I-3	6	5	6	5	7	7	7	8	6	7	6	9	7	10	93	90
II-1	5	5	8	8	9	8	8	9	10	10	9	12	10	13	92	87
II-2	6	6	8	9	11	10	10	11	11	17	18	20	18	29	90	71
II-3	6	5	6	5	7	5	6	6	6	5	6	8	6	8	95	92
III-1	4	4	4	4	6	6	6	7	8	8	6	8	8	11	94	89
III-2	4	4	4	3	5	3	4	5	5	3	10	14	14	19	97	81
III-3	4	3	4	4	6	4	6	7	4	5	5	7	6	8	96	92
IV-1	10	11	11	11	12	12	14	13	14	13	14	16	16	22	88	78
IV-2	12	13	12	13	12	12	14	13	12	11	13	14	13	17	88	83
IV-3	8	8	8	9	10	9	9	8	8	7	8	10	9	13	91	87
IV-4	11	10	10	10	12	14	14	15	14	14	14	16	15	20	86	80
IV-5	10	10	10	9	10	11	10	12	11	10	10	13	12	16	89	84
V-3	6	7	6	5	8	7	8	7	7	6	6	8	8	9	93	91
V-4	9	10	8	12	14	14	16	15	16	15	16	20	19	23	86	77
V-5	9	10	19	18	21	21	20	23	20	22	25	27	27	30	79	70

[1] Each value is the average of tests on three beams except for I-1 and II-1 in the air-entrained group, for which two beams were tested.
[2] Method of determination similar to that described in Tests for Air-Entraining Admixtures for Concrete. A.S.T.M. designation C 233-49T.
[3] Final reading at 192 cycles. [4] Final reading at 256 cycles. [5] Final reading at 123 cycles.
[6] Final reading at 147 cycles. [7] Final reading at 131 cycles. [8] Final reading at 220 cycles.

for Testing Materials.[5] The durability factors employed in this paper and used in plotting figures 1–5 were computed from the formula given below, which is similar to that given in the A.S.T.M. publication referred to (the only difference is in the selection of 60 percent of the relative dynamic E as the end point for the freezing and thawing test):

$$DF = \frac{PN}{M}$$

where:

DF=durability factor of the concrete.

P=relative dynamic modulus of elasticity N^2 in percentage of the dynamic modulus of elasticity at zero cycles (values of P will be 60 or greater).

M=number of cycles at which durability factor is calculated.

N=number of cycles at which P equals 60 percent: If P has not reached 60 percent at the number of cycles M at which the durability factor is calculated, then $N=M$ and $DF=P$.

For the purpose of this discussion it is assumed that a durability factor DF of 75 represents the line of demarcation between durable and nondurable concrete. The durability factors for the various aggregate combinations and cements are shown in bar diagram form in figures 1–5. In each case, values are shown for both the non-air-entrained and the air-entrained concrete at the end of 120 cycles for specimens cured 28 days and at the end of 120 and 270 cycles for specimens cured 180 days.

[5] American Society for Testing Materials, Designation C 233-49T, Book of A.S.T.M. Standards, Part 3, 1949, page 821.

Effects of Freezing and Thawing

Values for aggregate combinations I, the Mt. Hebron sand and Klamath Falls coarse aggregate, with cements 1, 2, and 3 are shown in figure 1. It will be noted that with all three cements, the non-air-entrained concrete containing this combination had very low resistance to freezing and thawing after 28 days of curing. Continuous moist curing to 180 days markedly improved resistance with all cements. Even with the additional curing, however, the durability factors at 120 cycles exceeded 75 in only one case (cement 1). When the 180-day cured specimens were subjected to 270 cycles of freezing and thawing, the durability factors were considerably reduced as compared to the values at 120 cycles, a further indication of the lack of durability of this aggregate combination.

The fact that the long curing period was of definite assistance in increasing durability would indicate the absence of any noticeable reaction between the aggregate and the alkalies in the cements in spite of the fact that the results of mortar-bar expansion tests with Klamath Falls basalt and cements 1 and 2 would indicate the possibility of a mild reaction (see table). The limited quantities of the aggregates available were insufficient for making tests to determine the contributing effect of the Mt. Hebron and the Klamath Falls materials separately to the poor freezing and thawing resistance encountered. Variations in the alkali content of the cements used with this aggregate combination did not appear to have much effect on resistance to freezing and thawing. Possibly any effect was masked by the low inherent resistance of the aggregate. These tests indicate that the combination of the Mt. Hebron sand and Klamath Falls coarse aggregate in non-air-entrained concrete is unsatisfactory for use in concrete for outdoor exposure where alternate freezing and thawing may occur.

The air-entrained concrete containing this aggregate combination showed, in all cases, greatly improved resistance as compared to the corresponding non-air-entrained concrete. However, the relative increases were much greater for the specimens cured 28 days than when the curing was extended to 180 days. Extending the freezing and thawing of the 180-day cured specimens to 270 cycles decreased the resistance in all cases, although the relative decreases were not nearly so great as for the corresponding concretes without entrained air. The durability factors of all of the air-entrained concrete specimens containing this aggregate combination exceeded 75 in all cases except one (with cement 2, cured 28 days).

From figure 2 it can be seen that aggregate combination II, the Umatilla sand and gravel, with and without air entrainment gave durability factors greater than 75 with two exceptions. These were the combinations with cement 2 in air-entrained concrete, cured 180 days and exposed to 270 cycles, and cement 3 without air entrainment, cured 28 days and exposed to 120 cycles. In both cases the durability factor was in excess of 60. These tests would indicate that the Umatilla sand and gravel are high-grade aggregates suitable for making concrete exposed to severe climatic conditions.

Durability factors for aggregate combination III, the sand and gravel from the Willamette River, are shown in figure 3. The similarity between figures 2 and 3 is immediately apparent, indicating that there is no practical difference in the concrete-making properties of the Umatilla and Willamette aggregates, at least insofar as durability is concerned. It is interesting to note, when comparing figures 2 and 3, that in the case of the non-air-entrained

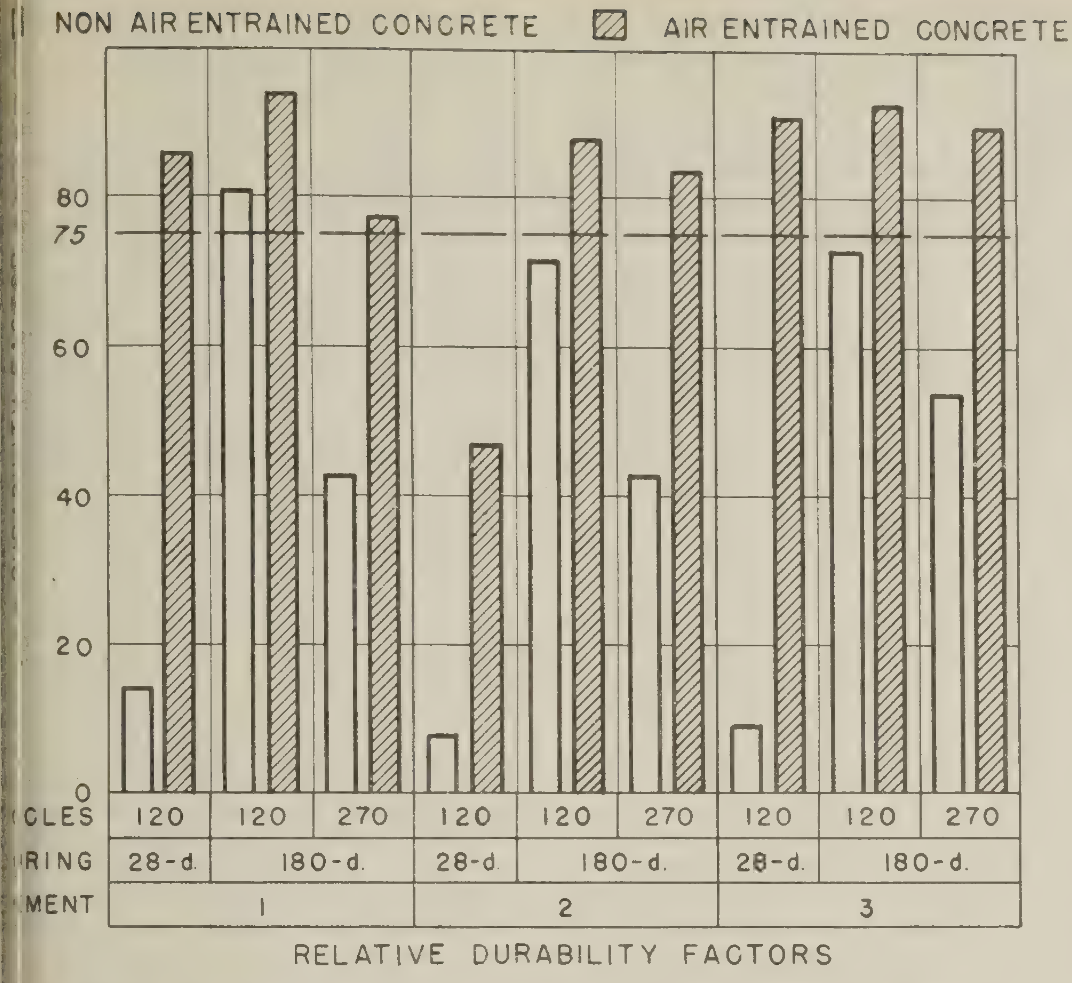

Figure 1.—Durability of aggregate combination I with cements 1, 2, and 3.

firming the opinion that there is little possibility of reactivity even with high alkali cements.

It is of interest to note that increasing the curing period for air-entrained concrete from 28 to 180 days decreased the resistance slightly with all five cements. This suggests the possibility that, after long moist curing periods, some of the minute voids created by air entrainment may eventually fill with water. With minor exceptions, this same trend was noted also in all of the other aggregate combinations except combination I.

Aggregate combination V, the Rock Island sand and gravel, was from the eastern section of the State of Washington and is the same as the aggregate used in the tests by Tremper to which reference has been made. He found that this aggregate in concrete, when used with cements high in alkali and given prolonged moist curing prior to testing, broke down quite rapidly in freezing and thawing. Similar concretes containing the same aggregate but cements of lower alkali content gave fair resistance to the action of freezing and thawing.

As may be seen from figure 5, the Rock Island aggregates in combination with cement 5, having a total alkali content of 0.66 percent (expressed as sodium oxide), showed somewhat lower resistance at the end of 180 days of moist storage than was obtained with cement 4 (0.44 percent al-

crete, cured 28 days, the order of resisce as affected by the cement is the ne—concrete containing cement 1 being most resistant and concrete containing nent 3 the least resistant. However, this ation does not hold for the longer curing iod and for this reason it may not be nificant, insofar as ultimate durability concerned.

As will be noted from figure 4, non-airrained concrete made with aggregate nbination IV, Potomac River sand and avel, developed poor resistance to freezing l thawing with all five cements. This gregate gives good service in the relaely mild climate in the Washington, D. C., a, but these tests indicate that without entrainment it would not be a suitable gregate where freezing and thawing is ere. Potomac River aggregate has a le range in mineral composition and it possible that occasionally it may contain ne reactive particles, but there is little no direct evidence of reactivity. The nparatively poor resistance of concrete de with this aggregate is probably due a combination of low bond strength and tain structural weaknesses of the aggree particles. The resistance of the non--entrained concrete at 120 cycles was atly improved by extending the curing riod to 180 days. Resistance was also atly benefited when air-entrainment was ed, more so than was true for the contes made with other aggregates, con-

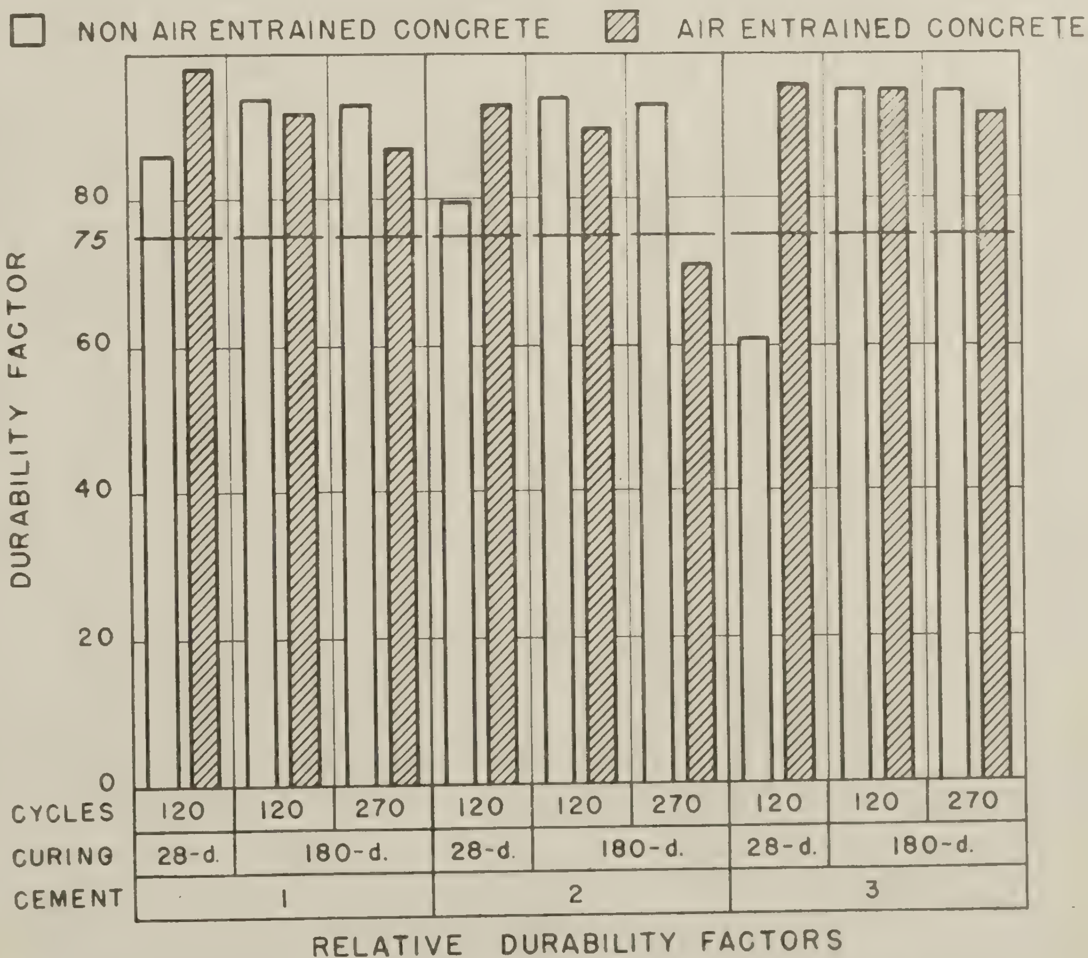

Figure 2.—Durability of aggregate combination II with cements 1. 2. and 3.

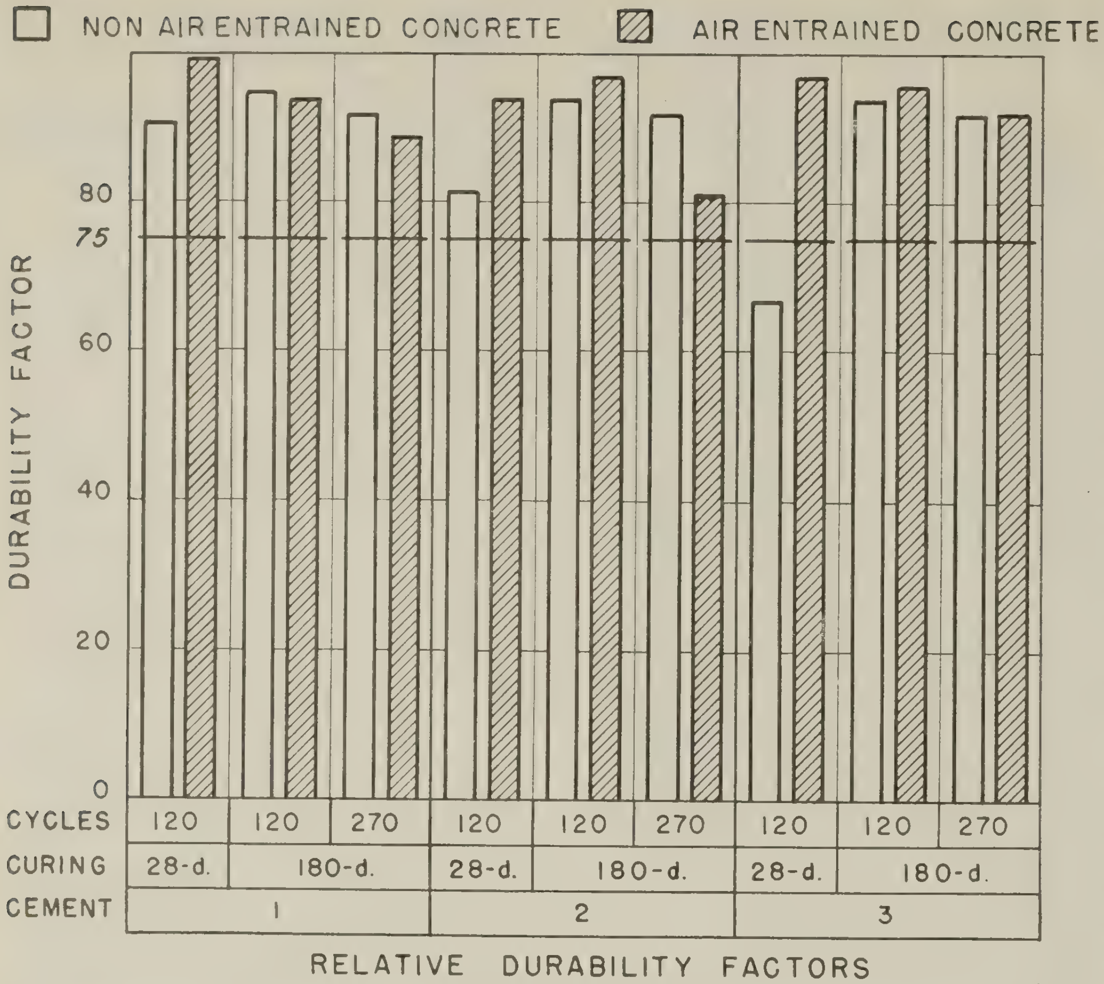

Figure 3.—Durability of aggregate combination III with cements 1, 2, and 3.

kali), and substantially lower resist than was obtained with cement 3 percent alkali). This is particularly nc able at the end of 270 cycles of free and thawing. On the other hand, the sults at the end of 28 days of moist sto were about the same for cement 5 (alkali) as for cement 3 (low alkali). T results would seem to verify Trem conclusion that the Rock Island aggre is slowly reactive with alkali in the cen The non-air-entrained Rock Island conc with cement 5 are the only ones which a 180 days of moist curing were less sistant than similar concrete cured 28 c This would also seem to verify Trem conclusion. Except for this tendenc be slowly reactive, the Rock Island ag gate in combination with both high low alkali cements had fairly good r tance to alternate freezing and thawir

In the field where reactive aggreg were suspected, there has been evidenc map cracking of the concrete pavem Several observers have thought that t cracks permitted easy entrance of w hence making the concrete more suscep to freezing and thawing action. This ductive reasoning may be correct, but difficult to simulate this condition in laboratory. Careful observation of the specimens failed to disclose evidenc cracking, but this in itself is not signifi since small specimens of the size under sideration permit adjustment of the stre

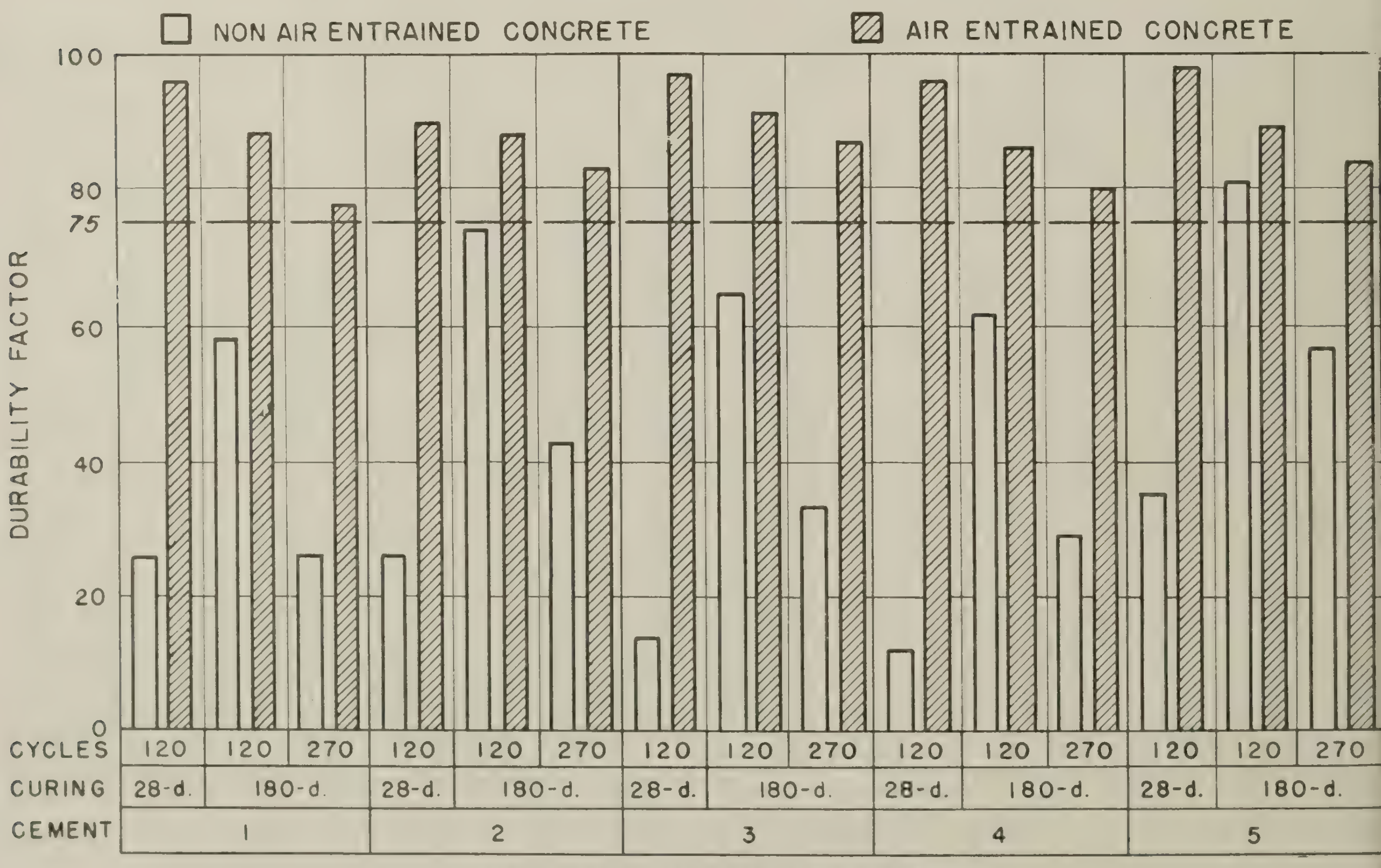

Figure 4.—Durability of aggregate combination IV with cements 1, 2, 3, 4, and 5.

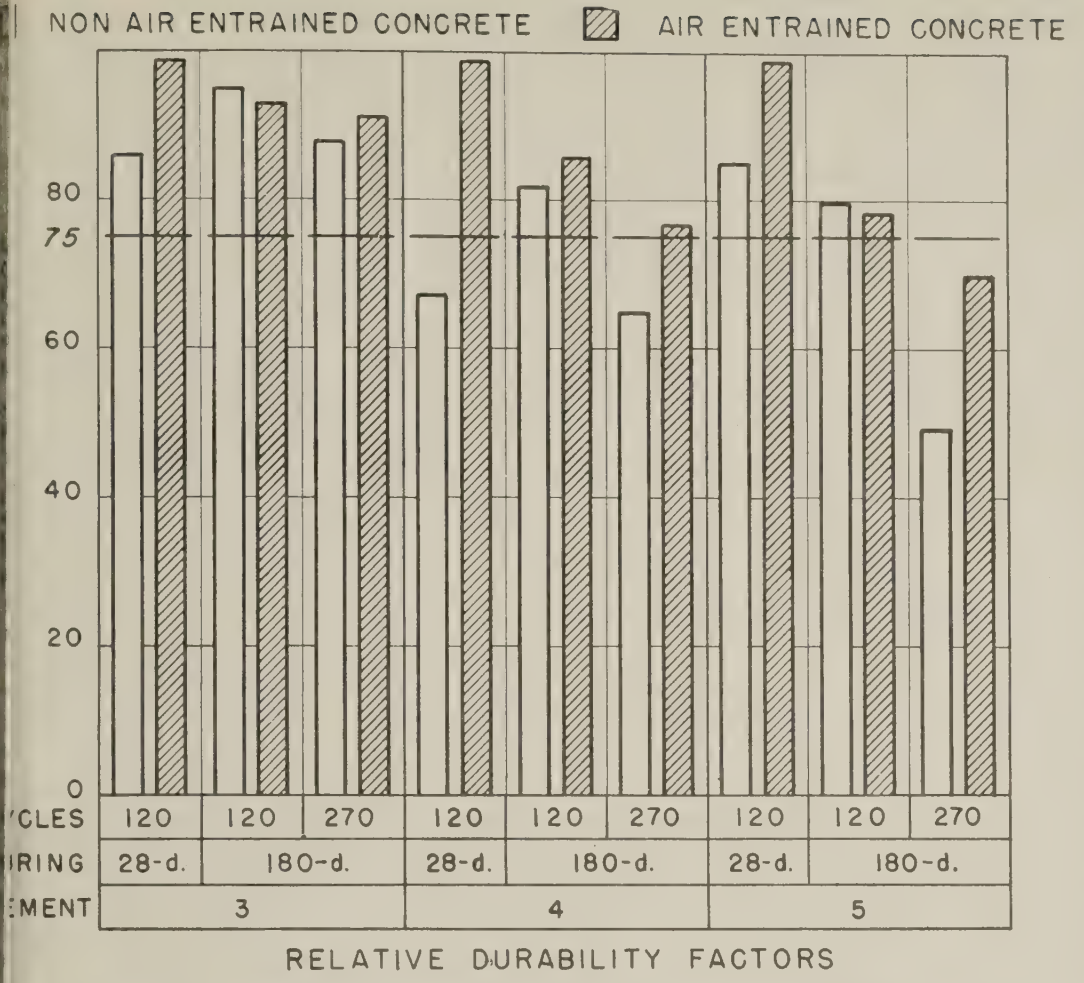

Figure 5.—Durability of aggregate combination V with cements 3, 4, and 5.

that cracking occurs only under extreme nditions of volume change.

Length change measurements, shown in bles 10 and 11, were made on all the contes subjected to freezing and thawing, t no significant information was developed at was not brought out by study of the rability factor.

Strength Tests

Tables 12 and 13 show the flexural engths and the compressive strengths of dified cubes from the beams remaining ter the freezing and thawing test was mplete, as well as from control beams. is customary to make such tests to conm the results indicated by the drop in nic E. For the non-air-entrained conete, however, no direct comparisons can made of the strengths of the concrete ade of the various combinations of aggretes and cements, because of the variable mber of cycles required to produce a 40-rcent drop in sonic E and the consequent fference in age at time of test, and also cause of the added curing obtained by the ntrol specimens in moist air at 70° F. hile the freezing tests were in progress. The frozen specimens do, of course, obtain some additional curing during the thawing phase of the cycle, but these temperatures are not favorable for promoting the curing action. This difference in curing is brought out by the observation that even though some of the frozen air-entrained concrete specimens showed little or no drop in N^2, a reduction of 10 to 15 percent in compressive strength of these same specimens was noted with an even greater reduction in the case of flexural strength.

Comparison of Durability Factors

To compensate for the variable number of cycles at which some of the specimens were tested, compressive and flexural strength durability factors somewhat similar to the durability factor calculated from sonic E were evolved. These factors were calculated by taking the ratio of the strength after freezing and thawing to the corresponding strength of the control specimens, multiplying this ratio by the number of cycles at time of test, and dividing the product by 120 in the case of the 28-day cured specimens or by 270 in the case of the 180-day cured specimens.

In table 14 the durability factors based on sonic determinations, on compressive strength loss, and on flexural strength loss have all been grouped for purposes of comparison. The durability factors for strength of specimens which withstood the maximum number of cycles are, of course, the same numerical values as the strength ratios shown in tables 10 and 11.

A comparison of the three durability factors indicates a reasonably good correlation as regards ability to differentiate between aggregate-cement combinations having poor resistance as measured by the sonic test and those combinations showing good resistance. For example, in all of the non-air-entrained concretes containing aggregate combinations I and IV, with all cements, the sonic test indicates low resistance (less than 75). For all of these combinations the corresponding factors calculated from the flexure test are also comparatively low (less than 50 in all cases). The corresponding compressive strength factors are also, with two exceptions, comparatively low (less than 70). It will be seen, therefore, that from the standpoint of selecting the definitely poor combinations any of the three factors may be used. The same general relations also hold for the three combinations which in general had good resistance as measured by sonic E. Six of the non-air-entrained combinations showed sonic E values of 75 or more at 120 cycles and seven combinations showed correspondingly high values at 270 cycles. For most of these the corresponding durability values in compression and flexure were also relatively high, although there were some individual exceptions to the trend as, for example, the comparatively low value for aggregate combination II with cement 2 in flexure at 120 cycles as compared to the sonic E and compression values.

It may be concluded that the reduction in sonic E reflects reasonably well the effects of freezing and thawing on both the compressive and flexural strength of the concrete, particularly in those cases where the resistance of the concrete is comparatively low.

It will be noted that the durability factor based on flexure strength is considerably lower than that based on compressive strength. Alternate freezing and thawing is probably more destructive to the outermost fibers than to the specimen as a whole. Since the flexural test is a measure of the bending strength of the outermost fibers it appears reasonable to expect lower relative strengths than indicated by the compression tests on the broken halves remaining after the flexure test.

(Tables 10–14 are on the following pages.)

Table 10.—*Decrease in resistance of concrete to freezing and thawing, after initial moist curing of 28 days, as measured by increase in length*[1]

Aggregate combination and cement	Percentage increase in length after—cycles of freezing and thawing (in thousandths of 1 percent)														
	8	16	24	32	40	48	56	64	72	80	88	96	104	112	120
Non-Air-Entrained Concrete															
I-1	8	23	42	--	--	--	--	--	--	--	--	--	--	--	--
I-2	14	42	59	--	--	--	--	--	--	--	--	--	--	--	--
I-3	17	39	48	--	--	--	--	--	--	--	--	--	--	--	--
II-1	5	6	6	7	9	10	11	13	16	17	19	18	20	20	24
II-2	5	6	5	9	10	12	15	15	16	18	24	24	25	26	27
II-3	3	5	3	7	11	13	17	21	28	32	36	38	39	41	48
III-1	4	4	5	6	6	7	8	12	13	14	16	17	17	16	20
III-2	1	5	6	7	9	10	12	15	17	17	21	24	22	22	24
III-3	3	4	2	6	10	12	19	24	31	33	37	39	41	44	46
IV-1	9	18	24	40	54	70	--	--	--	--	--	--	--	--	--
IV-2	9	14	20	30	38	49	--	--	--	--	--	--	--	--	--
IV-3	9	25	39	--	--	--	--	--	--	--	--	--	--	--	--
IV-4	13	37	58	--	--	--	--	--	--	--	--	--	--	--	--
IV-5	7	13	15	24	32	42	54	67	81	--	--	--	--	--	--
V-3	4	6	3	8	8	10	13	14	19	20	22	23	26	28	31
V-4	6	7	6	10	12	16	17	21	29	33	36	40	45	48	52
V-5	1	4	3	3	4	7	9	10	15	18	18	19	22	24	27
Air-Entrained Concrete															
I-1	4	4	6	7	8	9	11	13	15	15	15	16	17	18	18
I-2	2	3	8	7	9	12	15	18	23	22	23	25	27	29	31
I-3	3	5	6	9	8	10	12	14	16	17	18	18	20	20	22
II-1	4	2	4	4	4	4	7	6	11	10	9	10	11	11	11
II-2	1	1	3	3	4	6	7	7	10	10	9	10	9	11	11
II-3	2	5	3	4	4	6	9	9	9	11	10	12	11	11	10
[2]III-1	1	2	5	1	2	−1	4	8	9	8	8	8	9	11	11
III-2	1	2	5	3	4	3	7	7	10	9	7	9	8	9	9
III-3	1	3	4	1	4	4	7	5	9	7	7	8	7	9	8
IV-1	4	3	5	4	5	7	10	11	14	14	12	15	14	18	16
IV-2	3	5	7	6	9	7	11	11	15	16	16	18	18	18	17
IV-3	4	3	5	4	4	7	8	10	12	11	12	14	14	15	13
IV-4	3	3	5	5	5	8	9	11	14	13	12	12	14	15	16
IV-5	3	4	7	5	7	8	10	11	13	12	13	13	14	17	17
V-3	2	2	5	2	4	6	6	7	9	9	8	8	10	10	8
V-4	3	2	4	5	5	4	7	8	10	10	8	10	9	10	10
V-5	2	5	7	7	5	8	9	10	13	13	12	12	14	16	17

[1] Each value is average of tests on three beams except as indicated.
[2] Average of two tests only.

Table 11.—*Decrease in resistance of concrete to freezing and thawing, after initial moist curing of 180 days, as measured by increase in length*[1]

Aggregate combination and cement	Percentage increase in length after — cycles of freezing and thawing (in thousandths of 1 percent)													
	20	40	60	80	100	120	140	160	180	200	220	240	250	270
Non-Air-Entrained Concrete														
I-1	3	4	6	8	12	19	24	33	42	--	--	--	--	--
I-2	3	4	5	6	14	17	20	24	30	--	--	--	--	--
I-3	2	4	4	6	9	11	12	16	18	19	21	23	25	--
II-1	2	3	4	3	6	7	8	9	10	11	15	14	12	15
II-2	2	2	1	2	3	4	4	7	7	7	8	8	9	11
II-3	2	3	4	2	4	5	5	7	8	8	10	10	11	14
III-1	1	2	2	1	4	4	4	6	8	7	10	10	11	12
III-2	2	3	2	4	4	4	5	6	6	7	8	7	7	9
[2]III-3	2	2	2	3	2	3	4	4	6	7	8	8	10	11
IV-1	9	15	23	29	41	53	--	--	--	--	--	--	--	--
IV-2	12	16	20	22	31	40	50	60	70	--	--	--	--	--
IV-3	12	21	28	38	50	62	77	--	--	--	--	--	--	--
IV-4	11	18	22	31	42	52	--	--	--	--	--	--	--	--
IV-5	9	13	16	22	29	47	45	57	68	76	88	101	105	--
V-3	2	3	2	4	6	5	6	9	10	12	13	15	17	19
V-4	4	7	10	12	16	20	24	31	32	38	46	50	53	61
V-5	8	10	14	18	22	28	33	38	45	48	59	65	--	--
Air-Entrained Concrete														
[2]I-1	0	0	2	2	2	1	2	5	5	2	9	8	8	6
I-2	0	1	2	3	3	3	6	8	5	6	7	8	8	7
I-3	0	0	0	2	0	1	4	5	2	0	3	2	5	5
[2]II-1	0	0	2	2	3	2	5	10	7	9	8	7	10	12
II-2	1	2	2	3	3	3	5	8	6	7	9	8	9	10
II-3	2	2	2	3	3	3	6	8	8	5	6	8	7	6
III-1	0	1	1	1	2	1	4	7	6	5	7	8	11	13
[2]III-2	0	1	0	1	0	−1	2	4	8	7	7	7	7	8
III-3	0	0	0	1	2	0	4	5	2	2	2	1	2	2
IV-1	5	8	10	14	14	16	20	26	25	25	29	31	34	38
IV-2	5	6	8	11	10	13	17	21	19	20	25	27	28	30
IV-3	2	1	2	4	5	7	9	12	12	11	15	15	16	18
IV-4	3	3	4	7	6	8	10	15	14	13	17	19	19	18
IV-5	6	7	10	13	14	17	22	27	28	30	32	36	38	40
V-3	2	1	1	3	2	3	6	8	7	7	8	9	9	11
V-4	2	3	4	6	6	8	10	14	13	13	16	18	19	20
V-5	4	5	5	10	12	12	14	20	21	20	26	29	30	38

[1] Each value is average of tests on three beams except as indicated.
[2] Average of two tests only.

Table 12.—*Results of strength tests[1] after initial moist curing of 28 days and maximum of 120 cycles of freezing and thawing*

Aggregate combination and cement	Total age at test	Strength of control specimens		Average cycles of freezing and thawing at test	Strength of specimens after freezing and thawing		Strength ratio[4]	
		Compressive[2]	Flexural[3]		Compressive[2]	Flexural[3]	Compressive	Flexural
Non-Air-Entrained Concrete								
	Days	Lb./sq. in.	Lb./sq. in.		Lb./sq. in.	Lb./sq. in.	Percent	Percent
I-1	58	5,900	725	24	4,790	395	81	54
I-2	55	5,830	775	21	5,030	350	86	45
I-3	55	5,540	725	21	4,810	330	87	46
II-1	187	7,730	950	120	6,390	620	83	65
II-2	187	7,910	965	120	7,170	410	91	42
II-3	187	7,790	1,005	120	6,540	515	84	51
III-1	187	7,250	970	120	6,020	590	83	61
III-2	187	7,380	910	120	6,120	640	83	70
III-3	187	7,570	955	120	5,640	475	75	50
IV-1	92	6,780	810	53	5,120	405	76	51
IV-2	92	7,080	800	53	5,440	[5]420	70	52
IV-3	67	6,620	820	32	5,300	355	80	43
IV-4	58	7,270	865	24	5,590	420	77	49
IV-5	118	6,720	860	75	5,120	350	76	41
V-3	187	7,950	925	120	6,280	620	79	67
V-4	187	8,190	1,050	120	6,730	415	82	40
V-5	187	7,180	910	120	6,140	495	86	54
Air-Entrained Concrete								
I-1	187	6,120	785	120	5,270	550	86	70
I-2	187	6,310	820	120	5,010	370	79	45
I-3	187	6,150	835	120	5,320	415	87	50
II-1	187	7,060	945	120	6,450	685	91	72
II-2	187	6,570	870	120	5,880	505	89	58
II-3	187	7,240	980	120	6,240	640	86	65
III-1	187	6,070	860	120	5,510	[6]750	91	87
III-2	187	6,460	880	120	5,500	675	85	77
III-3	187	6,540	900	120	5,900	720	90	80
IV-1	187	6,390	880	120	5,640	635	88	72
IV-2	187	6,330	815	120	5,430	580	86	71
IV-3	187	6,670	785	120	5,630	640	84	82
IV-4	187	6,200	860	120	5,870	655	95	80
IV-5	187	6,610	825	120	5,690	575	86	70
V-3	187	7,010	970	120	6,280	730	90	75
V-4	187	7,060	860	120	6,430	695	91	81
V-5	187	6,640	[5]965	120	6,040	650	91	67

[1] Each value is average of three tests except as indicated.
[2] Tested as modified cubes, specimens 3- by 3- by 4-inch depth (halves of flexure specimens remaining after tests).
[3] Specimens 3- by 4- by 16-inch, tested by center-point loading on a 12-inch span with the bottom (4-inch depth) as molded in tension.
[4] Values indicate percentage of strength of corresponding control specimen.
[5] Average of two tests.
[6] One test.

Table 13.—*Results of strength tests[1] after initial moist curing of 180 days and maximum of 270 cycles of freezing and thawing*

Aggregate combination and cement	Total age at test	Strength of control specimens		Average cycles of freezing and thawing at test	Strength of specimens after freezing and thawing		Strength ratio[4]	
		Compressive[2]	Flexural[3]		Compressive[2]	Flexural[3]	Compressive	Flexural
Non-Air-Entrained Concrete								
	Days	Lb./sq. in.	Lb./sq. in.		Lb./sq. in.	Lb./sq. in.	Percent	Percent
I-1	498	6,820	930	192	5,720	450	84	48
I-2	498	7,340	860	192	6,750	395	92	46
I-3	600	7,340	970	256	5,910	315	81	32
II-1	622	8,090	1,035	270	7,620	870	94	84
II-2	622	7,890	1,015	270	7,550	740	96	73
II-3	622	8,580	1,060	270	7,440	715	87	67
III-1	622	7,380	1,000	270	6,950	725	94	72
III-2	622	7,450	990	270	6,780	775	91	78
III-3	622	7,920	980	270	7,150	750	90	77
IV-1	400	7,050	865	123	5,750	395	82	46
IV-2	498	7,060	790	192	5,510	500	78	63
IV-3	434	8,130	900	147	5,190	445	64	49
IV-4	412	8,650	940	131	6,760	445	78	47
IV-5	600	6,850	885	256	5,620	395	82	45
V-3	622	8,320	915	270	7,520	660	90	72
V-4	622	8,340	1,010	270	6,470	440	78	44
V-5	572	7,030	820	220	5,940	360	84	44
Air-Entrained Concrete								
I-1	622	6,650	875	270	6,080	[5]520	91	59
I-2	622	7,200	920	270	6,070	600	84	65
I-3	622	7,090	945	270	6,200	690	87	73
II-1	622	7,840	1,050	270	6,740	[5]540	86	51
II-2	622	7,370	930	270	6,670	530	91	57
II-3	622	7,980	1,020	270	7,300	590	91	58
III-1	622	6,350	965	270	5,970	655	94	68
III-2	622	7,560	1,010	270	6,470	550	86	54
III-3	622	6,900	[5]1,060	270	5,840	700	85	66
IV-1	622	7,150	915	270	6,100	550	85	60
IV-2	622	7,240	850	270	6,150	570	85	67
IV-3	622	7,310	790	270	6,360	575	87	65
IV-4	622	6,960	905	270	6,560	730	94	81
IV-5	622	7,130	[5]895	270	6,010	570	84	64
V-3	622	7,500	990	270	6,860	710	91	72
V-4	622	7,900	925	270	6,730	560	85	61
V-5	622	7,350	880	270	5,840	455	79	52

[1] Each value is average of three tests except as indicated.
[2] Tested as modified cubes, specimens 3- by 3- by 4-inch depth (halves of flexure specimens remaining after tests).
[3] Specimens 3- by 4- by 16-inch, tested by center-point loading on a 12-inch span with the bottom (4-inch depth) as molded in tension.
[4] Values indicate percentage of strength of corresponding control specimen.
[5] Average of two tests.

Table 14.—*Comparison of durability factors*

Aggregate combination and cement	Durability factor of specimens after—					
	28-day curing and maximum of 120 cycles of freezing and thawing			180-day curing and maximum of 270 cycles of freezing and thawing		
	Sonic	Compressive	Flexural	Sonic	Compressive	Flexural
NON-AIR-ENTRAINED CONCRETE						
I-1	14	16	11	43	60	34
I-2	8	15	8	43	65	33
I-3	9	15	8	54	77	30
II-1	86	83	65	93	94	84
II-2	80	91	42	93	96	73
II-3	61	84	51	95	87	67
III-1	91	83	61	92	94	72
III-2	82	83	70	92	91	78
III-3	67	75	50	92	90	77
IV-1	26	34	23	26	37	21
IV-2	26	31	23	43	55	45
IV-3	14	21	11	33	35	27
IV-4	12	15	10	29	38	23
IV-5	35	48	26	57	78	43
V-3	86	79	67	88	90	72
V-4	67	82	40	65	78	44
V-5	85	86	54	49	69	36
AIR-ENTRAINED CONCRETE						
I-1	86	86	70	78	91	59
I-2	47	79	45	84	84	65
I-3	91	87	50	90	87	73
II-1	98	91	72	87	86	51
II-2	93	89	58	71	91	57
II-3	96	86	65	92	91	58
III-1	100	91	87	89	94	68
III-2	94	85	77	81	86	54
III-3	97	90	80	92	85	66
IV-1	96	88	72	78	85	60
IV-2	90	86	71	83	85	67
IV-3	97	84	82	87	87	65
IV-4	96	95	80	80	94	81
IV-5	98	86	70	84	84	64
V-3	99	90	75	91	91	72
V-4	99	91	81	77	85	61
V-5	99	91	67	70	79	52

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Annual Report, Bureau of Public Roads, 1950. 25 cents.

HOUSE DOCUMENT NO. 462

Part 1 . . . Nonuniformity of State Motor-Vehicle Traffic Laws. 15 cents.
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Part 5 . . . Case Histories of Fatal Highway Accidents. 10 cents.
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Act I.—Uniform Motor-Vehicle Administration, Registration, Certificate of Title, and Antitheft Act. 10 cents.
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Construction of Private Driveways (No. 272MP). 10 cents.
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Factual Discussion of Motortruck Operation, Regulation, and Taxation. 30 cents.
Federal Legislation and Regulations Relating to Highway Construction. 40 cents.
Financing of Highways by Counties and Local Rural Governments, 1931–41. 45 cents.
Guides to Traffic Safety. 10 cents.
Highway Accidents. 10 cents.
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Highway Needs of the National Defense (House Document No. 249). 50 cents.
Highway Practice in the United States of America. 50 cents.
Highway Statistics, 1945. 35 cents
Highway Statistics, 1946. 50 cents.
Highway Statistics, 1947. 45 cents.
Highway Statistics, 1948. 65 cents.
Highway Statistics, 1949. 55 cents.
Highway Statistics, Summary to 1945. 40 cents.
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Identification of Rock Types. 10 cents.
Interregional Highways (House Document No. 379). 75 cents.
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Manual on Uniform Traffic Control Devices for Streets and Highways. 75 cents.
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Public Land Acquisition for Highway Purposes. 10 cents.
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ANNUAL REPORTS

(See also adjacent column)

Public Roads Administration Annual Reports:
1943. 1944. 1945.

MISCELLANEOUS PUBLICATIONS

Bibliography on Automobile Parking in the United States.
Bibliography on Highway Lighting.
Bibliography on Highway Safety.
Bibliography on Land Acquisition for Public Roads.
Bibliography on Roadside Control.
Express Highways in the United States: a Bibliography.
Indexes to PUBLIC ROADS, volumes 17–19, 22, and 23.
Road Work on Farm Outlets Needs Skill and Right Equipment.
Title sheets for PUBLIC ROADS, volumes 24 and 25.

U.S. GOVERNMENT PRINTING OFFICE: 1951—951455

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF AUGUST 31, 1951

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES	ACTIVE PROGRAM											
		PROGRAMMED ONLY			PLANS APPROVED, CONSTRUCTION NOT STARTED			CONSTRUCTION UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$13,140	$16,968	$8,671	364.1	$6,384	$3,238	154.4	$17,497	$8,562	391.2	$40,849	$20,471	909.7
Arizona	671	4,471	3,181	99.1	2,822	1,983	42.7	3,797	2,694	39.3	11,090	7,858	181.1
Arkansas	2,993	10,813	5,781	388.2	6,889	3,441	208.1	13,668	6,971	360.9	31,370	16,193	957.2
California	6,981	24,413	4,100	60.3	12,915	5,611	137.9	67,070	32,225	238.7	104,398	41,936	436.9
Colorado	3,591	2,371	1,335	73.1	3,307	1,808	88.2	14,607	7,988	330.2	20,285	11,131	491.5
Connecticut	3,203	3,250	1,675	11.3	4,802	2,360	8.2	9,555	5,171	12.3	17,607	9,206	31.8
Delaware	1,541	4,532	1,613	37.4	458	227	9.2	3,749	1,810	17.6	8,739	3,650	64.2
Florida	4,061	12,324	6,291	321.9	6,098	3,099	85.0	21,370	10,657	403.0	39,792	20,047	809.9
Georgia	3,952	17,392	8,859	445.4	10,038	4,266	82.9	29,317	14,588	570.1	56,747	27,713	1,098.4
Idaho	4,019	9,729	6,106	328.0	1,379	848	54.7	8,460	4,778	252.9	19,568	11,732	635.6
Illinois	18,419	38,731	20,947	352.3	16,136	8,113	128.7	72,415	37,240	646.3	127,282	66,300	1,127.3
Indiana	11,562	32,764	16,414	174.3	10,233	4,965	134.8	17,529	8,922	164.9	60,526	30,301	474.0
Iowa	1,737	10,870	5,566	347.6	6,953	3,512	290.7	21,199	10,950	771.5	39,022	20,028	1,409.8
Kansas	6,563	8,442	4,064	1,239.6	3,732	1,871	223.7	14,321	7,142	529.2	26,495	13,077	1,992.5
Kentucky	667	20,502	10,734	375.6	7,292	3,704	158.9	14,232	7,043	189.8	42,026	21,481	724.3
Louisiana	2,401	17,781	8,539	80.8	9,880	4,658	76.8	20,397	10,336	207.9	48,058	23,533	365.5
Maine	1,936	8,261	4,500	69.3	2,087	1,001	20.2	7,434	3,872	62.8	17,782	9,373	152.3
Maryland	4,484	1,936	974	33.8	6,285	3,871	19.7	11,298	4,924	39.6	19,519	9,769	93.1
Massachusetts	439	14,804	6,547	32.9	6,275	2,793	2.0	70,124	34,911	59.1	91,203	44,251	94.0
Michigan	3,220	23,126	11,570	423.7	9,334	4,658	261.8	51,108	21,528	391.2	83,568	37,756	1,076.7
Minnesota	3,981	5,101	2,667	799.9	5,463	2,853	409.8	28,053	15,450	945.3	38,617	20,970	2,155.0
Mississippi	7,847	5,140	2,462	230.7	6,166	3,252	158.0	16,032	8,184	477.4	27,338	13,898	866.1
Missouri	8,472	23,780	12,424	764.5	11,846	5,899	246.7	38,125	20,080	682.1	73,751	38,403	1,693.3
Montana	4,688	15,047	9,208	450.9	4,000	2,290	82.4	13,376	8,082	205.8	32,423	19,580	739.1
Nebraska	8,202	10,838	5,717	406.8	6,181	3,099	136.9	17,938	9,318	562.7	34,957	18,134	1,106.4
Nevada	2,814	4,464	3,711	89.4	1,763	1,482	135.7	2,795	2,328	96.0	9,022	7,521	321.1
New Hampshire	2,603	2,244	1,306	13.9	637	295	5.3	5,476	2,712	48.4	8,357	4,313	67.6
New Jersey	2,834	12,949	6,355	25.9	9,421	4,698	14.9	16,860	7,993	17.7	39,230	19,046	58.5
New Mexico	1,833	4,349	2,783	172.2	1,689	1,076	52.7	10,997	7,027	190.2	17,035	10,886	415.1
New York	25,545	76,919	39,785	177.7	20,883	10,093	144.4	115,568	53,499	381.2	213,370	103,377	703.3
North Carolina	4,565	15,231	7,505	357.0	4,327	2,005	103.3	25,667	12,671	563.3	45,225	22,181	1,023.6
North Dakota	1,874	6,286	3,258	1,104.9	8,858	4,435	721.2	8,599	4,295	721.1	23,743	11,988	2,547.2
Ohio	12,588	33,695	15,972	256.9	3,368	1,753	63.5	78,807	39,800	387.4	115,870	57,525	707.8
Oklahoma	3,747	14,986	6,977	240.5	3,558	2,004	56.2	23,179	11,959	352.8	41,723	20,940	649.5
Oregon	752	1,971	1,159	19.1	2,639	1,420	43.3	17,610	9,837	229.6	22,220	12,416	292.0
Pennsylvania	11,572	24,953	12,469	79.5	9,646	4,821	34.0	67,837	33,910	194.2	102,436	51,200	307.7
Rhode Island	1,592	5,613	2,806	45.7	714	357	1.7	14,646	7,447	12.3	20,973	10,610	59.7
South Carolina	2,842	10,412	5,311	238.0	3,258	1,629	85.1	10,652	5,556	172.5	24,322	12,496	495.6
South Dakota	858	6,501	3,778	610.0	5,310	3,075	347.7	10,498	6,096	669.4	22,309	12,949	1,627.1
Tennessee	2,147	13,673	6,500	394.6	8,170	3,765	146.3	26,280	12,539	353.6	48,123	22,804	894.5
Texas	9,038	1,459	621	27.5	16,207	8,372	128.1	56,214	26,752	687.9	73,880	35,745	843.5
Utah	2,359	5,349	4,140	110.0	537	389	9.5	4,695	3,422	111.6	10,581	7,951	231.1
Vermont	637	5,269	2,844	59.4	367	185	6.5	4,673	2,323	26.3	10,309	5,352	92.2
Virginia	5,634	18,385	9,109	465.6	8,962	4,498	248.7	18,389	9,058	258.5	45,736	22,665	972.8
Washington	2,452	10,419	4,584	127.8	2,956	1,469	32.4	20,843	9,993	159.3	34,218	16,046	319.5
West Virginia	2,765	12,245	6,151	163.1	4,206	2,118	38.9	11,139	5,634	117.8	27,590	13,903	319.8
Wisconsin	6,401	19,950	10,994	391.1	7,840	3,742	179.8	20,921	10,331	460.9	48,711	25,067	1,031.8
Wyoming	305	2,154	1,494	48.2	1,715	1,095	60.0	7,657	4,978	222.8	11,526	7,567	331.0
Hawaii	1,081	8,445	3,692	14.4	1,683	829	12.0	9,356	3,504	23.7	19,484	8,025	50.1
District of Columbia	3,535	2,864	1,432	.7	200	100	.7	4,079	2,232	3.2	7,143	3,764	4.6
Puerto Rico	3,043	10,336	4,820	46.8	3,091	1,474	3.3	8,998	4,080	42.5	22,425	10,374	92.6
TOTAL	244,186	674,507	339,501	13,191.4	298,960	150,599	5,897.6	1,205,106	603,402	15,056.0	2,178,573	1,093,502	34,145.0

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