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# Public Roads

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# Abrasion Resistance of Bridge Paints for Use in Alaska—Field and Laboratory Tests Evaluated

BY THE OFFICE OF RESEARCH AND DEVELOPMENT BUREAU OF PUBLIC ROADS

Reported by[1] BERNARD CHAIKEN, Principal Research Chemist, Materials Division

*Findings from a study of the resistance of paint systems to abrasion in the Alaska Copper River Delta area are reported in this article. The study was instituted because of the rapid and premature failure of a conventional highway paint system applied to old railroad bridges modified for highway use as part of the Copper River Highway-Edgerton Cutoff. Tests were made to determine the most suitable paint system for this area where it must resist abrasion from extremely large amounts of aeolian sand, silt, and ice crystals carried by winds having high velocities, as much as 90 m.p.h.*

*A field study and accelerated laboratory abrasion test were made to evaluate nine different paint systems. The superiority of a catalyzed polysulfide paint system was demonstrated by results of the field study and confirmed by the laboratory tests. The other paint systems evaluated were coal tar, drying oil, alkyd, vinyl, epoxy ester, zinc-rich inorganic, neoprene, and a conventional red lead primer and aluminum varnish system.*

*Laboratory confirmation was needed to assess the validity of the field findings as the thicknesses for the applied paint systems varied considerably in the field study. The laboratory tests confirmed that the polysulfide coating was inherently more resistant to abrasion than the other paints when comparable film thicknesses were tested. The author recommends that a catalyzed liquid polysulfide rubber coating replace the ordinary red lead primer and aluminum varnish paint system for maintenance of steel bridges exposed to the extreme abrasive forces prevalent in the Copper River Delta of Alaska. He recommends that the dry coating thicknesses be limited to 12 to 15 mils and suggests that incorporation of a rust-inhibitive primer would improve the resistance of the paint system to corrosion.*

## Introduction

DURING the period 1953 to 1957, several abandoned railroad bridges in the Copper River Delta area of Alaska, which had been built and operated for some years by the Copper River and Northwestern Railroad, were modified for highway use as part of the Copper River Highway-Edgerton Cutoff, FAS Route 851, State Route 10. Both the completed and proposed portions of the new highway will eventually make use of the abandoned roadbed of the old railroad between Cordova and Chitina. The completed portion of this highway is shown as a solid line in figure 1.

In 1957, all new structural steel and guardrails on several of the modified bridges were shop-primed and field-coated with a red lead oil-alkyd paint conforming to Federal Specification TT-P-86a, Type II. A two-package aluminum paint, conforming to Federal Specifications TT-A-468a and TT-V-81b, was used as the finish coat. This paint system, in common use in highway work, failed completely within 6 months in places where erosion by windblown sand and ice particles was extreme. Several examples of extremely rapid paint failures and subsequent corrosion are shown in figures 2 through 7. It was obvious that a much more abrasion-resistant paint system was needed to protect these bridges of the Copper River Delta, as well as bridges in other areas where wind-related abrasion is extreme.

The lower Copper River Valley between the town of Chitina and the Gulf of Alaska seems to be the only low-elevation break in the Chugach Mountains through which large masses of air pass freely from the interior high-pressure areas to the low-pressure systems common in the Gulf of Alaska. The resultant high-velocity north winds, at times as high as 90 m.p.h. or more, sweep great volumes of sand and silt into the Copper River Delta where several old railroad bridges have been converted to highway use. The location of these bridges is shown in figure 1. Although no precise wind data are available from the lower Copper River Valley, it is known that the old Copper River and Northwestern Railroad at one time had an anemometer destroyed by winds of approximately 90 m.p.h. There are also old reports of loaded copper ore cars having been overturned by the extreme force of the wind. Maximum winds of 70–80 m.p.h. have been recorded at the Cordova airport, near the coast, some 14 miles west of the Mile 27 Bridge. However, it is known that stronger winds are common at the bridge sites in the Delta.

The windborne sand and silt comes from the interior glaciers as well as numerous sandbars located in the Delta. During the winter months the quantity of aeolian sand and silt is somewhat reduced but the windborne ice particles then become the main agent of erosion. In addition to persistent northerly winds, the Delta is subject to 98 inches of precipitation per year as measured at the Cordova airport. The general area is subject to a low annual mean temperature of 28° F. (21° F. in December to 53° F. in July).

After the rapid failure of the standard paint system applied to the newly erected steel in 1957, a careful inspection of the older parts of the structures was made to ascertain the condition of the paint that had been used by the railroad some years before. This older paint seemed to be in fairly good condition, as can be seen in figure 8. As a result, an effort was made to determine the nature of the paint used by the railroad and the history of its painting operations.

The railroad, completed about 1910, was built to serve the Kennecott Copper Mines in the Wrangell Mountains, some 160 miles from the seaport town of Cordova. The railroad

[1] The work reported here was performed in cooperation with the Planning and Research Section, Alaska Department of Highways, the Materials Section, Anchorage Division of the U.S. Bureau of Public Roads Regional Office, and the Public Roads research laboratories in Arlington, Va. Work performed by the State was financed with Federal-Aid funds.

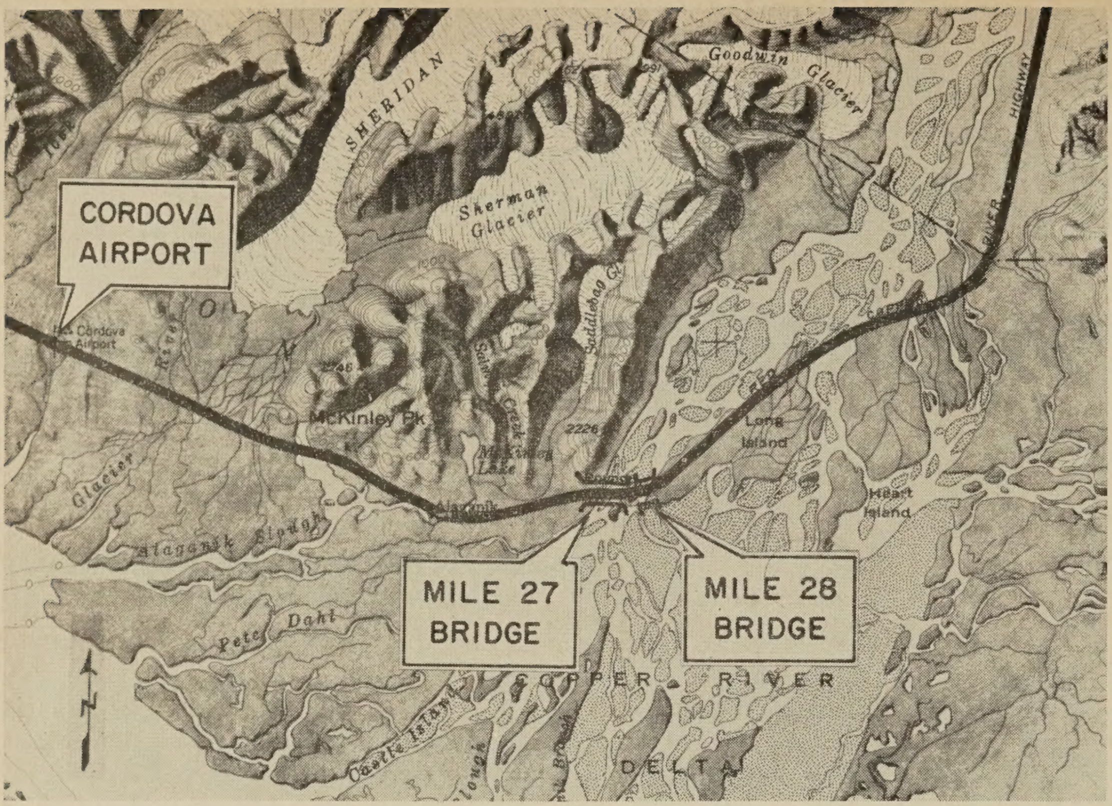



*Figure 1.—Completed section of the Copper River Highway, Alaska, is shown by the solid line. Sites for the paint tests are also indicated, Mile 27 and Mile 28 Bridges at the Copper River Delta.*

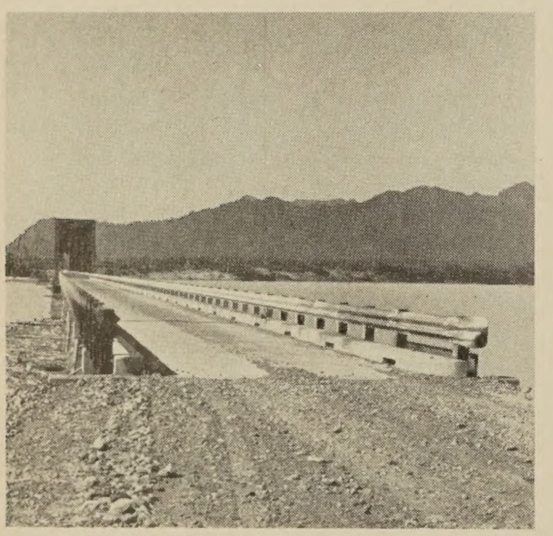


*Figure 2.—Paint erosion on guardrail of east abutment of Mile 27 Bridge.*

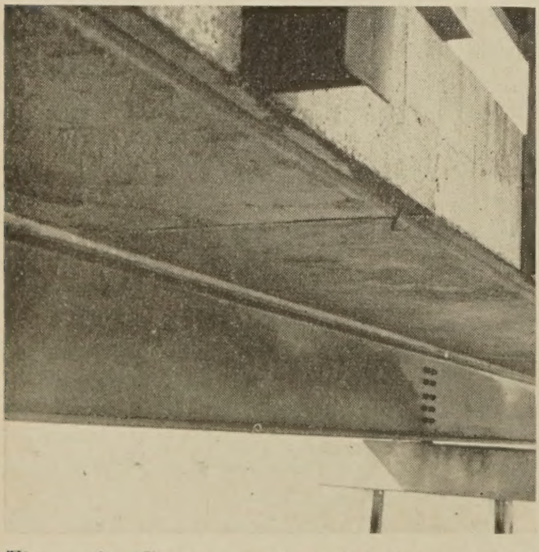


*Figure 3.—Severe erosion of paint on steel and even pitting of the concrete on north side of east end of Mile 27 Bridge.*

was operated through 1937, at which time the Kennecott mining operations were terminated. Records and field examination showed that the first paint system used on several of the railroad bridges, including those at Mile 27 and Mile 28, was a red lead shop primer, a field primer of *Marine Tockolith*, and a final coat of *RIW Waterproof Black*. Subsequent correspondence with the original paint supplier for these projects indicated that the final field coat was undoubtedly their *RIW No. 49 Black*, which was commonly used in that period, over an intermediate coat of their *RIW Marine Tockolith* (now called *RIW Tockolith*). It was further determined that the first repainting of the railroad bridges was done in 1917, some 7 years after the initial construction. In repainting, the metal was scraped and wire-brushed by hand, then brush-coated with a red lead and linseed oil primer, and brush topcoated with *Metalastic Black*. A routine spot repainting program, in which the 1917 paint system was employed, was reinstituted in 1921 and continued until the railroad was abandoned in 1937, according to a letter from Martin Fredeen, who was in charge of maintenance on the Copper River and Northwestern Railroad.

In 1958, much of this maintenance paint continued to provide effective protection to the unmodified structural steel after more than 25 years of exposure to the harsh climatic conditions described. Efforts in 1958 to obtain the *Metalastic* topcoat paint from the manufacturer were unsuccessful as its manufacture had been discontinued. Even the generic composition of this paint was unavailable at that time. However, subsequent to the completion of the study described in this article, private communications with R. T. Gabbert of the Sherwin-Williams Co. provided information on the probable composition of the Metalastic topcoat used in maintenance painting of the railroad bridges. His review of the company's old literature suggested that the Metalastic black topcoat paint used in the 1917–21 period may have been their Metalastic Black No. 2, a graphite drying oil paint having a probable composition of 32.8 percent pigment and 67.2 percent vehicle, which probably was a linseed or linseed tung oil blend. The pigment probably was composed of materials and percentages, as listed: Graphite, 54.; fibrous talc, 29.3; amorphous silica, 9.4; carbon (channel) black, 4.4; litharage, 2.5.

## *Conclusions and Recommendations*

From the paint systems studied and the results obtained, it is concluded that thick films of a catalyzed liquid polysulfide rubber coating (like System 8) should offer the best protection for steel bridges exposed to the extreme abrasive forces prevalent in the Copper River Delta of Alaska and that this system should replace the ordinary red lead primer and aluminum varnish system in standard use in that area. A word of caution and perhaps some modification of System 8 might be in order to assure good practical results. In field exposures D and E some sagging of the polysulfide system occurred perhaps as a consequence of too thick a film. It is therefore recommeded that the paint thickness be restricted to a dry thickness range of 12 to 15 mils to prevent the occurrence of sagging. It is of interest that the manufacturer of this coating recommended a 10-mil thickness. In a paint system similar to No. 8, inclusion of a rust-inhibitive primer would probably be very beneficial in increasing the paint's resistance to corrosion. This conclusion was reached because the panels used in the field study were sandblasted before being painted. Such a surface pretreatment would not always be practical—hand cleaning is more often used—and the danger of metal corrosion would be greater unless an adequate primer were used.

## *Purpose and Scope of Investigation*

Because of the premature failures of the standard paint system applied in 1957, and the unavailability of the effective maintenance paint used previously by the railroad, an outdoor exposure study was planned in 1958 and initiated in 1959 by the Materials Section, Anchorage Division, of the Bureau of Public Roads Regional Office in Alaska. This exposure study was later taken over—the Alaska Department of Highways assumed primary highway responsibility in July 1960—and completed by the Planning and Research Section of the Alaska Department of Highways.

A specification for a paint system providing adequate resistance to the natural abrasive forces encountered in the Copper River Delta was established as the ultimate goal for the field study. To arrive at this specification, different paint systems were exposed and their performance under outdoor exposure was evaluated. Many paint manufacturers, suppliers, and other authorities were consulted concerning the most suitable paint systems to be studied. Nine different paint systems were finally selected for evaluation, including as a control the standard system then in use in Alaska.

The paint systems selected for evaluation are identified, as follows: System No. 1, coal tar; System No. 2, drying oil; System No. 3, alkyd; System No. 4, vinyl; System No. 5, epoxy ester; System No. 6, zinc-rich inorganic; System No. 7, neoprene; System No. 8, polysulfide rubber; and System No. 9, aluminum varnish over red lead primer. Some explanations of a few of the paint systems are given here.

System No. 2 was presumably representative of the original *Tockolith* paint system used by the railroad during the original construction of several of the bridges in 1907–09 and was one that reportedly gave fairly good service prior to the use, starting in 1917, of the *Metalastic* maintenance paints.

System No. 3 was supposed to have been a phenolic system and was so represented by the paint supplier. However, subsequent laboratory analysis of this system, after the field study was well underway, revealed that it was alkyd based rather than phenolic.

System No. 9 was supposed to be the same as that used on the new steel and was intended to be the control system included for comparison. However, it was somewhat different than had been anticipated. The primer did not strictly conform to Federal Specification TT–P–86a, Type II, as it contained zinc oxide and zinc chromate along with red lead, iron oxide, and siliceous matter. In addition, the aluminum finish paint was ready-mixed rather than a two-package paint.

The investigation originally had been limited to an outdoor exposure study on bridges of the Copper River Highway, where severe paint deterioration had been noted. The paint systems were to be applied at normal coating thicknesses or as recommended by the paint supplier or producer. After the field investigation was well underway, it seemed worthwhile to make a complete chemical analysis of the paints used and to make laboratory abrasion tests for evaluation of the effects of film thickness differences. Accordingly, samples of the paint were sent to the Arlington laboratories of the U.S. Bureau of Public Roads, Materials Division for these purposes.

## *Field Exposures*

Two bridges of the Copper River Highway located at the Copper River Delta were selected for the field exposure tests, which were started in August 1959. These were the Mile 27 Bridge and Mile 28 Bridge, 14 miles east of the Cordova airport. Five types of paint exposure conditions were arranged. They will be referred to as exposures A, B, C, D, and E. They were selected to provide a range from severe to negligible erosion, as previously noted on the structural steel. In exposures A, B, and C, painted panels were strapped to different parts of the bridges, and in exposures D and E, actual parts of the steel structures were coated with the paint systems being investigated.

### Exposures A, B, and C

For exposures A, B, and C, 54 steel panels, $\frac{3}{16}$- by 12- by 18-inches in size, were prepared. All edges were ground smooth, and the test surfaces were sandblasted to a uniform bright gray metallic color to remove millscale and rust. Immediately before its being painted, each panel was cleaned with inhibited 1,1,1-trichloroethane solvent by cloth, and the prime coat was brushed on; the rest of the paint system was then applied, also by brush. Only one side of each panel was so coated and each of the nine paint systems was applied to six panels. The coating film thickness and drying time between coats used were those recommended by each supplier or manufacturer. Application data concerning the test panels for exposures A, B, and C are shown in table 1. Generally, the drying time between coats was overnight; the exceptions for special coatings and washcoats are listed in table 1.

In exposure A, one panel of each paint system was clamped to a mounting board attached to the vertical guardrail members on the north side of the eastern abutment of Mile 27 Bridge. Consequently, each panel was in a vertical position facing north and windward. This exposure was considered to be the most severe with respect to windborne abrasive elements. The arrangement of panels in this exposure is shown in figures 9 and 10. The arrangement shown in figure 10 seemed to have some significance, which will be discussed later, because the wind direction was from 15° below and 45° to the left of being perpendicular to the plane of the panels. Consequently, abrasive action was greatest at

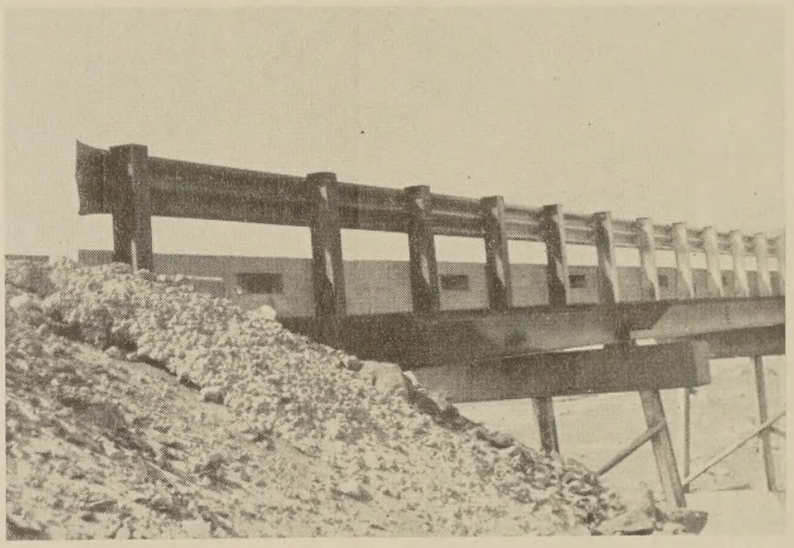


*Figure 4.—Severe paint erosion on guardrail of east end of Mile 27 Bridge.*

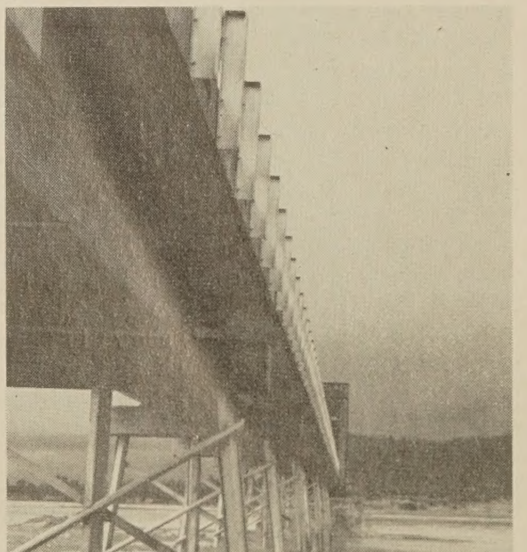


*Figure 5.—Severe erosion on windward side of I-beam piles on Mile 27 Bridge.*

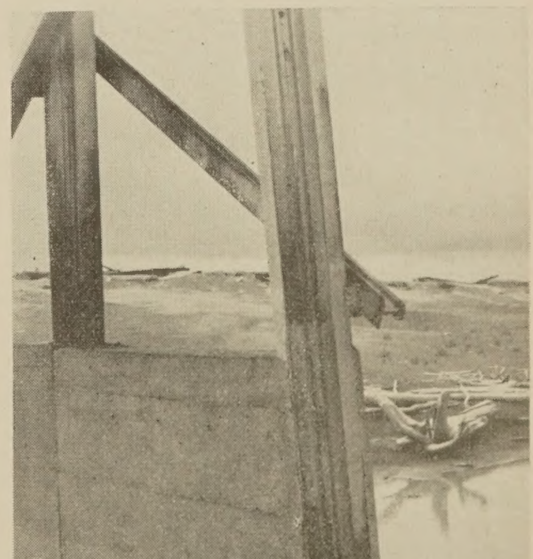


*Figure 6.—Close-up view of erosion shown in figure 5.*

*Figure 7.—Paint erosion on end post of Mile 28 Bridge.*

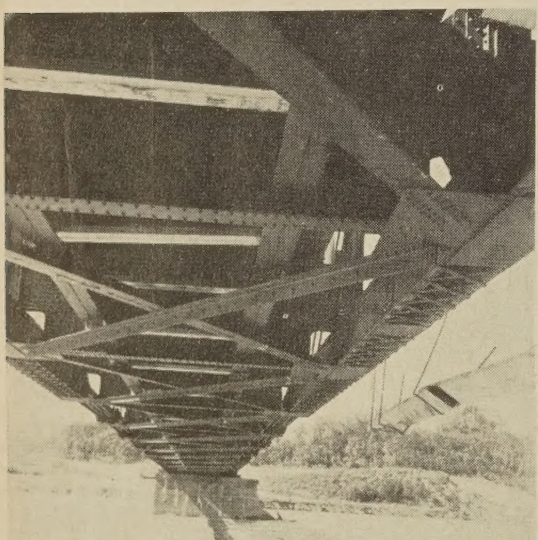


*Figure 8.—Sound condition of Metalastic Black maintenance paint used by railroad on underside of Mile 28 Bridge.*

*Figure 9.—Test mounting boards for exposure A near end of right guardrail at east abutment of Mile 27 Bridge.*

**Table 1.—Data on application of Paint Systems to panels for exposures A, B, and C [1]**

| Paint System | Individual coatings | | | | Dry film thickness [2] | |
|---|---|---|---|---|---|---|
| | Name | Color | Coats applied | Minimum drying time per coat | Individual coating [3] | Total for Paint Systems |
| | | | *Number* | *Hours* | *Mils* | *Mils* |
| No. 1, Coal tar: | | | | | | |
| 1-1 Washcoat | Vinyl butyral [4] | Olive, clear | 1 | 1 | 0.2–0.5 | 6–7 |
| 1-2 Primer | Coal tar cutback | Black | 1 | 24 | 3.5 | |
| 1-3 Finish | Coal tar emulsion | do | 1 | -------- | 2–3 | |
| No. 2, Drying oil: | | | | | | |
| 2-1 Primer | White lead and zinc oxide in oil. | Gray | 1 | 48 | 0.8–1.0 | 3–4 |
| 2-2 Finish | Carbon black and red lead in oil. | Black | 1 | 24 | 2.2–3.0 | |
| No. 3, Alkyd: | | | | | | |
| 3-1 Primer | Red lead alkyd | Brown | 1 | 24 | 0.5–0.8 | 1.5 |
| 3-2 Finish | do | Orange | 1 | 24 | 0.8–1.0 | |
| No. 4, Vinyl: | | | | | | |
| [5] 4-1 Washcoat | Vinyl butyral [4] | Olive, clear | 1 | 1 | 0.2–0.5 | 3.5–4.0 |
| [6] 4-2 Finish | Titanium dioxide vinyl | White | 3 | Several | 3.2–3.8 | |
| No. 5, Epoxy ester: | | | | | | |
| 5-1 Primer | Red lead epoxy ester | Brown | 1 | 24 | 1.0–1.5 | 3.0 |
| 5-2 Finish | Titanium dioxide epoxy ester | White | 1 | 24 | 1.5–2.0 | |
| No. 6, Zinc-rich inorganic: | | | | | | |
| 6-1 Finish | Inorganic zinc silicate [7] | Gray | 2 | 1 | 3–4 | 3–4 |
| 6-2 Cure | Phosphoric acid solution | Colorless | 2 | Several | 0 | |
| No. 7, Neoprene: | | | | | | |
| 7-1 Primer | Neoprene solution | Amber, clear | 1 | <24 | 0.2–0.5 | 4 |
| 7-2 Finish | Carbon black and neoprene | Black | 1 | 24 | 3.5–3.8 | |
| No. 8, Polysulfide: | | | | | | |
| 8-1 Primer | Chlorinated rubber | Colorless | 1 | <24 | 0.5 | 15–20 |
| 8-2 Finish | Catalyzed polysulfide [8] | Black | 1 | 48 | 15–20 | |
| No. 9, Aluminum varnish: | | | | | | |
| 9-1 Primer | Red lead alkyd | Brown | 1 | 24 | 0.5–0.8 | 2 |
| 9-2 Finish | Aluminum oleoresinous varnish.[9] | Aluminum | 1 | 24 | 1.2–1.5 | |

[1] 3/16- by 12- by 18-inch steel panels. Panels sandblasted on one side to bare metal to remove millscale; cleaned with hibited 1, 1, 1-trichloroethane by cloth; and coating brushed on.
[2] Determined magnetically by Elcometer gage.
[3] Average or range for the six panels.
[4] One part catalyst to six parts resin by volume.
[5] Same as washcoat 1-1.
[6] Paint appeared rubbery and did not brush well.
[7] 23 pounds zinc dust to 2/3 gallon of liquid component.
[8] One part accelerator to 10 parts base resin by weight.
[9] Ready-mixed.

the left and bottom where System 6 was located, and least at the top right, where System 5 was located.

In exposure B, four replicate panels of each paint system were clamped to mounting boards attached to the east end of the northern lower chords of Mile 28 Bridge. The panel exposures were vertical, north, and 45° from windward; the same as in exposure A. However, exposure B was considered to be less intense than that of exposure A. The arrangement of these panels is shown in figure 11. All four panels of each system were grouped together, Systems 1, 2, 3, and so on, arranged in increasing order from east (left) to west (right).

Exposure C was the least severe of the three panel exposures, as it was in a somewhat sheltered area. This exposure was used to obtain a relative measure of coating performance under minimal wind conditions. In this exposure, one panel of each paint system was clamped to a mounting board attached to the opposite, and therefore protected (south) side of the same chord on Mile 28 Bridge as for exposure B. Panel positions were thus vertical and facing south. The arrangement of the panels for Systems 1 through 9 is shown in figure 12.

### Exposures D and E (paint on bridge su faces)

In addition to the three types of panel e posures, each of the paint systems was appli directly in exposures D and E to parts of t bridge surface of the Mile 28 Bridge simulate actual painting practices. T bridge surfaces were either wire-brushed left unprepared as necessitated by the c served condition of the existing paint on t steel. An attempt was made to obtain t same film thickness with each applied coati as had been obtained in the test panel seri In both of these exposures, the paint w applied to the vertical faces (inboard w surfaces) of the top eastern end posts: Syst 1 paint was applied at the top and the otl systems were applied in order downward System 9 at the bottom.

At exposure D, which faced north, most of the original field paint had been eroc away; consequently, this area was wire-brusl to remove rust and loose scale before the t paints were applied. Exposure here v vertical and north and therefore subject considerable erosion. At exposure E, original paint on the bridge surface was good condition and therefore no special p cleaning was done. These test paints w

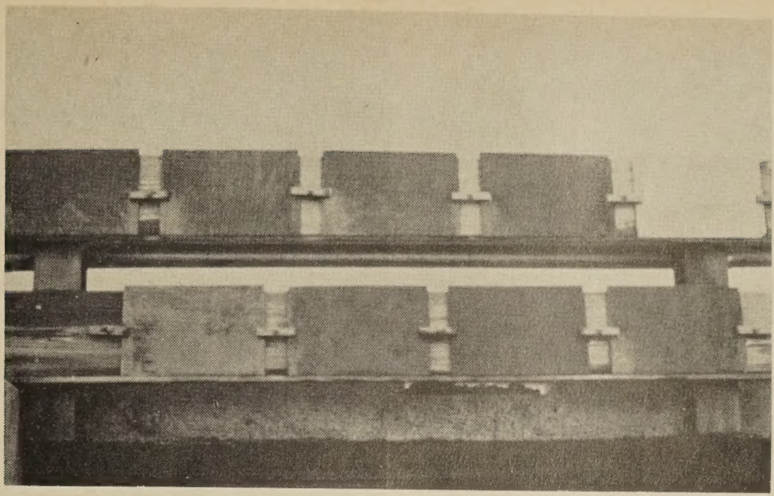


*Figure 10.—Close-up view of panel arrangement at exposure A (top row, Systems 1 through 5; bottom row, Systems 6 through 9).*

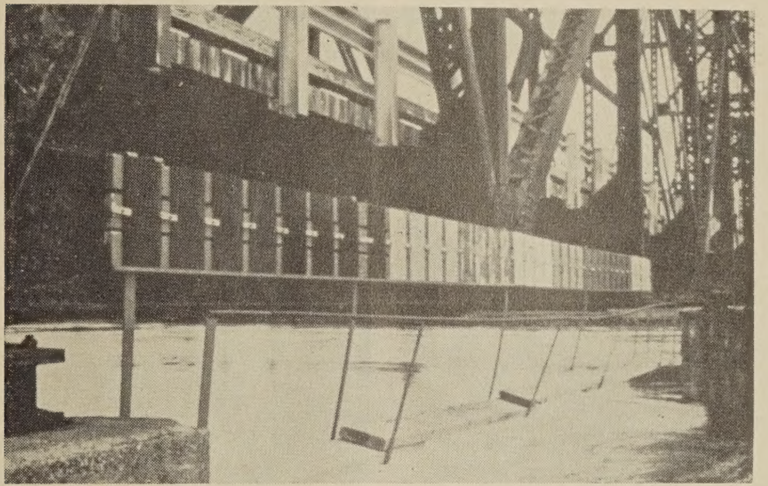


*Figure 11.—Panel arrangement at exposure B on east end of northern lower chords of Mile 28 Bridge.*

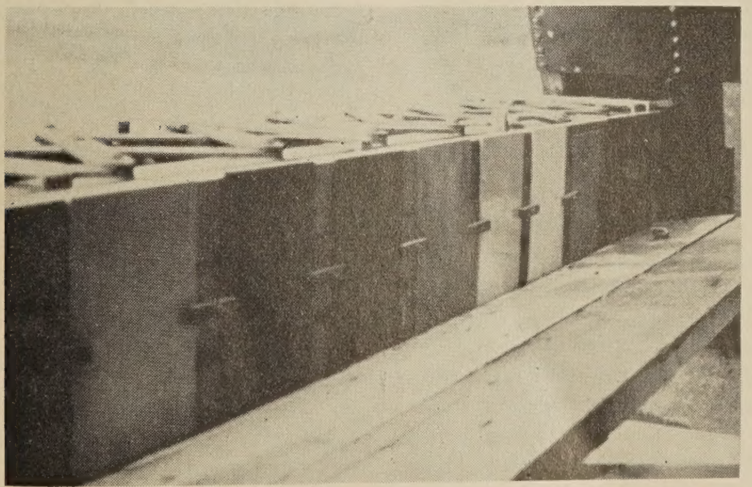


*Figure 12.—Exposure C. Panels attached to same chord as exposure B, but on opposite side, facing south.*

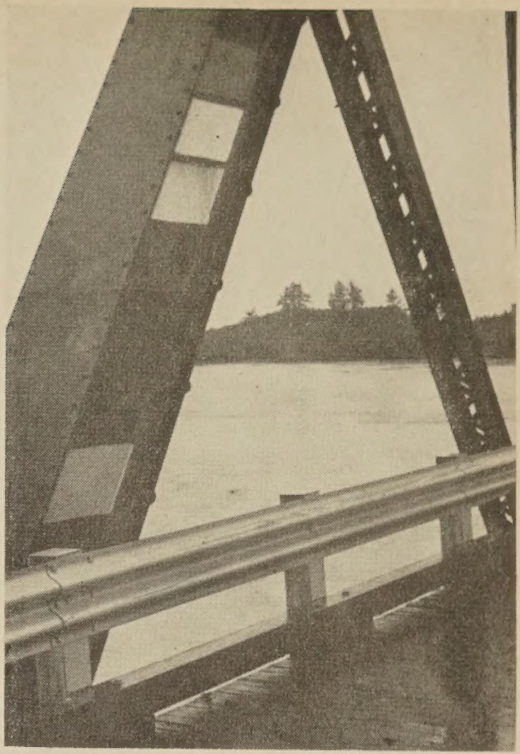


*Figure 13.—Exposure D at Mile 28 Bridge faced north and subject to erosive forces.*

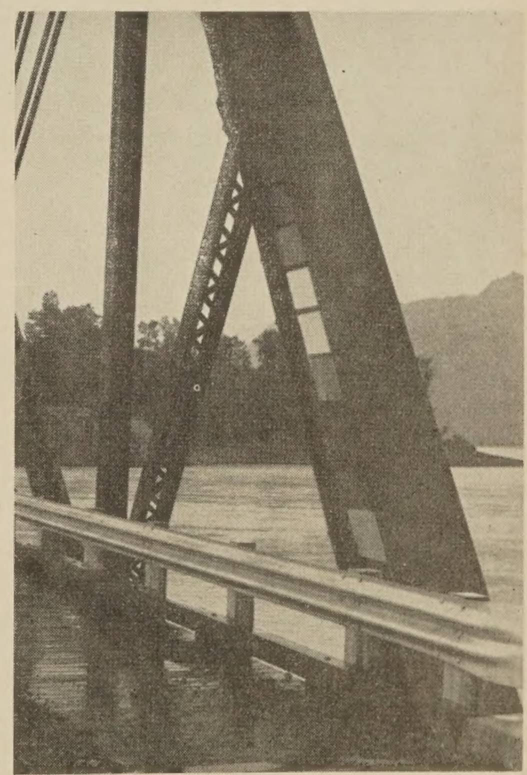


*Figure 14.—Exposure E at Mile 28 Bridge faced south and protected.*

also applied in a vertical plane, but facing south so that effective wind forces were not as severe as in exposure D. The arrangement of painted surfaces in these two exposures is shown in figures 13 and 14.

## Results and Evaluation of Field Exposures

During the exposures, the paint systems were periodically rated visually for factors of general appearance, erosion, rusting, gloss, chalking, checking, cracking, flaking, scaling, peeling, dirt, mildew, fading, darkening, and yellowing. In addition, blistering and sagging were noted when evident. These ratings were made on a 10 to 0 scale; 10 is perfect and 0 represents complete failure, in accordance with the standard procedures and pictorial standards provided for these factors by the American Society for Testing Materials (*1*).[2] Because only the trends in general appearance, erosion, and rusting seemed to be significant, the observed ratings for the other factors listed have not been included in this article so that the volume of reported data could be kept at a minimum. The test panels in exposures A, B, and C also were periodically measured for film thickness of the remaining coating by a magnetic dry-film thickness gage to obtain information that could be used to evaluate more fully the erosion resistant properties of the paint system. Such thickness measurements were not made on the paints applied directly to the structure in exposures D and E.

[2] References indicated by italic numbers in parentheses are listed on page 113.

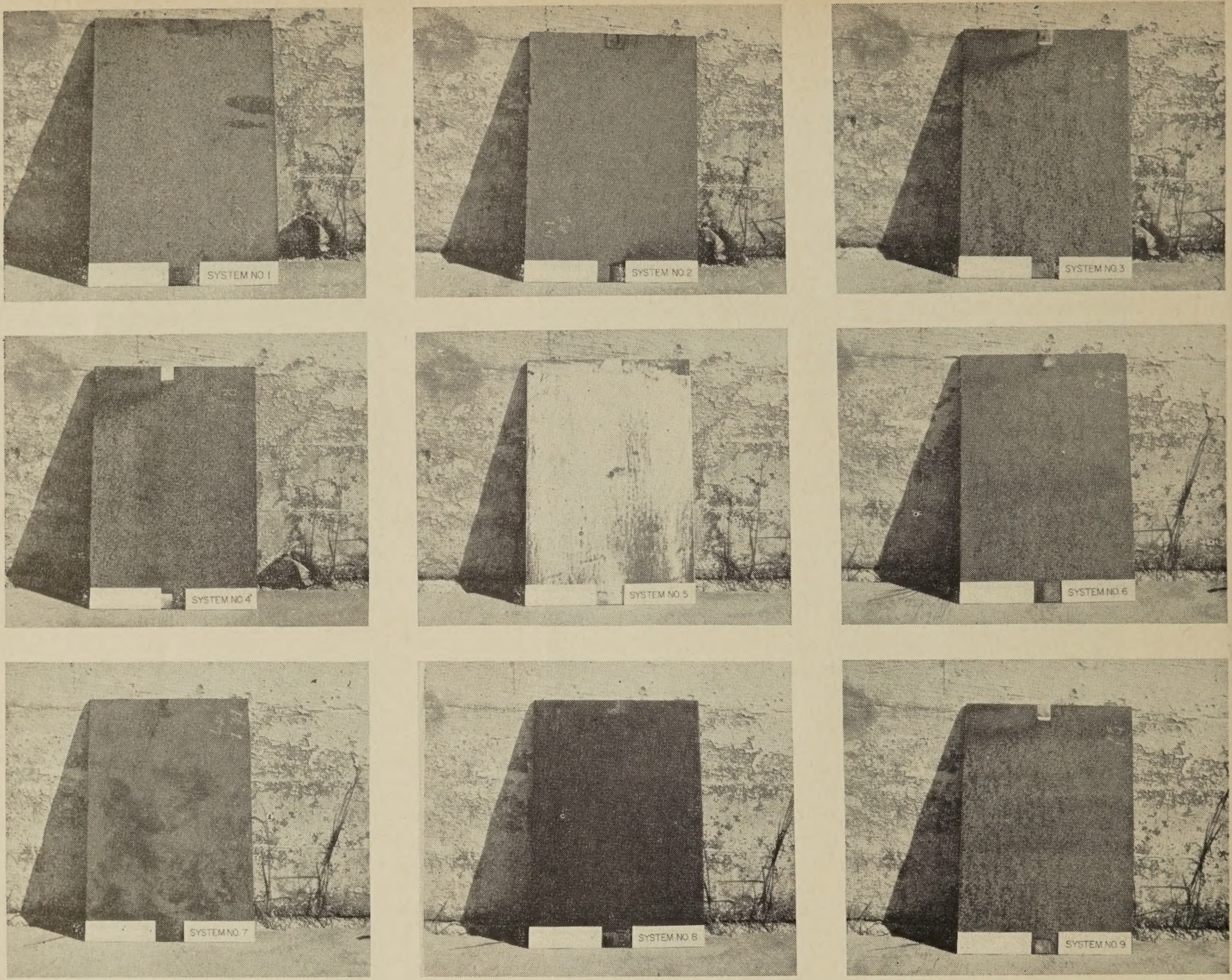



*Figure 15.—Condition of Paint System panels after termination of exposure A.*

The condition of the paint systems after 38 months at exposure A is illustrated on the nine panels photographed for figure 15. As had been expected, exposure A was confirmed as being the most severe of the five exposure conditions. Although the full extent of rusting is not too clearly shown by the black-and-white photographs, a field examination of these nine panels showed that, except for paint Systems 5 and 8, all systems tested failed badly. However, System 5 had substantial rusting, but paint System 8 was still in very excellent condition. The comparative performance of each of the paint systems is shown more clearly in figure 10, which was photographed during the exposure, and the superiority of Systems 5 and 8 are more clearly delineated.

The harsh effects of exposure A as compared to exposures B and C are demonstrated vividly in figure 16; the left, middle, and right panels in each block are from exposures A, B, and C, respectively. The panels on the left, from exposure A, were each time in the poorest condition.

The final ASTM numerical ratings for all five exposure conditions are shown in table 2 in terms of general appearance, erosion, and rusting. These results reflect quantitatively the condition of each paint system after either 23 or 38 months of exposure, depending upon when the particular exposure was terminated.

The tabulated results show that in exposure A, the most severe, only System 8, polysulfide rubber, provided complete protection to the underlying metal after the full 38 months of exposure. The general appearance and rust resistance of this system was considered to be excellent, and it received a rating of 10. Its performance in the other four, but less severe, exposures was equally good, except for some sagging noted in exposures D and E that perhaps was caused by the application of too thick a film of paint. System 5, epoxy ester, was the only coating other than System 8 that still provided some significant protection to the metal in exposure A after 23 months. However, performance of System 5 was considerably poorer than that of System 8 (fig. 15 and table 2). In the other exposures, B through E, the epoxy ester system appeare to give fair performance, as shown in table but it was outranked or equaled by Systen 1 and 2, as well as System 8. Personn conducting the field study were of the opinic that System 5 had unknowingly been place in a favored position in exposure A because the wind direction at that exposure, as d scribed earlier. This might also be the e planation for the relative difference in th system's performance at site A as compar to the other exposure sites.

To arrive at some overall quantitati evaluation of the paint systems in all fi exposures, the numerical ratings for the im portant factors, such as general appearanc erosion, and rusting at the age of 23 month were added together for all five exposure The results are shown in table 3. It obvious from the cumulative total ratin that System 8 demonstrated overall supe ority, having a cumulative total rating of 14 out of a possible maximum rating of 15 Systems 2 and 5 were next; they had ratin of 117 and 116, respectively. The presumab

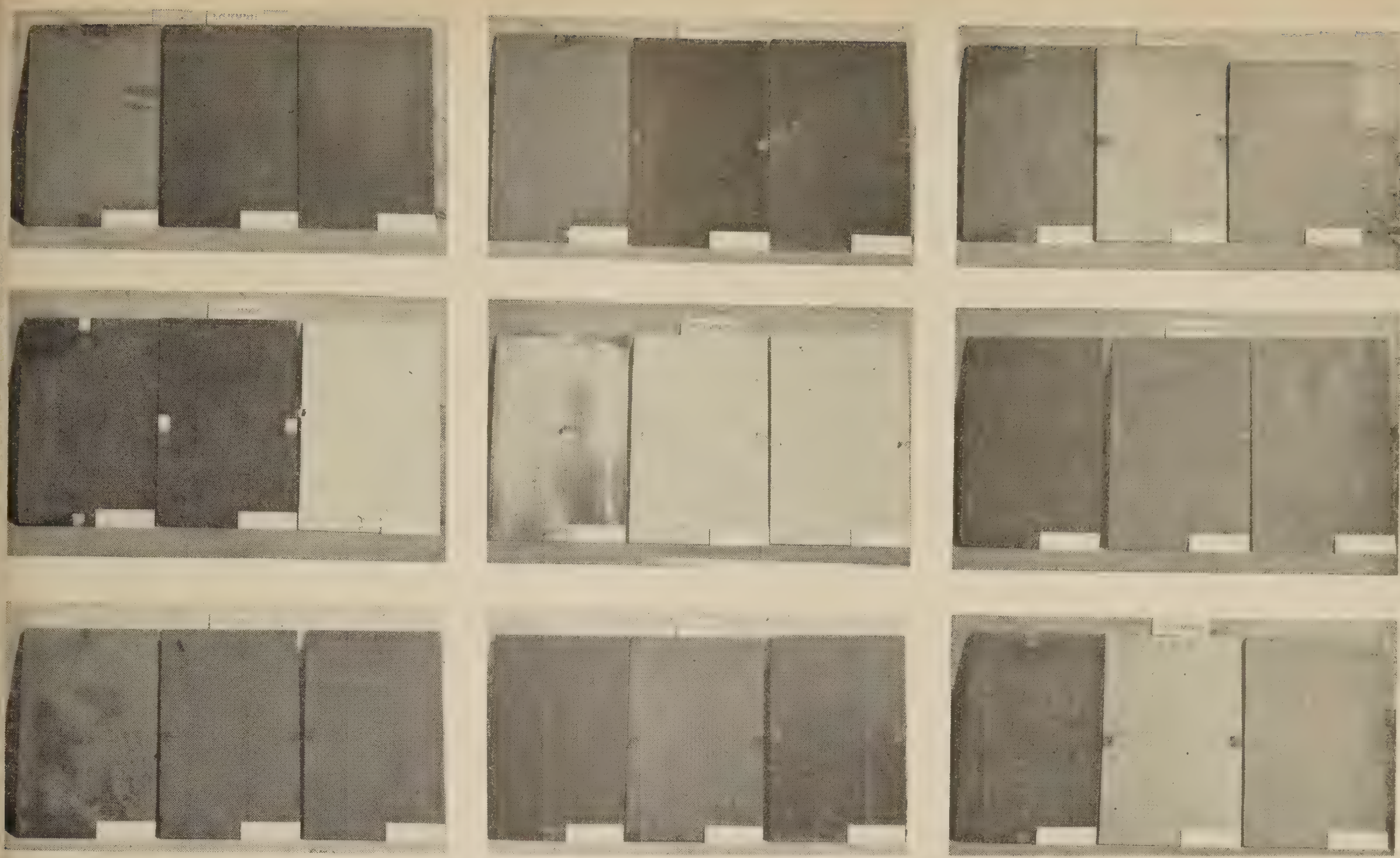


*Figure 16.—Comparative condition of Paint Systems after exposures A, B, and C.*

andard system, No. 9, was inferior to many the paint systems studied, and the vinyl and nc-rich paint systems (4 and 6) were among ie most inferior.

Information on the important factor of isting was used to explore the trends on isting under the most severe test of exposure . The change in the amount of rusting in lation to exposure time is shown in figure 17. he results shown here confirmed the impressions formed from other data that System 8 as superior to the other paint systems and dicated that it should have an expected life r in excess of the 3-year test of severe posure.

Although System 5 had the next best erformance, on the basis of test results, it ould not be expected to give maintenance-free rvice for more than 2 to 3 years. This onclusion is based on a rule-of-thumb inciple in maintenance painting of steel that hen the numerical value of resistance to isting diminishes to 6, repainting becomes ecessary. The resistance to rusting of System 5 diminished to 6 after 23 months of posure.

**ffect of original film thickness**

A serious question might be raised regarding ie entire exposure study concerning the tent to which the original film thickness of ich coating contributed to ultimate performance. It could be argued that, by creasing the original film thickness of some

**Table 2.—Panel ratings for general appearance, erosion, and rusting [1]**

| Condition rated | Paint System— | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 |
| EXPOSURE A, 23 MONTHS [2] | | | | | | | | | |
| General appearance | 0 | 0 | 0 | 0 | 4 | 0 | 2 | 10 | 0 |
| Erosion | 0 | 0 | 0 | 0 | 6 | 0 | 3 | 9 | 0 |
| Rusting | 0 | 1 | 0 | 1 | 6 | 2 | 2 | 10 | 2 |
| EXPOSURE B, 38 MONTHS | | | | | | | | | |
| General appearance | 7 | 9 | 4 | 0 | 8 | 1 | 8 | 9 | 5 |
| Erosion | 9 | 10 | 8 | 0 | 7 | 1 | 8 | 10 | 6 |
| Rusting | 10 | [3] 10 | 8 | 0 | 8 | 2 | 7 | 10 | [3] 8 |
| EXPOSURE C, 38 MONTHS | | | | | | | | | |
| General appearance | 8 | 9 | 6 | 6 | 8 | 1 | 7 | 8 | 3 |
| Erosion | 10 | 10 | 9 | 9 | 10 | 1 | 5 | 10 | 4 |
| Rusting | 10 | 10 | [3] 8 | [3] 8 | 7 | 2 | 6 | 10 | [3] 4 |
| EXPOSURE D, 23 MONTHS | | | | | | | | | |
| General appearance | 10 | 10 | 10 | 1 | 8 | 0 | 9 | 9 | 9 |
| Erosion | 8 | 8 | 7 | 1 | 8 | 0 | 8 | 8 | 6 |
| Rusting | 10 | 10 | 10 | 6 | [3] 8 | 4 | 10 | [4] 10 | [3] 8 |
| EXPOSURE E, 23 MONTHS | | | | | | | | | |
| General appearance | 8 | 10 | 9 | 2 | 8 | 4 | 10 | 8 | 8 |
| Erosion | 6 | 10 | 8 | 1 | 10 | ([5]) | 10 | 10 | 8 |
| Rusting | [3] 10 | [3] 10 | 10 | 10 | 10 | 7 | 10 | [4] 10 | 10 |

[1] ASTM method of rating where 10=perfect and 0=complete failure.
[2] Ratings after 38 months of exposure A was: for System 5, general appearance 2, erosion 1, and rusting 3; for System 7, general appearance 0, and erosion 0; for System 8, general appearance 10, erosion 9, and rusting 10.
[3] Some blistering noted.
[4] Some sagging noted.
[5] Erosion rating not made.

of the apparently inferior paints, better performance could have been attained. This is a reasonable and legitimate point and is discussed in more detail in relation to the laboratory studies. However, some consideration is given here to the field exposure data collected on film thickness.

The data obtained on the film thickness of each paint system, both before and during exposure, are shown in table 4. These data were obtained only for the test panels in exposures A, B, and C. Similar data were not recorded for exposures D and E as existing paint, where intact, had not been removed from the bridge member before application of the test paint. Consequently, accurate measurements of the applied test paints were not

**Table 3.—Cumulative numerical analysis of panel ratings for general appearance, erosion, and rusting at 23 months' exposure**

| Paint System | Total ratings of panels for exposures A through E | | | |
|---|---|---|---|---|
| | General appearance | Erosion | Rusting | Cumulative total |
| 1 | 33 | 33 | 40 | 106 |
| 2 | 38 | 38 | 41 | 117 |
| 3 | 29 | 32 | 36 | 97 |
| 4 | 9 | 11 | 25 | 45 |
| 5 | 36 | 41 | 39 | 116 |
| 6 | 6 | [1] 2+ | 17 | [1] 25+ |
| 7 | 36 | 34 | 35 | 105 |
| 8 | 44 | 47 | 50 | 141 |
| 9 | 25 | 24 | 32 | 81 |

[1] Erosion rating for Paint System 6 in exposure E was not made and therefore is not included in totals.

**Table 4.—Decrease in film thickness in relation to length of exposure**

| Paint System | Average film thickness, mils | | | | | | |
|---|---|---|---|---|---|---|---|
| | Before exposure | After exposure by months | | | | | |
| | | 11 | 15 | 23 | 33 | 35 | 38 |
| EXPOSURE A (MILE 27 BRIDGE, NORTH EXPOSURE, 1 PANEL EACH) | | | | | | | |
| 1 | 6.5 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 2 | 3.5 | 0.2 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 3 | [1] 1.5(4.5) | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 4 | 3.8 | 1.5 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 5 | [1] 3.0(5.0) | 4.0 | 2.1 | 1.9 | [2] 0.4 | [2] 0.2 | [2] 0.2 |
| 6 | 3.5 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 7 | 4.0 | 2.2 | 1.7 | [2] 0.4 | 0.0 | 0.0 | 0.0 |
| 8 | 18.0 | 13.5 | 9.5 | 9.1 | 8.8 | 8.7 | 8.7 |
| 9 | 2.0 | 0.4 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| EXPOSURE B (MILE 28 BRIDGE, NORTH EXPOSURE, 4 PANELS EACH) | | | | | | | |
| 1 | 6.5 | 6.5 | 5.3 | 4.9 | 4.0 | 3.6 | 3.5 |
| 2 | 3.5 | 3.4 | 2.4 | 1.8 | 2.1 | 1.8 | 1.6 |
| 3 | [1] 1.5(4.5) | 3.0 | 1.8 | 1.6 | 1.6 | 1.2 | 1.0 |
| 4 | 3.8 | 1.1 | 0.7 | 0.0 | 0.0 | 0.0 | 0.0 |
| 5 | [1] 3.0(5.0) | 4.6 | 3.7 | 3.0 | 3.0 | 4.1 | 2.4 |
| 6 | 3.5 | 2.6 | 2.3 | 2.8 | 2.1 | ---------- | 2.0 |
| 7 | 4.0 | 1.8 | 1.8 | 2.4 | 1.7 | ---------- | 1.5 |
| 8 | 18.0 | 15.0 | 14.6 | 11.2 | 8.8 | ---------- | 9.0 |
| 9 | 2.0 | 2.2 | 1.4 | 2.1 | 1.1 | ---------- | 1.0 |
| EXPOSURE C (MILE 28 BRIDGE, PROTECTED SOUTH EXPOSURE, 1 PANEL EACH) | | | | | | | |
| 1 | 6.5 | 4.5 | 4.0 | 4.2 | 3.9 | ---------- | 4.0 |
| 2 | 3.5 | 2.5 | 1.6 | 1.3 | 1.9 | ---------- | 1.8 |
| 3 | [1] 1.5(4.5) | 4.8 | 3.9 | 4.6 | 3.9 | ---------- | 3.6 |
| 4 | 3.8 | 3.0 | 1.8 | 1.9 | 1.1 | ---------- | 1.7 |
| 5 | [1] 3.0(5.0) | 4.5 | 2.5 | 2.9 | 2.6 | ---------- | 2.5 |
| 6 | 3.5 | 3.0 | 2.0 | 1.6 | 1.8 | ---------- | 1.7 |
| 7 | 4.0 | 2.0 | 1.1 | 1.0 | 1.0 | ---------- | 1.0 |
| 8 | 18.0 | 18.0 | 13.6 | 15.0 | 10.6 | ---------- | 12.0 |
| 9 | 2.0 | 2.2 | 1.0 | 0.9 | 0.9 | ---------- | 0.9 |

[1] These measurements of film thickness are very questionable as subsequent readings, especially in the milder exposur B and C, were substantially more, which meant that the paint was thicker than recorded originally. Consequently, t values shown in parentheses were obtained by extrapolation to zero exposure time and probably are measurements closer the true original thicknesses.

[2] Measured on irregular areas where paint remained between eroded and rusted patches.

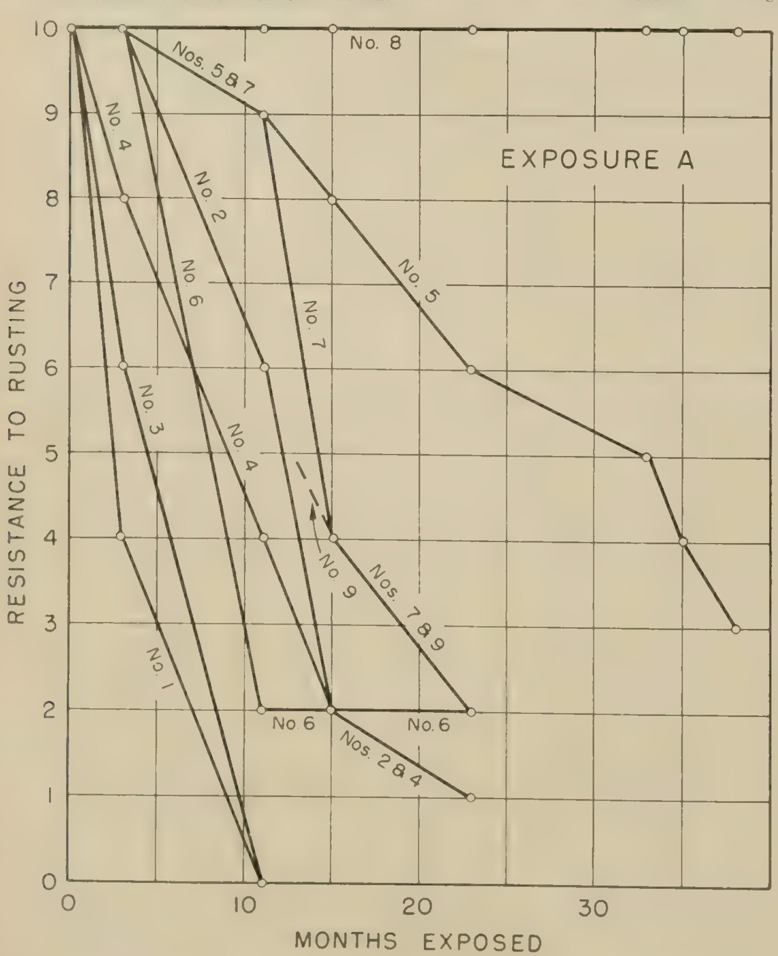



*Figure 17.—Decrease in resistance to rusting at exposure A in relation to duration of exposure.*

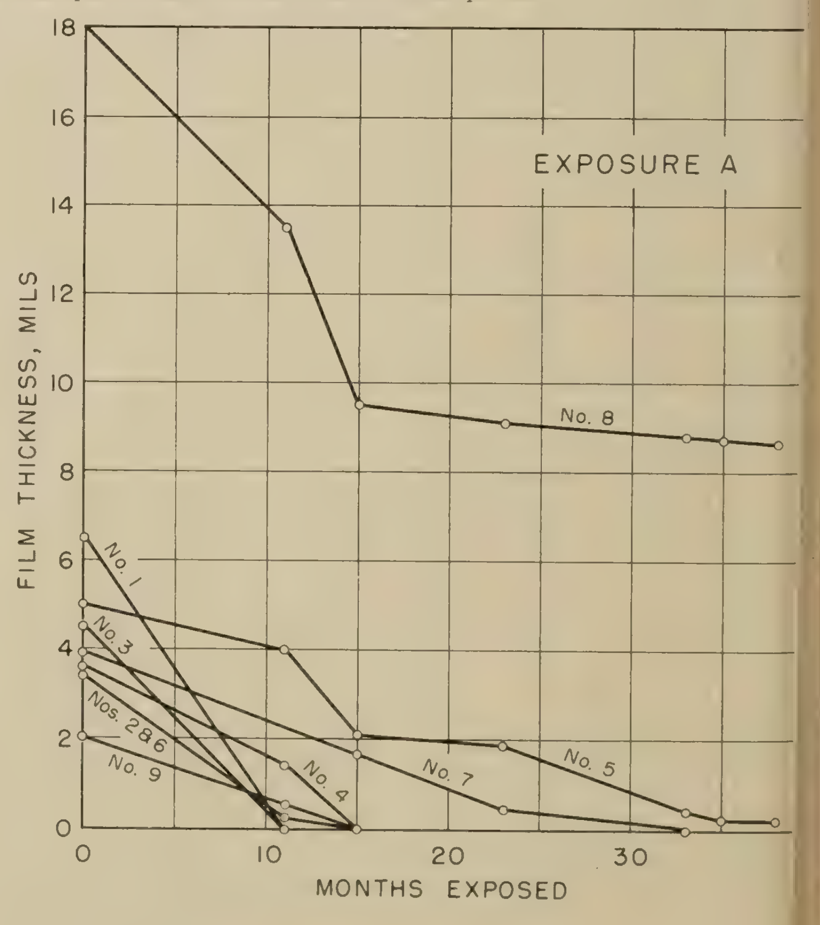



*Figure 18.—Decrease in film thickness of each Paint System duri exposure A.*

**Table 5.—Analysis of Paint System 1**

| Paint System 1 | Washcoat coating, 1-1 | | | Primer 1-2 | Finish 1-3 |
|---|---|---|---|---|---|
| | Resin component | Acid catalyst component | Mixture [1] | | |
| General properties: | | | | | |
| Color | ---------- | ---------- | Olive, clear | Black | Black |
| Weight per gallon ........ pounds | 7.1 | ---------- | ---------- | 11.0 | 10.3 |
| Viscosity ........ Krebs units | 72 | ---------- | ---------- | ca. 141 | 95 |
| Drying time: | | | | | |
| Set to touch ........ hours | ---------- | ---------- | 1/60 | 1 | ½ |
| Dry through ........ do | ---------- | ---------- | 1/10 | <24 | 1 |
| At dry film thickness ........ mils | ---------- | ---------- | ---------- | 4.4 | 4 |
| Composition, general: | | | | | |
| Pigment ........ pct. by wt. | 6.2 | None | ---------- | None | ---------- |
| Volatile ........ do | 84.7 | 84.1 | ---------- | 55.8 | 47.1 |
| Nonvolatile vehicle ........ do | 9.1 | 15.9 | ---------- | 44.2 | 52.9 |
| Nature of pigment | Zinc chromate with perhaps some magnesium silicate. | None | ---------- | ---------- | ---------- |
| Volatile: | | | | | |
| Nature, from infrared analysis | Isopropanol | Ethyl alcohol and water. | ---------- | Aromatic | Water |
| Nonvolatile vehicle composition: | | | | | |
| Nature, from infrared analysis | Polyvinyl butyral resin. | Phosphoric acid | ---------- | Coal tar | Coal tar |
| General nature | (2) | (2) | (2) | Coal tar cutback. | Coal tar emulsion. |

[1] Mixture of 1 part catalyst to 6 parts resin component, by volume.
[2] Washcoat 1-1 was basic zinc chromate vinyl butyral.

possible. Consideration of only the data on exposure A (table 4)—because the potential performance of the test paints were best sorted out in this harsh exposure—shows that the film thickness remaining after 23 or even 38 months of exposure reinforces the previous findings that paint Systems 5 and 8 were far superior to the others and that No. 8 furnished a very substantial amount of protection even after 38 months of exposure. These same data for exposure A are shown plotted in figure 18.

From data visual in figure 18, it is again apparent that paint System 8 will retain a satisfactory amount of film protection far beyond 38 months. The flat slope of the line (fig. 18) for System 8 from 15 to 38 months suggests a somewhat long future life for this paint. An expectation that the relative overall slopes of each line in figure 18 might have been used to predict the ultimate performance of each paint had they all been applied at equal thicknesses was not fulfilled. However, data plotted for Systems 1, 2, 3, and 6 provided steeper slopes overall, indicating generally that these paints were inferior to the rest of the paint systems. Data for the other paints are represented by flatter slopes, thus indicating that these paints might provide longer periods of protection if applied in thicker films.

**Table 6.—Analysis of Paint System 2**

| Paint System 2 | Primer coating, 2-1 | Finish coating, 2-2 |
|---|---|---|
| General properties: | | |
| Color | Gray | Black |
| Weight per gallon .. lbs. | 13.9 | 8.5 |
| Viscosity .. Krebs units | 54 | 61 |
| Drying time: | | |
| Set to touch ... hours | 6 | Within 24 |
| Dry through .... do | Within 24 | Within 36 |
| At dry film thickness mils | 2.6 | 3.9 |
| Composition, general: | | |
| Pigment ... pct. by wt. | 59.4 | 14.5 |
| Volatile ........ do | 12.3 | 8.5 |
| Nonvolatile vehicle Pct. by wt. | 28.3 | 77.0 |
| Pigment composition: | | |
| Basic carbonate white lead ........ percent | 39.8 | --------- |
| Siliceous matter .. do | [1] 26.9 | --------- |
| Zinc oxide ........ do | 18.8 | --------- |
| Calcium carbonate do | 14.4 | --------- |
| Carbon black ..... do | --------- | 81.2 |
| Lead ($Pb_3O_4$) ..... do | --------- | 8.4 |
| Iron oxide ($Fe_2O_3$) . do | --------- | 0 |
| Nonvolatile vehicle composition: | | |
| Iodine number of extracted fatty acids | 117 | 181 |
| Nature, from infrared analysis | (2) | (3) |
| General nature | (4) | (5) |

[1] Includes quartz.
[2] Essentially a modified drying oil (such as linseed) with other oils and/or resin.
[3] Essentially a processed or limed drying oil (such as linseed).
[4] White lead and extender pigment, in modified drying oil.
[5] Carbon black and red lead in modified drying oil.

## *Laboratory Evaluation*

As mentioned previously, a laboratory evaluation of the paint systems, as a supplement to the field studies, initially had not been considered. The laboratory work was undertaken after the field study was well underway. The purposes of this supplementary investigation were twofold. First, as complete an analysis of each paint as possible was desirable to generically define each paint and perhaps relate such information to performance and, at the same time, obtain a basis for preparing suitable specifications for the ultimate use of any promising paint system. Second, in the field tests the paints were applied in the film thicknesses recommended by the paint supplier or as normally applied in conventional practices. The field study, therefore, was limited to a comparison of paints of different film thicknesses. As film thickness may have contributed greatly to abrasion resistance, especially System 8, a decision was made to explore in accelerated laboratory tests the inherent abrasion resistance of each paint by the use of controlled film thicknesses. Thus the effect of film thickness could be eliminated or accounted for. This phase of the laboratory work was considered very important, as one of the systems may have had a strong advantage over others in the field study because it was applied more thickly. The laboratory work was conducted in the laboratories of the Bureau of Public Roads, Washington, D.C., and the findings are described in the following paragraphs.

## *Nature and Composition of the Paints*

Complete physical, chemical, and infrared spectral analyses of each paint system were made and the results are detailed in tables 5 through 13. The methods used for chemical analysis and physical examination generally were those described by the Federal Test Method for paints (*2*). Infrared spectral analysis was conducted, where necessary, to generically identify each vehicle and volatile thinner. The spectroscopic methods used have been described previously in an issue of the Official Digest (*3*). Tables 5 through 13 contain all the detailed data on composition, but some brief and amplifying remarks on these findings are given in the following paragraphs for each of the paint systems.

### System 1, coal tar

System 1 consisted of a washcoat, a primer, and a finish coat. The washcoat was a zinc chromate vinyl butyral coating; the primer was an unpigmented coal tar cutback; and the finish coat was a coal tar emulsion. Other than the washcoat, no rust-inhibitive pigment was present in this system.

**Table 7.—Analysis of Paint System 3**

| Paint System 3 | Primer coating, 3-1 | Finish coating, 3-2 |
|---|---|---|
| General properties: | | |
| Color | Brown | Orange |
| Weight per gallon .... pounds | 11.9 | 17.0 |
| Viscosity ........ Krebs units | 53 | 66 |
| Drying time: | | |
| Set to touch ........ hours | 2 | ¾ |
| Dry through ........ do | 6 | 5 |
| At dry film thickness .. mils | 2.0 | 1.9 |
| Composition, general: | | |
| Pigment ........ pct. by wt. | 52.7 | 65.1 |
| Volatile ........ do | 30.7 | 17.5 |
| Nonvolatile vehicle ..... do | 16.6 | 17.4 |
| Pigment composition: | | |
| True red lead ($Pb_3O_4$) .... pct. | 48.9 | 84.8 |
| Siliceous matter ........ do | 15.7 | [1] 9.2 |
| Zinc oxide ........ do | 11.6 | ------- |
| Iron oxide ($Fe_2O_3$) ....... do | 9.0 | ------- |
| Zinc chromate ($ZnCrO_4$) do | 7.3 | ------- |
| Nonvolatile vehicle composition: | | |
| Phthalic anhydride .. percent | 30.1 | 19.5 |
| Oil acids ........ do | 55.5 | 65.5 |
| Nature, from infrared analysis | (2) | (3) |
| General nature | (4) | (5) |

[1] Essentially talc.
[2] Medium oil-alkyd resin.
[3] Long oil-alkyd resin.
[4] Mixed pigment (red lead, siliceous matter, zinc oxide, zinc chromate, iron oxide) in alkyd vehicle.
[5] Red lead in alkyd vehicle.



785-220—65——2

**Table 8.—Analysis of Paint System 4** [1]

| Paint System 4 | Finish coating, 4–2 |
|---|---|
| General properties: | |
| Color | White |
| Weight per gallon ... pounds | 8.2 |
| Viscosity ... Krebs units | 94 |
| Drying time: | |
| Set to touch ... hours | 3 |
| Dry through ... do | 8 |
| At dry film thickness ... mils | 1.2 |
| Composition, general: | |
| Pigment ... pct. by wt. | 11.1 |
| Volatile ... do | 74.1 |
| Nonvolatile vehicle ... do | 14.8 |
| Pigment composition: | |
| Titanium dioxide ... percent | 95.3 |
| Nonvolatile vehicle, composition: | |
| Nature from infrared analysis | Vinyl chloride-vinyl acetate copolymer with perhaps some polyvinyl alcohol. |
| General nature | Titanium dioxide vinyl copolymer paint. |

[1] Washcoat 4–1 for System 4 was the same as used for System 1, as shown in table 5.

### System 2, drying oil

System 2 consisted of a primer and finish coat, both of a drying-oil type. The primer pigment was not rust inhibitive, containing only a mixture of white lead and extender pigments, and the finish coat was pigmented primarily with carbon black but also contained a small amount of red lead. Therefore, System 2 was not typically rust inhibitive.

### System 3, alkyd

System 3 consisted of a primer and finish coat of the oil alkyd type. The primer pigment contained both red lead and zinc chromate, which were substantially extended by other pigments, and the finish coat was pigmented essentially with red lead. A red lead alkyd paint such as this normally is not used as a finish coat because of its poor weathering properties. Evidently, it was incorporated in this system only upon the recommendations of the supplier and without prior consideration having been given to details of its pigmentation. This entire paint system was intended to be, and was represented by the supplier to be, a phenolic system; actually it was an alkyd system as determined by these analyses, which included infrared spectroscopy.

**Table 9.—Analysis of Paint System 5**

| Paint System 5 | Primer coating, 5–1 | Finish coating, 5–2 |
|---|---|---|
| General properties: | | |
| Color | Brown | White |
| Weight per gallon ... pounds | 13.9 | 8.5 |
| Viscosity ... Krebs units | 76 | 47 |
| Drying time: | | |
| Set to touch ... hours | 1/4 | 1 |
| Dry through ... do | 1 | 6 |
| At dry film thickness ... mils | 3.2 | 1.8 |
| Composition, general: | | |
| Pigment ... pct. by wt. | 58.7 | 19.8 |
| Volatile ... do | 22.4 | 57.4 |
| Nonvolatile vehicle ... do | 18.9 | 22.8 |
| Pigment composition: | | |
| True red lead ($Pb_3O_4$) ... pct. | 48.8 | ------ |
| Siliceous material ... do | 16.9 | ------ |
| Iron oxide ($Fe_2O_3$) ... do | 8.0 | ------ |
| Zinc oxide ($ZnO$) ... do | 1.4 | ------ |
| Titanium dioxide ($TiO_2$) do | ------ | 91.1 |
| Zinc chromate ($ZnCrO_4$) do | 10.1 | ------ |
| Nonvolatile vehicle: | | |
| Nature, from infrared analysis | (1) | (1) |
| General nature | (2) | (3) |

[1] Epoxy ester.
[2] Mixed pigment (red lead, siliceous matter, iron oxide, zinc oxide, zinc chromate) in epoxy ester vehicle.
[3] Titanium dioxide epoxy ester paint.

### System 4, vinyl

System 4 consisted of a vinyl butyral washcoat, as used in System 1, and a finish coat of titanium dioxide pigmented vinyl paint, most likely a copolymer of vinyl chloride and vinyl acetate. Except for the washcoat, the system did not contain rust-inhibitive pigmentation.

### System 5, epoxy ester

System 5, although originally represented as an epoxy, consisted of an epoxy ester primer and finish coat. The primer contained both red lead and zinc chromate extended with other pigments, and the finish coat was pigmented with titanium dioxide.

### System 6, zinc-rich inorganic

System 6, typical of many commercially available post-cured zinc-rich silicate paints, consisted of a two-package zinc and sodium silicate coating that is post cured with a solution of phosphoric acid after it has been mixed and applied.

### System 7, neoprene

System 7 consisted of a primer and finish coat, both neoprene based. The primer did not contain pigmentation, and the finish coat was pigmented essentially with carbon black. The system did not contain rust-inhibitive pigments.

### System 8, polysulfide

System 8 consisted of a primer of an unpigmented chlorinated rubber solution and a finish coat that was a two-package catalyzed liquid polysulfide rubber. This system was referred to as a rubber coating by the supplier and did not contain a rust-inhibitive primer.

### System 9, aluminum varnish

System 9 consisted of a primer, which was an extended red lead alkyd paint, the same as in System 3, and a finish coat of a ready-mixed aluminum oleoresinous varnish. This system was intended to represent the standard paint system in general use in Alaska, but it was not exactly the same because of the over-extended red lead pigment, alkyd rather than drying oil in the primer, and the fact that the finish coat was a ready-mixed aluminum paint rather than a two-package system mixed just prior to its use.

### Laboratory abrasion study

The method used for the laboratory abrasion test is that given by ASTM Method D 658–44 (*1*). It was selected because the mode of abrasive action seemed to approximate the special condition in Alaska more closely than other available abrasion tests. Briefly stated, the test is conducted on a painted panel of known film thickness that is exposed to silicon carbide abrasive particles impelled by a constant air pressure and airflow. After the coating has been worn through to the metal substrate, the weight of the abrasive particles consumed in the test is determined and divided by the original thickness of the coating. The calculated result is called an abrasion coefficient whose units are grams per mil. The larger this coefficient, the more resistant the coating to abrasive forces. More specific details of the procedure used are given in the following paragraphs and in reference *1*.

Only the finish coat of each paint system was included in this test, as it is this coat that must resist abrasion and protect the underlying primer, when present, from exposure and wear. Controlled thicknesses of each such finish coating were applied by an applicator blade to clean, tin-coated panels. After being aged for a few months to achieve adequate and maximum film curing, the coated panels were then evaluated for abrasion resistance. For comparison, auxiliary tests on fresh films were conducted on several of the more promising coatings to more closely approximate the practical field conditions, where a newly painted surface might be exposed to abrasive forces.

The coated panels were measured for film thickness by both a micrometer and a magnetic gage. Each test panel was then mounted at a 45° angle in a Gardner Grit Blast Abrasiometer, as shown in figure 19. A schematic of the entire assembly is shown in the referenced ASTM Method (*1*). The hopper, a

**Table 10.—Analysis of Paint System 6**

| Paint System 6 | Finish coating, 6–1 [1] | | Acid cure solution 6–2 |
|---|---|---|---|
| | Powder | Solution | |
| General properties: | | | |
| Color | (2) | (2) | Colorless |
| pH | ------ | ------ | 1.9 |
| Composition: | | | |
| Metallic zinc ($Zn$) ... pct. by wt. | 83.5 | ------ | ------ |
| Zinc oxide ($ZnO$) ... do | 3.5 | ------ | ------ |
| Lead ($Pb$) ... do | 12.3 | ------ | ------ |
| Siliceous material (as $SiO_2$) ... do | ------ | 21.8 | ------ |
| Sodium oxide ($Na_2O$) ... do | ------ | 7.0 | ------ |
| Phosphoric acid ($H_3PO_4$) ... do | ------ | ------ | 23.0 |
| Volatile solvent pct. by wt. | None | 62.9 | 50.5 |
| Nature of solvent from infrared analysis | ------ | Water | (3) |
| General nature | (4) | (4) | (5) |

[1] Components mixed for use in proportion of 23 poun[ds] powder to 2/3 gallon of solution.
[2] Color of cured film of mixture of powder and solution w[as] gray.
[3] Mixture of alcohol and ketone (probably isopropanol a[nd] acetone).
[4] Post-cured inorganic zinc-rich silicate coating.
[5] Phosphoric acid-alcohol-ketone curing solution.

**Table 11.—Analysis of Paint System 7**

| Paint System 7 | Primer coating, 7-1 [1] | Finish coating, 7-2 |
|---|---|---|
| General properties: | | |
| Color | Amber, clear | Black |
| Weight per gallon....lbs.. | (2) | 8.1 |
| Viscosity....Krebs units.. | (2) | 121 |
| Drying time: | | |
| Set to touch.....hours.. | (2) | 1/6 |
| Dry through.....do.... | (2) | 2 |
| At dry film thickness mils.. | (2) | 2.0 |
| Composition, general: | | |
| Pigment......pct. by wt.. | None | 17.5 |
| Volatile.............do.... | 79.2 | 63.5 |
| Nonvolatile vehicle.do.... | 20.8 | 19.0 |
| Pigment composition: | | |
| Carbon black....percent.. | .......... | [3] 88 |
| Siliceous material...do.... | .......... | [3] 12 |
| Nonvolatile vehicle composition: | | |
| Nature, from infrared analysis | (4) | (5) |
| General nature | (6) | (7) |

[1] Material received was viscous and may have lost some solvent before analysis.
[2] Too viscous for test.
[3] Approximate value.
[4] Mixture of neoprene and possibly a phenolic resin.
[5] Neoprene.
[6] Neoprene solution.
[7] Carbon black neoprene.

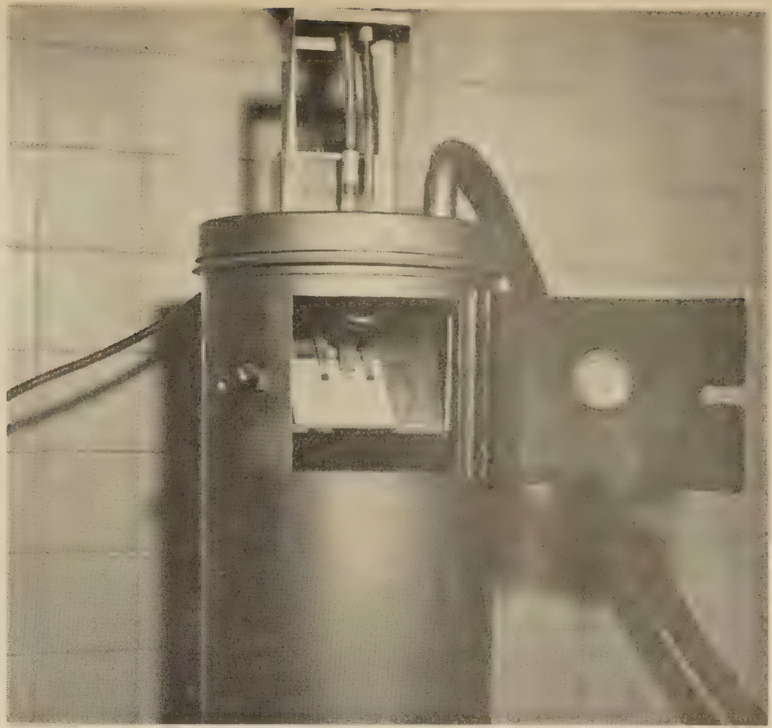


*Figure 19.—Test panel mounted in the Grit Blast Abrasiometer.*

**Table 12.—Analysis of Paint System 8**

| Paint System 8 | Primer coating, 8-1 | Finish coating, 8-2 [1] | |
|---|---|---|---|
| | | Base | Accelerator |
| General properties: | | | |
| Color | Colorless | Black | Brown |
| Weight per gallon pounds.. | 7.9 | 11.4 | 25.3 |
| Viscosity Krebs units.. | (2) | (2) | (2) |
| Drying time: | | | |
| Set to touch hours.. | 1/6 | ------ | ------ |
| Dry through hours.. | 1/2 | ------ | ------ |
| At dry film thickness mils.. | 1.3 | ------ | ------ |
| Composition, general: | | | |
| Pigment pct. by wt.. | None | 31.5 | 72.5 |
| Volatile....do.... | 73.3 | 11.1 | 5.1 |
| Nonvolatile vehicle..do.... | 26.7 | 57.4 | 22.4 |
| Pigment composition: | | | |
| Carbon black percent.. | -------- | 75.3 | ------ |
| Calcium carbonate...do.... | -------- | 22.2 | ------ |
| Lead dioxide ($PbO_2$)...do.... | -------- | ------ | 76.1 |
| Nonvolatile vehicle composition: | | | |
| Nature, from infrared analysis.. | (3) | (4) | (5) |
| General nature..... | (6) | (7) | (7) |

[1] Mixed in proportion of 10 parts base+1 part accelerator by weight.
[2] Too viscous for test.
[3] Chlorinated rubber.
[4] Liquid polysulfide resin.
[5] Styrene polymer or copolymer.
[6] Chlorinated rubber solution.
[7] General nature for finish coating 8-2 was lead oxide catalyzed liquid polysulfide rubber.

**Table 13.—Analysis of Paint System 9 [1]**

| Paint System 9 | Finish coating, 9-2 |
|---|---|
| General properties: | |
| Color | Aluminum |
| Weight per gallon......pounds.. | 7.9 |
| Viscosity..........Krebs units.. | 44 |
| Drying time: | |
| Set to touch............hours.. | 2½ |
| Dry through..............do.... | Within 40 |
| At dry film thickness....mils.. | 1.9 |
| Composition, general: | |
| Pigment...........pct. by wt.. | 11.1 |
| Volatile....................do.... | 46.1 |
| Nonvolatile vehicle........do.... | 42.8 |
| Pigment composition: | |
| Nature | Essentially aluminum. |
| Nonvolatile vehicle composition: | |
| Iodine number of extracted fatty acids | 107 |
| Nature, from infrared analysis... | Modified drying oil varnish. |
| General nature | Ready-mixed aluminum oleoresinous varnish paint. |

[1] Primer 9-1 for System 9 was the same as used for System 3, as shown in table 7.

shown in figure 20, was loaded with a weighed amount of silicon carbide, grain size 180, type GG, which previously had been sieved as required by the test method, and the abrasive particles were admitted into the test chamber. These abrasive particles were impelled by the airflow and impinged upon the coating at a 45° angle. The flow of abrasive particles was stopped at a point when the coating had been worn through to the substrate to the degree required by the method. This condition was monitored through a glass window in the test apparatus (fig. 20). A typical condition of a test coating showing the abraded spot at the conclusion of the test is shown in figure 21. The weight of abrasive particles consumed in the test was determined, and the abrasion coefficient was calculated.

The results of the laboratory abrasion tests are shown in table 14 in terms of abrasion coefficient, where each value shown represents the average of two or more determinations. Generally, agreement between individual test results was good. Because the abrasion coefficient is usually considered to be independent of film thickness, a test of a single coating thickness is normally sufficient. However, to assess this belief more fully, tests were conducted on more than one coating thickness. An attempt was also made to include or bracket the thickness of finish coating used in the field study. For System 8, however, it was not practical to conduct additional tests at the 15- to 20-mil film thicknesses used in the field study. A 1½-mil coating of this material had such an unusually high abrasive resistance, and a single test consumed so much time and abrasive material, that it was considered impracticable to conduct additional tests at the 15- to 20-mil thicknesses.

Several things are evident from the abrasion results shown in table 14. First, the abrasion coefficient was not always completely independent of the initial film thickness of the coating. Allowing for the known reproducibility and variations of the test method, it is apparent that in several systems the abrasion coefficient was higher for thicker films of the same material. Several explanations of this

*Figure 20.—Abrasiometer, door closed during test, and hopper at top.*

**Table 14.—Laboratory abrasion test results [1]**

| System coating tested | Dry film thickness [2] | Abrasion coefficient [3] |
|---|---|---|
| | *Mils* | |
| 1–3 | 2 | 5 |
| | 4 | 5 |
| | 10 | 6 |
| 2–2 | 1 | 47 |
| | 1½ | 76 |
| | 3 | 116 |
| 3–2 | 1½ | 18 |
| | 2 | 24 |
| 4–2 | 1 | 49 |
| | 2 | 56 |
| 5–2 | 1 | 40 |
| | 2 | 54 |
| 6–1, 2 | 2½ | [4] 130 |
| 7–2 | ½ | [5] 750 |
| | 1½ | 860 |
| | 2 | 990 |
| 8–2 | 1½ | 1, 155 |
| | | [6] $\gg$ 2, 100 |
| 9–2 | 1½ | 18 |
| | 3½ | 29 |

[1] ASTM Method D 658–44, *Standard Method of Test fo Abrasion Resistance of Coatings of Paint, Varnish, Lacque and Related Products with the Air Blast Abrasion Teste* Finish coatings only were applied to tin panels and unles otherwise stated were aged 4 to 5 months before test. Eac result is an average of two or more determinations.
[2] Precise film thickness measurements were made an used in the calculation of the abrasion coefficient. Fo convenience, results were grouped under the nominal fil thickness shown in the table that most closely approximate the actual thickness.
[3] Weight in grams of carborundum used per mil of fil thickness to wear through film.
[4] The reliability of the results for this particular coatin is highly doubtful because of the unusual difficulty in vi ually observing a clear-cut endpoint.
[5] Tested on fresh film, 20 hours old.
[6] Tested on fresh film, 24 hours old.

are possible; such as, greater retention of impinging abrasive particles by thicker films provide a buildup of a more abrasive-resistant armor coating; or else, greater elasticity of thicker films because of the greater retention of solvents, plasticizers; or perhaps less overall embrittlement throughout the thicker film because of more limited oxidation. These aspects were not investigated further.

Another and more important observation that can be made from table 14 data is that the abrasive resistance qualities of Systems 7 and 8 exceeded by far those of the other paints when films of comparable thickness were compared. Of these two, No. 8 exhibited the most abrasion resistance, having an abrasion coefficient of 1,155. This system exceeded the abrasion resistance of the poorest system (No. 1) by a tremendous factor (more than 200 times) and exceeded that of the so-called standard system (No. 9) by a factor of 65. This latter result might be extrapolated to mean that if System 9 lasted only 6 months in a severe abrasive atmosphere, then System 8 should theoretically last 65 times as long, or some 30 years. This extrapolation would obtain provided no other degradation conditions were operative, such as film oxidation or loss of adhesion. Obviously, other degradation factors are always present to affect the life of a paint. It is also apparent from the results shown in table 14 that, should a fresh film of the finish coating of System 8 be exposed to an abrasive environment, the abrasion resistance of such a paint would not be affected to any large extent by an early exposure and its resistance might even be increased.

In summary, the relative order of abrasion resistance in the laboratory tests, with the coatings ranked from best to poorest, is as follows:

$$8>7>6>2>4,\ 5>3,\ 9>1.$$

Both of the better Systems, 7 and 8, were rubberlike systems; that is, neoprene and polysulfide, respectively. Interestingly, System 9, which had relatively poor resistance, is similar to the standard paint normally used in Alaska, and the one that caused all the difficulty in the painting of the reconstructed highway bridges.

## *Field and Laboratory Results Compared*

Both the field and laboratory results are in good agreement that the polysulfide paint, System 8, had by far the most abrasion resistance of any of the paint systems tested. The excellent abrasive resistance of a thin film of this material in the laboratory test clearly showed that the field study conclusions were not seriously biased by the use of thicker films of this coating as compared to the other coatings studied. The excellent abrasive resistant properties of the polysulfide coating is in accord with unpublished findings of A. G. Roberts of the National Bureau of Standards These latter findings were based on both serv ice performance and a laboratory NBS Abra sive Jet Method that is similar but much mor intense than the ASTM test used in this work Details of the NBS abrasive test have bee described by Roberts (4).

The lack of agreement between the labora tory and field study findings relative to Syster 7, the neoprene paint, were somewhat di appointing. The laboratory results indicate that this System has excellent abrasic resistance. This was also true according t findings of Mr. Roberts of the Bureau Standards. Yet the field exposure study d not show, in any dramatic sense, the super ority of this coating as it did for the pol sulfide coating. A possible explanation f this discrepancy is that the neoprene syste (No. 7) used in the field did not contain rust-inhibitive primer, and that the relative thin coating used for this system may ha permitted the passage of moisture and cons quent undercutting and failure of the paint b corrosion of the underlying metal. As t backs of the test panels were not coated, unde cutting of this film by corrosion products w in fact a real possibility.

It will be recalled that the epoxy est system (No. 5) ranked next to polysulfi (No. 8) in affording protection in field exposu A, the more severe exposure. Yet the labor tory abrasion tests did not reveal any substa tial abrasive-resistant properties for th

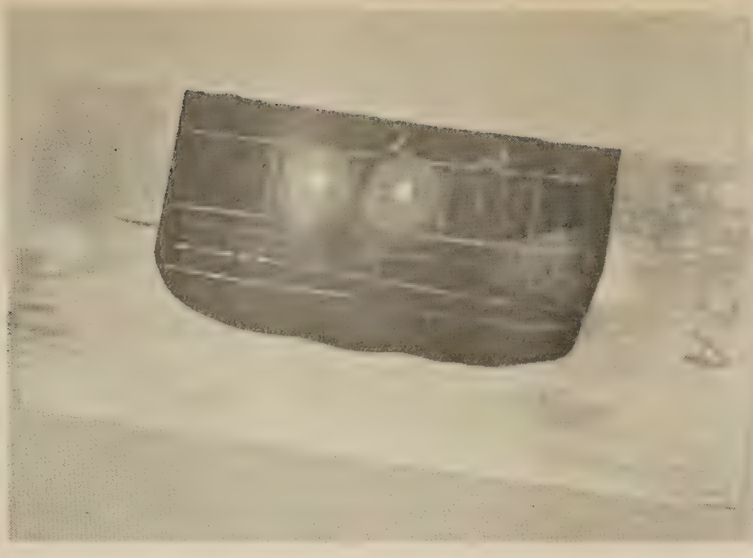


*Figure 21.—Test panel after laboratory abrasion test, showing two test areas where coating was worn through to bare metal.*

ystem. This discrepancy can perhaps be xplained as before, on the basis that System was in a favored position in field exposure A. s indicated previously in this article, System was so located in field exposure A that it eceived the least wind action of the nine oatings tested.

The evidence from both the field and labora-ory results showed that the remainder of the aints, including the presumably standard stem (No. 9), did not offer any promise for atisfactory performance for the type of xposure conditions existing in the Copper iver Delta.

## ACKNOWLEDGMENT

The early work of the Bureau of Public oads former Materials Section in the Anchorage Division office is acknowledged, especially because this office recognized and defined the problem and then planned and conducted much of the field investigation before Alaska acquired statehood. The author is also indebted to the Planning and Research Section of the Alaska Department of Highways for its efforts in completing the field study started by Public Roads and for making available pertinent data, information, and reports relating to the field exposure study. The laboratory work of Ray Cherwinski in conducting much of the chemical and infrared analysis of the paint systems is also acknowledged.

## REFERENCES

(*1*) *1964 Book of ASTM Standards: Part 21, Paint, Varnish, Lacquer, and Related Products—Tests for Formulated Products and Applied Coatings*, by the American Society for Testing and Materials.

(*2*) *Federal Test Method Standard No. 141: Paint, Varnish, Lacquer, and Related Materials; Methods of Inspection, Sampling, and Testing*, by the Federal Supply Service, General Services Administration, May 15, 1958.

(*3*) *Infrared Spectroscopy: Its Use as an Analytical Tool in the Field of Paints and Coatings*, by the Infrared Spectroscopy Committee of the Chicago Society for Paint Technology, Official Digest of the Federation of Societies for Paint Technology, vol. 33, No. 434, Part 2, March 1961.

(*4*) *Improved NBS Abrasive Jet Method for Measuring Abrasion Resistance of Coatings*, by A. G. Roberts, ASTM Bulletin No. 244, February 1960, pp. 48–51.

# Voids, Permeability, and Film Thickness Related to Asphalt Hardenin

*BY THE OFFICE O*
*RESEARCH AND DEVELOPMEN*
*BUREAU OF PUBLIC ROAD*


**Reported by [1] JOSEPH F. GOODE, Highway Research Engineer, and LAWRENCE A. LUFSEY, Highway Engineering Technician, Materials Division**



***The possibility of using air permeability measurements as control factors in the design of bituminous mixtures was investigated during the study reported in this article. Although air permeability as a possible control factor in bituminous pavement compaction also is of interest to highway engineers, this investigation was limited only to the effect of air permeability on the composition of bituminous mixtures.***

***The authors obtained no confirmation that air permeability measurements should be a factor in the design of bituminous mixtures. They did, however, determine that a criterion based on a combination of (1) the thickness of the asphalt coating on the aggregate particles and (2) the air void content of the bituminous mixture should provide a design for a more serviceable mixture. Bituminous mixtures designed according to this criterion should not be so subject to early pavement failure caused by excessive hardening of the asphalt during service.***


## *Introduction*

MOST bituminous paving engineers and technologists agree that high air permeability, high content of air voids, and thin bituminous coatings on the aggregate particles should be avoided in the design and construction of bituminous pavements. They also agree that a low permeability is required for the pavement to serve as a protective cover for the underlying courses. Many believe that the rate at which air can flow through a pavement affects the rate of hardening of the bituminous material and that high permeability is therefore undesirable. It has long been recognized that a high proportion of air voids and thin bituminous coatings can be contributing causes of an excessively high rate of hardening of the bituminous material and the subsequent early failure of the pavement surface.

Nicholson (*1*),[2] for example, stated in a 1937 AAPT paper, ". . . it follows that the less voids there are in a given paving mixture, the less the action of air on the asphalt." In another paper presented at the same meeting, Hubbard and Gollomb (*2*) concluded: ". . . Rapidity of hardening is a function of film thickness, temperature and time, . . ." These authors also concluded: ". . . To insure long life, prevent cracking and possible disintegration in an asphalt pavement or wearing course, due to hardness of the asphalt, the following . . . rules should be strictly observed: . . . Use as high a percentage of asphalt as possible without reducing stability below the minimum required to prevent displacement under traffic. In this way film thickness is increased to the maximum practical extent and air permeability decreased. . . . Compress all asphalt mixtures thoroughly so that they will be (as) impermeable to air as possible."

More recently, in a 1958 AAPT paper, Heithaus and Johnson (*3*) showed that the degree of asphalt hardening increases strongly as the initial air void content of the pavement increases. Although permeability was not measured, the authors attributed the increase in rate of hardening to the increase in air permeability, which was assumed to be a function of voids. In 1961, Goode and Owings (*4*) reported the results of a laboratory-field study showing that high content of initial air voids tends to cause rapid hardening of the asphalt and early deterioration of the pavement surface. Air permeability and asphalt film thickness were not evaluated in that study and could have been factors affecting asphalt hardening and pavement performance. The number of other research studies on the effect of air void content on pavement characteristics and performance is too large for inclusion as references in this article.

References to research on the effect of asphalt film thickness on rate of hardening are very limited. Probably the most important research was reported at the 1959 meeting of the Association of Asphalt Paving Technol gists by Campen, Smith, Erickson, and Mer (*5*). These authors established minimum a maximum values of film thickness for sati factory pavement performance.

Research employing air permeability mea urements also has been somewhat limite Ekse and Zia (*6*), at the 1953 meeting AAPT, described a method of measuring permeability to be used in control of paveme compaction. In 1955 McLaughlin and Goe (*7*) reported on the use of a special device f measuring air permeability. Results of th study showed good relationships between pe meability and air voids for a particular mi ture but none between permeability a changes in mixture properties caused by free ing and thawing. Asphalt hardening was n a variable in the study.

In a 1960 report, Ellis and Schmidt ( described a new simple device for measuri air permeability of laboratory samples and pavement in situ. Since the publication this report and a subsequent report by He and Schmidt (*9*) considerable attention h been directed toward air permeability mea urements. Kari and Santucci (*10*) present a paper on the subject at the February 19 meeting of AAPT and Warner and Moave zadeh (*11*) at the June 1964 meeting of t American Society for Testing and Materi (ASTM).

None of these referenced studies related permeability to asphalt hardening. A pa ment having high air permeability might expected to show early evidence of distr caused by rapid hardening of the aspha But, high permeability could be the result one or more of these factors: (1) insuffici asphalt in the mixture or inadequate comp tion accompanied by a high percentage of voids in the mixture; or (2) the use of an gregate gradation that has an excessively h surface area accompanied by a thin film asphalt coating on the aggregate partic The actual cause of the rapid hardening of asphalt could be high air void content, t asphalt film, or a combination of the t No data have been provided to prove that permeability per se is a factor that affects rate of asphalt hardening where aggreg gradation is a variable. All three fact might contribute to the rate of asph hardening.

[1] Presented at the annual meeting of the Association of Asphalt Paving Technologists, Philadelphia, Pa., February 15–17, 1965.

[2] References indicated by italic numbers in parentheses are listed on page 222.

**Table 1.—Properties of asphalt**

| | | |
|---|---|---|
| Original asphalt: | | |
| Penetration | 77° F., 100 g., 5 seconds | 91 |
| Saybolt furol viscosity | 275° F., second | 190 |
| Specific gravity | 77°/77° F. | 1.018 |
| Flash point | ° F. | 625 |
| Soluble in $CCl_4$ | percent | 99.86 |
| After standard oven test: | | |
| Change in weight | percent | +0.01 |
| Penetration of residue | 77° F., 100 g., 5 seconds | 87 |
| Retained penetration | percent | 96 |
| After thin film oven test: | | |
| Change in weight | percent | +0.08 |
| Penetration of residue | 77° F., 100 g., 5 seconds | 62 |
| Retained penetration | percent | 68 |
| Ductility of residue | 77° F., 5 cm./min., cm. | 250+ |

## *Objectives of Study*

The objectives for the study reported in this rticle were to determine: (1) the best proedure for measuring air permeability of laborory compacted specimens; (2) whether air ermeability was of sufficient importance to e a part of a mix design procedure; (3) the elative effect on the rate of asphalt hardening air voids, air permeability, and thickness of e asphalt film coating and particles; and ) whether the exponent 0.45 used by the ureau of Public Roads in setting up its radation Chart (*12*) was the most suitable alue for use with crushed stone aggregate.

## *Conclusions*

Based on the limited number of materials, ixtures, and aging conditions used in the udy reported in this article on resistance to phalt hardening, the following conclusions ppear justified:

- The exponent 0.45 used in establishing e BPR Gradation Chart—proved to be tisfactory for use with gravel mixtures in nother study (*12*)—is equally satisfactory r use with mixtures containing crushed stone the coarse aggregate. As in the other udy, test results indicated that 0.435 might a better exponent but the actual differences results are not considered practically gnificant.
- When measuring air permeability, the rtical face of the asphalt specimen should be aled to prevent leakage of air between the de of the specimen and the rubber membrane the testing mold. When a seal is not used, st results will be excessively high, especially hen air void contents are less than 7 percent d mixtures are coarse-grained. Paraffin is a tisfactory sealing material but should not be ed on specimens that are to be extracted for ts on recovered asphalt.
- Air permeability is a function of aggregate adation as well as of air void content. The ect of differences in gradation is much more onounced when air void content is high.
- When the mix is designed to have an air id content of about 4 or 5 percent (normal Marshall procedure), the compacted specin will have very low air permeability gardless of aggregate gradation. Therefore, e use of air permeability is not a necessary terion for use in the design of dense graded phalt mixtures.

**Table 2.—Properties of aggregate components and aggregate blends**

| Properties | Crushed granite | Natural sand No. 1 | Natural sand No. 2 | Natural sand No. 3 | Limestone dust |
|---|---|---|---|---|---|
| AGGREGATE COMPONENTS | | | | | |
| Apparent specific gravity | 2.82 | 2.67 | 2.67 | 2.66 | 2.71 |
| Bulk specific gravity | 2.75 | 2.58 | 2.58 | 2.63 | [1] 2.71 |
| Aggregate passing sieve: | | | | | |
| ½-inch ... percent | 100 | ------- | ------- | ------- | ------- |
| No. 8 ... do | 0 | 100 | ------- | ------- | ------- |
| No. 16 ... do | 0 | 63 | ------- | 100 | ------- |
| No. 30 ... do | 0 | 9 | 100 | 99 | ------- |
| No. 50 ... do | 0 | 1 | 51 | 88 | 100 |
| No. 100 ... do | 0 | 1 | 25 | 19 | 97 |
| No. 200 ... do | 0 | 0.4 | 9.1 | 1.0 | 89.1 |
| Aggregate by weight in: | | | | | |
| Blend A ... percent | 41 | 31 | 6 | 9 | 13 |
| Blend B ... do | 47 | 28 | 5 | 8 | 12 |
| Blend C ... do | 53 | 25 | 5 | 7 | 10 |
| Blend D ... do | 59 | 22 | 4 | 6 | 9 |
| Blend E ... do | 65 | 19 | 3 | 5 | 8 |
| AGGREGATE BLENDS | | | | | |
| | A | B | C | D | E |
| Aggregate passing sieve: | | | | | |
| ½-inch ... percent | 100 | 100 | 100 | 100 | 100 |
| ⅜-inch ... do | 91 | 89 | 88 | 87 | 85 |
| No. 4 ... do | 73 | 68 | 64 | 60 | 56 |
| No. 8 ... do | 59 | 53 | 47 | 41 | 35 |
| No. 16 ... do | 48 | 43 | 38 | 33 | 28 |
| No. 30 ... do | 31 | 27 | 24 | 21 | 18 |
| No. 50 ... do | 24 | 22 | 19 | 17 | 14 |
| No. 100 ... do | 16 | 15 | 13 | 11 | 10 |
| No. 200 ... do | 12.3 | 11.3 | 9.5 | 8.5 | 7.5 |
| Apparent specific gravity [2] | 2.734 | 2.743 | 2.751 | 2.760 | 2.769 |
| Effective specific gravity | 2.715 | 2.726 | 2.738 | 2.749 | 2.761 |
| Bulk specific gravity [2] | 2.669 | 2.677 | 2.685 | 2.693 | 2.702 |
| Surface area ft.[2] ... per lb. | 50.5 | 46.4 | 40.3 | 35.7 | 31.5 |

[1] Not measured but assumed to be same as apparent specific gravity.
[2] Computed from preceding data in table.

- When aggregate gradation is a variable, none of the factors—air void content, air permeability, or bituminous film thickness expressed as bitumen index—can be used singly to satisfactorily indicate a mixture's resistance to asphalt hardening. Test results indicate that use of a combined factor of a ratio of air void content to bitumen index is satisfactory for comparing resistance to asphalt hardening of different mixtures, regardless of the gradation of the aggregate blend. Air permeability is not required as a part of the combined factor.
- The Marshall method of mix design might be improved by the substitution of a maximum voids-bitumen index ratio for the presently used maximum air void content. Confirmation of this conclusion could not be established because of the limitations of the study. However, further research on the possibility of this change is desirable.

## *Materials Used in Study*

The mixtures used for the study were prepared from an 85 to 100 penetration grade of asphalt and five different combinations (blends) of aggregate—½-inch in maximum size and containing crushed stone, natural sand, and limestone dust. The selection of the blends used in this study was based on the BPR Gradation Chart, which will be discussed later. The properties of the asphalt and aggregates are given in tables 1 and 2. The gradations of the component and combined aggregates are listed in table 2, which also contains data on surface area and on apparent, effective, and bulk specific gravities.

The gradations of the five blends of aggregate are identified and illustrated on the BPR Gradation Chart of figure 1. Aggregate blend C, of which 47 percent passed the No.

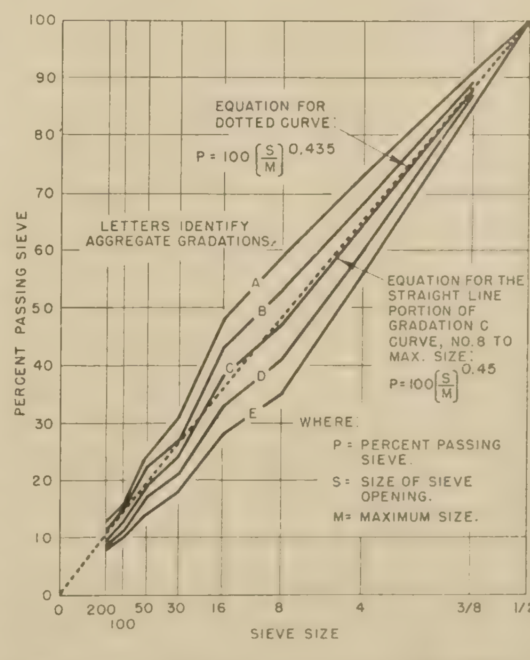



*Figure 1.—Gradation of aggregate blends used.*

8 sieve and 9.5 percent passed the No. 200 sieve, was intended to represent a maximum density blend. For the most part, blend C plots as a straight line from 0 percent passing the theoretical 0 sieve size to 100 percent passing the maximum sieve size. The aggregate gradations for blends B and A, respectively, were selected so that 6 and 12 percentage points more material would pass the No. 8 sieve than for blend C. Likewise, the aggregate gradations for blends D and E, respectively, were selected so that 6 and 12 percentage points less material would pass the No. 8 sieve than for blend C. The gradations of the parts of the aggregates that pass the No. 8 sieve, converted to 100 percent passing the sieve, were selected so that they would be approximately the same for all five blends.

### *Test Conducted*

The test specimens were 4 inches in diameter and 2½ inches high and were molded by the gyratory procedure described under the heading *Details of Study*. Initially, specimens were made with different amounts of asphalt to determine the asphalt contents that would produce approximately 4-percent air void content for each of the five blends. These results were established as the highest asphalt contents to be used in the study. Intermediate and low asphalt contents were established by an arbitrary reduction of 1 and 2 percentage points, respectively, in the amount of asphalt. The intermediate and low asphalt contents produced about 7- and 10-percent air voids, respectively, in the compacted specimen.

Two series of tests were conducted. For the first series, mixtures were prepared with each of the five blends of aggregate at their respective three predetermined asphalt contents. Three specimens of each mixture were compacted for testing for each of four curing conditions: (1) no oven curing or outdoor storage; (2) 12-day oven curing at 140° F.; (3) 63-day oven curing at 140° F.; and (4) 303-day outdoor aging.

For the second series of tests, mixtures were prepared with three aggregate blends, A, C, and E, using each of the respective three predetermined asphalt contents. Four specimens of each mixture were compacted for each of the first two curing conditions. Specimens containing the three blends were also molded using the intermediate asphalt contents and tested for the third curing condition—63-day oven curing at 140° F. In each series of tests, the test specimens for each mixture were grouped so as to obtain sets comparable as to average density and time of room storage prior to start of curing. Details of procedure used in grouping are given in the discussion on details of the study.

Prior to their being grouped, all specimens were tested for bulk specific gravity, information from which mineral voids, air voids, and mineral voids filled with asphalt were computed. The average results for both series of tests are given in table 3, together with the asphalt contents used and the computed bitumen indexes.

When curing had been completed, the specimens were tested for Marshall stability

**Table 3.—Identification and physical characteristics of compacted specimens**

| Mixture | | | Test series 1,[1] compacted specimens | | | | Test series 2,[2] compacted specimens | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Identification | Asphalt content by weight of aggregate | Bitumen index[3] (multiply by $10^{-3}$) | Bulk specific gravity | Mineral voids[4] | Air voids[5] | Mineral voids filled with asphalt | Bulk specific gravity | Mineral voids[4] | Air voids[5] | Mineral voids filled with asphalt |
| | *Percent* | | | *Percent* | *Percent* | *Percent* | | *Percent* | *Percent* | *Percent* |
| AH | 5.16 | 1.02 | 2.399 | 14.5 | 4.4 | 69 | 2.398 | 14.6 | 4.4 | 69 |
| AM | 4.16 | .82 | 2.353 | 15.4 | 7.6 | 51 | 2.360 | 15.1 | 7.3 | 52 |
| AL | 3.16 | .63 | 2.318 | 15.8 | 10.3 | 35 | 2.320 | 15.7 | 10.2 | 35 |
| BH | 4.88 | 1.05 | 2.415 | 14.0 | 4.5 | 68 | ------- | ------ | ------ | ------ |
| BM | 3.88 | .84 | 2.368 | 14.8 | 7.7 | 48 | ------- | ------ | ------ | ------ |
| BL | 2.88 | .62 | 2.332 | 15.3 | 10.4 | 32 | ------- | ------ | ------ | ------ |
| CH | 5.03 | 1.25 | 2.427 | 13.9 | 4.2 | 70 | 2.428, | 13.9 | 4.1 | 70 |
| CM | 4.03 | 1.00 | 2.380 | 14.8 | 7.4 | 50 | 2.381 | 14.7 | 7.3 | 50 |
| CL | 3.03 | .75 | 2.342 | 15.3 | 10.2 | 34 | 2.342 | 15.3 | 10.2 | 34 |
| DH | 5.08 | 1.42 | 2.435 | 14.0 | 4.2 | 70 | ------- | ------ | ------ | ------ |
| DM | 4.08 | 1.14 | 2.390 | 14.7 | 7.3 | 51 | ------- | ------ | ------ | ------ |
| DL | 3.08 | .86 | 2.348 | 15.4 | 10.2 | 34 | ------- | ------ | ------ | ------ |
| EH | 5.35 | 1.70 | 2.435 | 14.5 | 4.2 | 71 | 2.439 | 14.3 | 4.0 | 72 |
| EM | 4.35 | 1.38 | 2.394 | 15.1 | 7.1 | 53 | 2.398 | 15.0 | 7.0 | 53 |
| EL | 3.35 | 1.06 | 2.346 | 16.0 | 10.3 | 35 | 2.355 | 15.7 | 10.0 | 36 |

[1] Average of 12 specimens for each mixture.
[2] Average of 12 specimens for AM, CM, and EM; and 8 specimens for each of other mixtures.
[3] Pounds of asphalt per $ft.^2$ surface area of aggregate determined by California procedure.
[4] Based on bulk specific gravity of aggregate.
[5] Based on effective specific gravity of aggregate.

and extracted for tests on recovered asphalt, which included penetration, softening point, and ductility. Before the Marshall tests were made, and a few times before they were cured, selected groups of specimens were tested for air permeability by three procedures, which will be described later.

### *Exponent Used for BPR Gradation Chart*

The exponent 0.45 used in establishing the horizontal scale of the BPR Gradation Chart was based on the findings of L. W. Nijboer and the research study on gravel mixtures by Goode and Lufsey described in a 1962 AAPT paper (*12*). Goode and Lufsey's research study suggested that 0.435 might be a better exponent but the authors agreed that it was not significantly different from 0.45, the exponent based on the earlier work of Nijboer; therefore, they decided to use the latter as a basis for the gradation chart.

Figure 1, which shows the aggregate gradations of the blends used in the study reported here on the BPR Gradation Chart, includes a dotted flat curve indicating a maximum density gradation based on the use of 0.435 as the exponent. This curve would have been a straight line had the entire figure been based on this exponent rather than on 0.45. Aggregate blend C was intended to represent maximum density. Its plotted gradation was for the most part a straight line connecting the point at 100 percent passing the maximum size and the point at 0 percent passing the theoretical 0 sieve size.

Aggregate blends B and D were intended to have appreciably lower densities, but the mineral voids curves of figure 2 show that they did not. Blend B was as dense as blend C, and D was almost as dense. Therefore, the best equation for obtaining data on maximum density appears to be one that can be plotted between the curves for blends C and B bu closer to the curve for blend C and one havin 0.435 as the exponent—such as is shown by th dotted curve in figure 1. Thus, there is a indication confirming that 0.435 might be better exponent than 0.45 for determinin maximum density for mixtures containi ½-inch maximum size crushed stone. How ever, it is doubtful that the slight difference the results obtained by use of the two exp nents is sufficient to justify revision of th BPR Gradation Chart.

### *Details for Air Permeability Tes*

Figures 3 and 4 are, respectively, a phot graph and a schematic drawing of the ai permeability apparatus used in the stu reported here. The apparatus and the meth of its use differ from that developed by E and Schmidt (*8*) in that airflow is created use of a vacuum of an air pressure line and th rate of airflow is determined by use of one three different size flowmeters. Calibrati

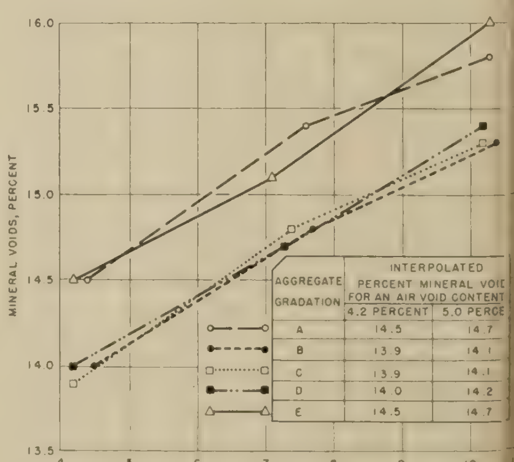
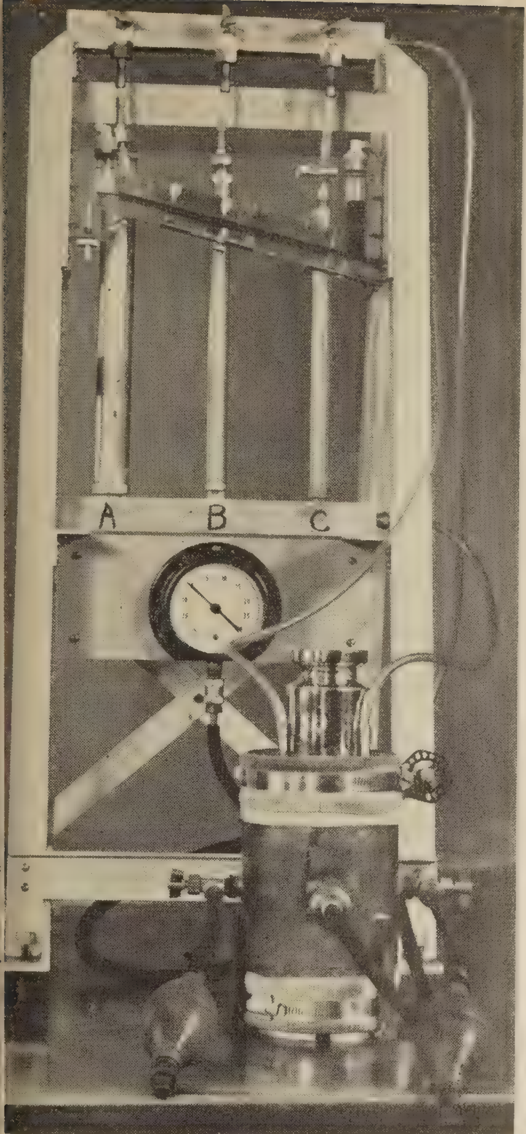



*Figure 2.—Relation between air voids a mineral voids for specimens of st series 1.*



igure 3.—*Apparatus used to measure air permeability.*

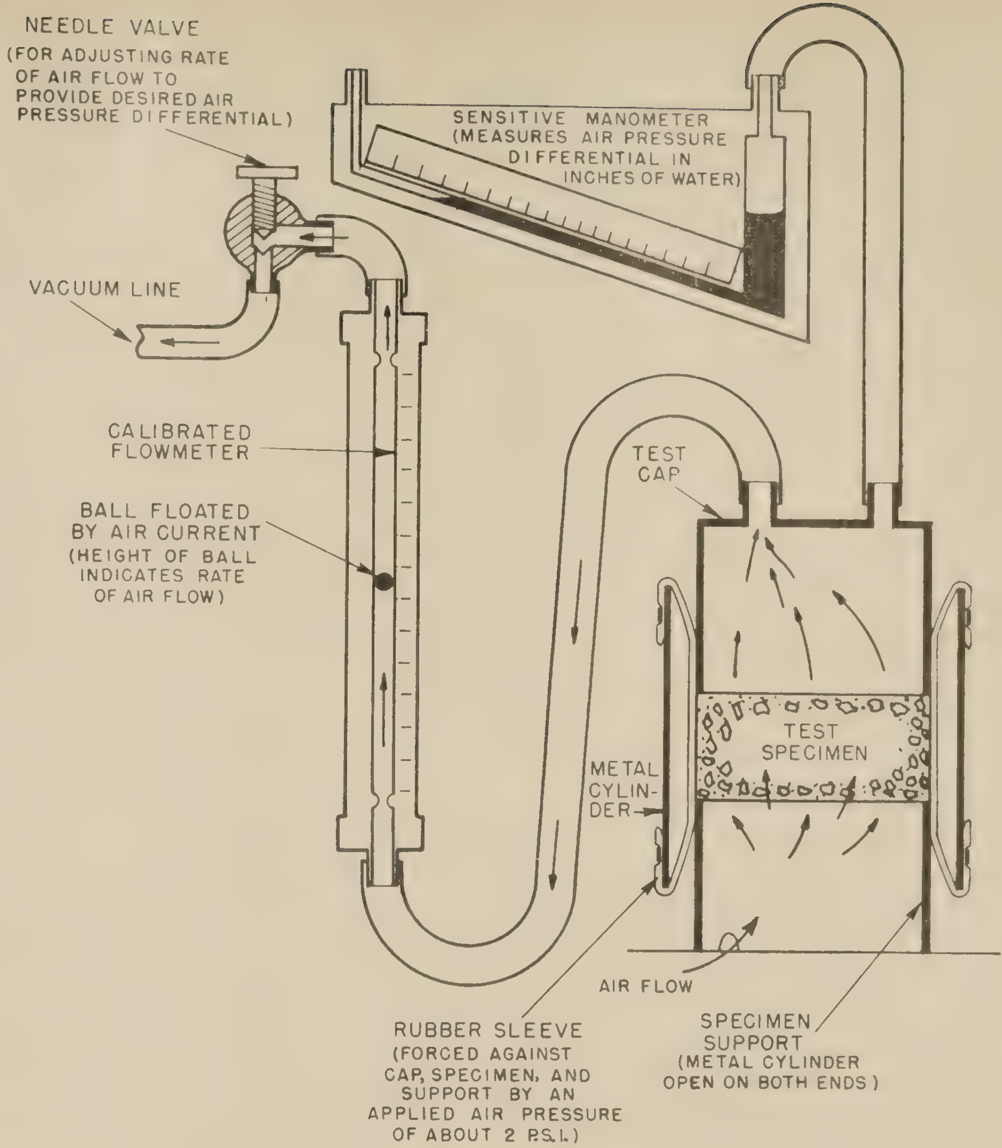



*Figure 4.—Air permeability test.*

ıarts are used for converting flowmeter read- gs to rates of airflow in millimeters per inute. The principal advantage of using ıis apparatus over others employing a falling ɛad of water for creating airflow is that ming of flow is not required while maintain- g a consistent pressure differential.

Preliminary work with the Public Roads ›paratus showed that to obtain reliable easurements of permeability, a seal on the ›rtical face of the test specimen was re- ıired most of the time to prevent leakage air between the side of the specimen and the ıbber membrane. Paraffin, applied hot ith a brush, was an effective sealing material.

Paraffin was used as a seal for the uncured ıd oven cured specimens of the first series of sts. Before the stability tests were made e specimens were warmed slightly in an ren and most of the paraffin was removed. an effort to prevent paraffin contamination, e outer one-fourth of an inch of the specimen as discarded before the extraction. Un- rtunately, as discovered later, sufficient raffin contamination remained to make the sults of tests on these extracted asphalts eless. Therefore, the second series of tests as conducted. Wet clay was tried as a seal on several of the remaining specimens, and a limited amount of data was obtained from specimens on which no seal had been used.

Tables 4, 5, and 6 contain the permeability results for the tests made when the vertical faces of the specimens were sealed with paraffin, with wet clay, and unsealed, respectively. Rates of airflow were determined for pressure differentials of ½, 1, and 2 inches of water. Permeability in fundamental units was computed, as described in the details of the study, and averaged.

Table 7 contains a comparison of the effects of no seal, the paraffin seal, and the wet clay seal on the determined permeability for three different aggregate blends and at three levels of air void content. The specimens sealed with wet clay had less permeability than those sealed by paraffin, although at the high air void content of 10 percent the percentage differences between permeability were small. When wet clay was used, indications that moisture had entered the voids of the specimens were noted for several specimens during the testing. Therefore, it is believed that the results indicating lower permeability were caused by moisture blocking some of the capillaries of the specimen and not that wet clay was a more effective seal than paraffin.

All the specimens not sealed had higher permeabilities than those sealed with the paraffin. The percentage differences in permeability were slight at 10-percent air voids but very pronounced at 4-percent voids, especially for blend E, the coarsest of the three.

Results of the tests, which are given in table 7, show that the vertical face of the specimen requires sealing for reliable values of permeability. Paraffin is an effective sealing material but should not be used when the specimen is to be extracted for tests on the recovered asphalt. There probably are other sealing materials that would be suitable for extraction tests as well as permeability tests, but this was not investigated. The particular wet clay used in the study reported here did not appear to be a suitable sealing agent because of the possible blocking of airflow channels by moisture.

## *Effect of Gradation and Air Voids on Permeability*

The relationships obtained between air void content, gradation of aggregate blend, and air permeability when paraffin was used

Table 4.—Air permeability results when paraffin was used to seal vertical face of specimen

| Mix identification | Rate of airflow [1] for specimens from test series 1, for pressure differential of— | | | | | | | | | Computed average permeability [2] for pressure differential of— | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | ½ inch of water | | | 1 inch of water | | | 2 inches of water | | | ½ inch of water | 1 inch of water | 2 inches of water | Average |
| | N-1 [3] | P-1 [4] | Average | N-1 [3] | P-1 [3] | Average | N-1 [3] | P-1 [4] | Average | | | | |
| | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *K* | *K* | *K* | *K* |
| AH | 2 | 2 | 2 | 6 | 5 | 5.5 | 15 | 12 | 13.5 | 4 | 5.5 | 6.5 | 5.5 |
| AM | 27 | 29 | 28 | 54 | 53 | 54 | 101 | 108 | 104 | 55 | 53 | 51 | 53 |
| AL | 53 | 52 | 52 | 108 | 105 | 106 | 213 | 207 | 210 | 101 | 103 | 102 | 102 |
| BH | 3 | 3 | 3 | 7 | 6 | 6.5 | 16 | 13 | 14.5 | 6 | 6.5 | 7 | 6.5 |
| BM | 35 | 38 | 36 | 67 | 73 | 70 | 129 | 143 | 136 | 70 | 68 | 66 | 68 |
| BL | 72 | 68 | 70 | 143 | 140 | 142 | 278 | 270 | 274 | 136 | 138 | 133 | 136 |
| CH | 3 | 3 | 3 | 7 | 5 | 6 | 21 | 10 | 16 | 6 | 6 | 8 | 6.5 |
| CM | 45 | 42 | 44 | 86 | 78 | 82 | 171 | 157 | 164 | 86 | 80 | 80 | 82 |
| CL | 125 | 127 | 126 | 238 | 248 | 243 | 502 | 515 | 508 | 245 | 237 | 247 | 243 |
| DH | 5 | 3 | 4 | 9 | 4 | 6.5 | 25 | 8 | 16.5 | 8 | 6.5 | 8 | 7.5 |
| DM | 59 | 57 | 58 | 111 | 113 | 112 | 215 | 222 | 218 | 113 | 109 | 106 | 109 |
| DL | 217 | 217 | 217 | 455 | 447 | 451 | 903 | 890 | 896 | 423 | 439 | 436 | 433 |
| EH | 7 | 2 | 4.5 | 14 | 4 | 9 | 31 | 5 | 18 | 9 | 9 | 9 | 9 |
| EM | 72 | 63 | 68 | 139 | 113 | 126 | 240 | 203 | 222 | 132 | 123 | 108 | 121 |
| EL | 571 | 525 | 548 | 1,033 | 919 | 976 | 1,812 | 1,719 | 1,766 | 1,068 | 951 | 860 | 960 |

[1] A 2-p.s.i. pressure was used to force rubber membrane against side of specimen. Vacuum was used to produce airflow.
[2] Permeability per cm.², cm.³ per sec., *K* (multiply by $10^{-10}$).
[3] Uncured group of specimens.
Oven cured group of specimens, after 12 days at 140° F.

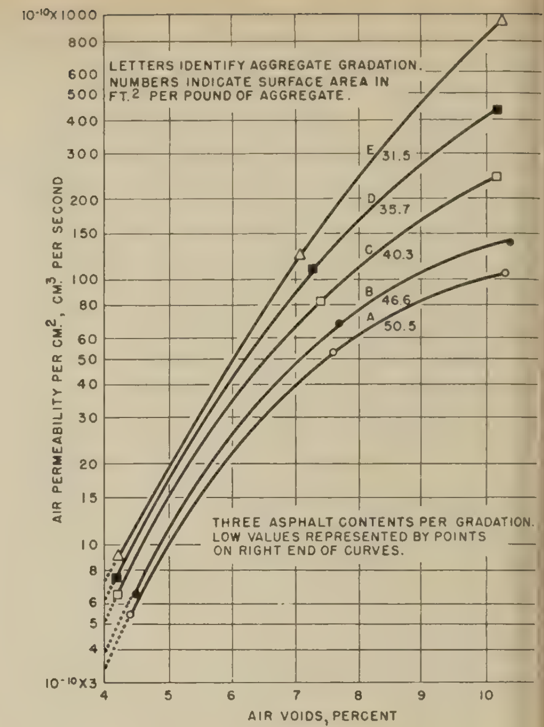



***Figure 5.—Relation between air voids an[d] permeability.***

as a seal are shown in figure 5. The different contents of air voids were obtained by the use of different asphalt contents rather than different compactive efforts. All specimens were compacted by a gyratory compaction procedure that produced a density close to what would be obtained from the 50-blow Marshall procedure.

In confirmation of the findings of Hein and Schmidt (*9*), data in figure 5 show very definitely that permeability is not necessarily proportional to air voids when aggregate gradation is a variable. As noted previously, the order of aggregate gradation of the blends in sequence was from finest to coarsest A, B, C, D, and E. The magnitude of permeability at all void contents followed the same sequence (fig. 5). For example, at 7-percent air voids the permeabilities for the finest to the coarsest gradation were, respectively 40, 48, 65, 90, and 115 each $\times 10^{-10}$ cm.³ per sec.

The effect of aggregate gradation was much more pronounced when air void content was high than when low; for example, at 10-percent air void content, specimens made from aggregate blend E had a permeability of $800 \times 10^{-10}$ cm.³ per sec., which was 8 times that for the specimens containing aggregate A; at 4-percent air void content, the specimens made with aggregate blend E had a very low permeability, $7 \times 10^{-10}$ cm.³ per sec., which was only twice that for specimens containing aggregate blend A. Because of the low permeabilities that might be expected when air void content is 4 or 5 percent, regardless of aggregate gradation, no need is apparent for an air permeability requirement in a mix design procedure for a dense-graded mixture in which the asphalt content is estal[b]lished to produce about 4 percent of air void[s].

## *Bitumen Index*

Bitumen index as determined by t[he] California procedure was selected to represe[nt] film thickness. An index was employed [to] avoid an implication that all particles a[re] coated with a uniform thickness of bitumino[us] material. If it is desirable to convert bitume[n] index to film thickness in microns, as em[ployed] by Campen and others (*5*), this ca[n] be done by multiplying the bitumen ind[ex] by 4,870. These authors indicated that t[he] film thickness should be at least 6 microns [to] obtain satisfactory pavement performanc[e]. This corresponds to a bitumen index of [at] least $1.23 \times 10^{-3}$. From test data present[ed]

Table 5.—Air permeability when wet clay was used to seal vertical face of specimen

| Mix identification [1] | Test series 1, specimens [2] from group S-1 | | | | | | | | | | | | | | Test series 2, specimens [3] from group N-2 | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Airflow rate [4] for pressure differential of— | | | | | | | | | Air voids | Computed permeability [5] | | | | Airflow rate [4] for pressure differential of— | | | Air voids | Computed permeability [5] | | | |
| | ½ inch of water | | | 1 inch of water | | | 2 inches of water | | | | For pressure differential of— | | | Average | | | | | For pressure differential of— | | | Average |
| | Prior to storage | After storage | Average | Prior to storage | After storage | Average | Prior to storage | After storage | Average | | ½ inch of water | 1 inch of water | 2 inches of water | | ½ inch of water | 1 inch of water | 2 inches of water | | ½ inch of water | 1 inch of water | 2 inches of water | |
| | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Percent* | *K* | *K* | *K* | *K* | *Ml./min.* | *Ml./min.* | *Ml./min.* | *Percent* | *K* | *K* | *K* | *K* |
| AM | 15 | 12 | 14 | 30 | 22 | 26 | 59 | 42 | 50 | 7.6 | 27 | 25 | 24 | 25 | 10 | 22 | 43 | 7.3 | 19 | 21 | 21 | 20 |
| AL | 42 | 46 | 44 | 80 | 94 | 87 | 159 | 192 | 176 | 10.3 | 86 | 85 | 86 | 86 | 42 | 84 | 171 | 10.2 | 82 | 82 | 83 | 82 |
| CM | 14 | 23 | 18 | 27 | 44 | 36 | 52 | 84 | 68 | 7.4 | 35 | 35 | 33 | 34 | 34 | 63 | 123 | 7.3 | 66 | 61 | 60 | 62 |
| CL | 107 | 89 | 98 | 200 | 183 | 192 | 399 | 378 | 388 | 10.2 | 191 | 187 | 189 | 189 | 105 | 204 | 409 | 10.2 | 205 | 199 | 199 | 201 |
| EM | 13 | ----- | 13 | 19 | ----- | 19 | 33 | ----- | 33 | 7.1 | 25 | 19 | 16 | 20 | 32 | 58 | 100 | 7.0 | 62 | 56 | 49 | 56 |
| EL | 430 | 428 | 429 | 741 | 781 | 761 | 1,329 | 1,315 | 1,322 | 10.3 | 836 | 741 | 644 | 740 | 541 | 910 | 1,589 | 10.0 | 1,054 | 886 | 774 | 905 |

[1] Specimens for mixtures AH, CH, and EH were tested also. Their rates of airflow were less than 2 ml. per minute, the lowest rate that can be measured with the apparatus, and permeabilities were less than $1 \times 10^{-10}$.
[2] Specimens from outdoor storage group.
[3] Uncured group of specimens.
[4] A 3-p.s.i. pressure was used to force rubber membrane against vertical face of specimen. Pressure was used to produce airflow for S-1 specimens and vacuum for N-2 specimens.
[5] Air permeability per cm.², cm.³ per sec., *K* (multiply by $10^{-10}$).

**[Ta]ble 6.—Air permeability when no seal was used for vertical face of specimen [1]**

| Mix identification | Rate of airflow,[2] for pressure differential of 1 inch of water | Computed permeability [3] |
|---|---|---|
| | *Ml. per min.* | *K* |
| AH | 22 | 21 |
| AM | 61 | 59 |
| AL | 130 | 127 |
| CH | 119 | 116 |
| CM | 169 | 165 |
| CL | 302 | 294 |
| EH | 436 | 425 |
| EM | 676 | 658 |
| EL | 1, 547 | 1, 507 |

[1] Test series 2, specimens from group P-2 oven cured 12 [day]s at 140° F.
[2] A 3-p.s.i. air pressure was used to force rubber membrane [aga]inst vertical face of specimen. Pressure was used to [re]duce airflow.
[3] Permeability per cm.², cm.³ per sec., $K$ (multiply by [10⁻]¹⁰).

**[Ta]ble 7.—Effect on measured permeability when the vertical face of specimen was not [s]ealed with paraffin, or sealed with clay**

| Air voids for aggregate blends | Interpolated permeability [1] | | |
|---|---|---|---|
| | Not sealed | Sealed with paraffin [2] | Sealed with wet clay [3] |
| *Percent* | *K* | *K* | *K* |
| A: | | | |
| 4.0 | 18 | 3 | <1 |
| 7.0 | 54 | 40 | 19 |
| 10.0 | 122 | 98 | 75 |
| C: | | | |
| 4.0 | 114 | 5 | <1 |
| 7.0 | 160 | 67 | 40 |
| 10.0 | 280 | 230 | 175 |
| E: | | | |
| 4.0 | 425 | 7 | <1 |
| 7.0 | 658 | 115 | 37 |
| 10.0 | 1, 507 | 800 | 710 |

[1] From data of tables 3 through 6. Permeability per c[m.]², cm.³ per sec., $K$ (multiply by $10^{-10}$).
[2] Tests had shown paraffin to be an effective seal.
[3] Wet clay appears to be effective in sealing vertical face of s[pe]cimen, but it is indicated that moisture from the wet c[la]y enters and tends to seal capillaries of the specimen, [the]reby causing decrease in permeability.

**[Ta]ble 8.—Surface area of aggregate and [b]itumen index compared with film [t]hickness**

| Aggregate | | Mixture [1] | | | |
|---|---|---|---|---|---|
| Blend | Surface area, ft.² per lb. of aggregate | Identification | Asphalt content,[2] percent | Bitumen index | Film thickness, microns |
| A | 50.5 | AH | 5.16 | 1.02×10⁻³ | [3] 5.0 |
| B | 46.4 | BH | 4.88 | 1.05×10⁻³ | [3] 5.1 |
| C | 40.3 | CH | 5.03 | 1.25×10⁻³ | 6.1 |
| D | 35.7 | DH | 5.08 | 1.42×10⁻³ | 6.9 |
| E | 31.5 | EH | 5.35 | 1.70×10⁻³ | 8.3 |

[1] At asphalt content producing about 4-percent air voids [in c]ompacted specimens.
[2] Aggregate basis.
[3] Film thickness less than 6 microns.

in table 8, a comparison can be made of the bitumen index and film thickness for each of the five aggregate blends at which the asphalt contents produced about 4-percent air void content in the specimens. Aggregate blends A and B would be considered unsatisfactory by the 6-micron thickness criterion. These two blends would normally be considered unsuitable for asphaltic concrete surface course mixtures because of their high dust contents, 12.3 and 11.3 percent, respectively. When blend C, having the most dense gradation of blends, was used for the test specimens, the film was approximately at the 6-micron thickness criterion.

## *Effect of Mix Properties on Asphalt Hardening*

The data used in studying the effect of mix properties on asphalt hardening are summarized in tables 9 and 10. From the data for the uncured and cured specimens, three methods for rating asphalt hardening are suggested: (1) percentage of retained penetration, (2) percentage gain in Marshall stability, and (3) change in softening point temperature. The latter method proved to be less suitable than the other two and was not used. The first two methods were used in determining the data for figures 6, 7, 8, and 9 to show the effect of the variables on the degree of asphalt hardening in the molded specimens subjected to 12 days of oven curing at 140° F. These figures were developed from the data obtained from the second series of tests in which only three aggregate blends were used, A, C, and E.

The relationships between bitumen index per se and degree of asphalt hardening for each of the three blends at three asphalt contents are shown in figure 6. The asphalt contents are not identified on the figure but the higher asphalt contents are represented by the points on the right end of the curves. In the upper part of the figure, retained penetration indicates resistance to hardening and in the lower part, gain in Marshall stability.

The curves at the top or bottom of figure 6 show, for a particular blend, a definite trend for asphalt hardening to decrease as bitumen index increases; the retained penetration increased and the gain in Marshall stability decreased. These data indicate but do not prove that bitumen index is an important factor affecting asphalt hardening. The fact that the curves for the three aggregate blends are widely separated indicates that at least one other factor also affects the rate of asphalt hardening.

In figure 7 the retained penetration and gain in Marshall stability have been plotted against air permeability instead of the bitumen index (fig. 6). Data for the specimens having high asphalt content are on the left. The curves in figure 7 illustrate the definite trend, for a particular aggregate blend, which indicates that asphalt hardening increases as permeability increases. The wide spread between the curves again indicates that at least one other factor affects the rate of asphalt hardening.

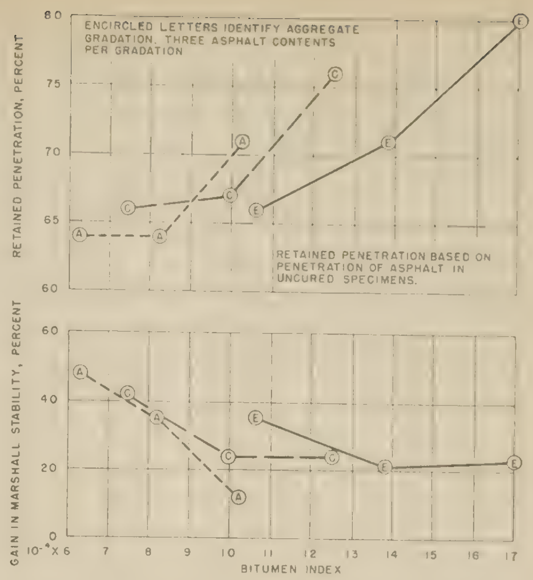



*Figure 6.—Effect of bitumen index on asphalt hardening after 12 days of oven curing at 140° F.*

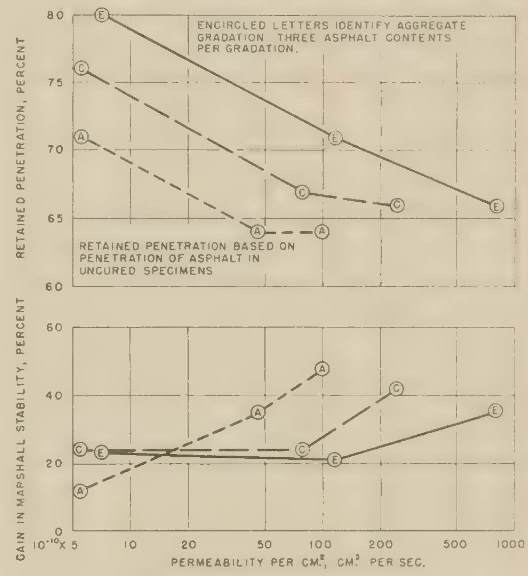



*Figure 7.—Effect of permeability on asphalt hardening after 12 days of oven curing at 140° F.*

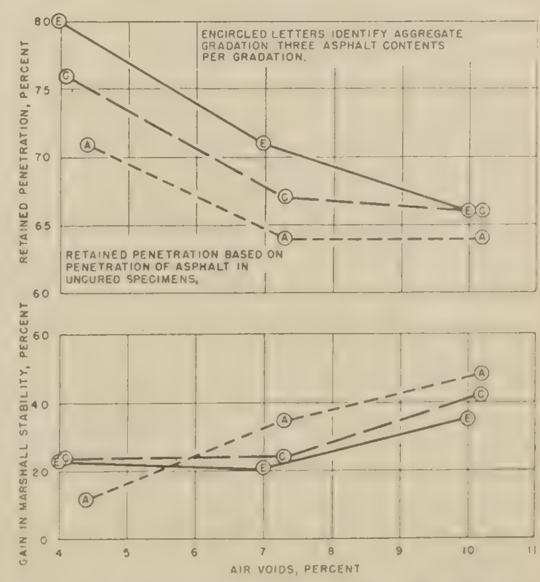



*Figure 8.—Effect of air voids on asphalt hardening after 12 days of oven curing at 140° F.*

**Table 9.—Results of test on specimens aged outdoors, test series 1**

| Mix identification | Air voids | Bitumen index (multiply by $10^{-3}$) | Voids-bitumen index ratio [1] | Uncured specimens [2] | | | Specimens aged outdoors 303 days [5] | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Permeability [3] | Marshall [4] | | Marshall [4] | | | Recovered asphalt [5] | |
| | | | | | Stability | Flow | Stability | Flow | Increase in stability | Penetration | Softening point |
| | *Percent* | | | *K* | *Pounds* | | *Pounds* | | *Percent* | | *° F.* |
| AH | 4.4 | 1.02 | 4.3 | 5.5 | 1,750 | 8 | 2,510 | 12 | 43 | 40 | 131 |
| BH | 4.5 | 1.05 | 4.3 | 6.5 | 1,770 | 9 | 2,530 | 11 | 43 | 40 | 132 |
| CH | 4.2 | 1.25 | 3.4 | 6.5 | 1,650 | 10 | 2,160 | 11 | 31 | ------ | 132 |
| DH | 4.2 | 1.42 | 3.0 | 7.5 | 1,580 | 11 | 2,010 | 12 | 27 | 40 | 131 |
| EH | 4.2 | 1.70 | 2.5 | 9 | 1,300 | 13 | 1,700 | 14 | 31 | 44 | 128 |
| AM | 7.6 | .82 | 9.3 | 53 | 1,470 | 8 | 2,200 | 11 | 50 | 38 | 132 |
| BM | 7.7 | .84 | 9.2 | 68 | 1,400 | 8 | 2,250 | 11 | 61 | 40 | 130 |
| CM | 7.4 | 1.00 | 7.4 | 82 | 1,300 | 8 | 2,180 | 11 | 68 | 39 | ------- |
| DM | 7.3 | 1.14 | 6.4 | 109 | 1,300 | 8 | 1,920 | 11 | 48 | 38 | 130 |
| EM | 7.1 | 1.38 | 5.1 | 121 | 1,230 | 9 | 1,760 | 12 | 43 | 39 | 131 |
| AL | 10.3 | .63 | 16.3 | 102 | 870 | 7 | 1,590 | 10 | 83 | 38 | 129 |
| BL | 10.4 | .62 | 16.8 | 136 | 960 | 6 | 1,540 | 10 | 60 | 38 | 129 |
| CL | 10.2 | .75 | 13.6 | 243 | 1,060 | 7 | 1,680 | 9 | 58 | 38 | 131 |
| DL | 10.2 | .86 | 11.9 | 433 | 1,100 | 7 | 1,680 | 10 | 53 | 37 | 132 |
| EL | 10.3 | 1.06 | 9.7 | 960 | 940 | 7 | 1,690 | 11 | 80 | 37 | 132 |

[1] Percent air voids divided by ($10^3 \times$ bitumen index).
[2] Precoating with paraffin contaminated the asphalt and the results of tests on recovered asphalt were discarded. Marshall stability also could have been affected.
[3] Permeability per $cm.^2$, $cm.^3$ per sec., $K$ (multiply by $10^{-10}$).
[4] Averages of three tests.
[5] Only one recovery for each mixture, average ductility was at least 230 cm.

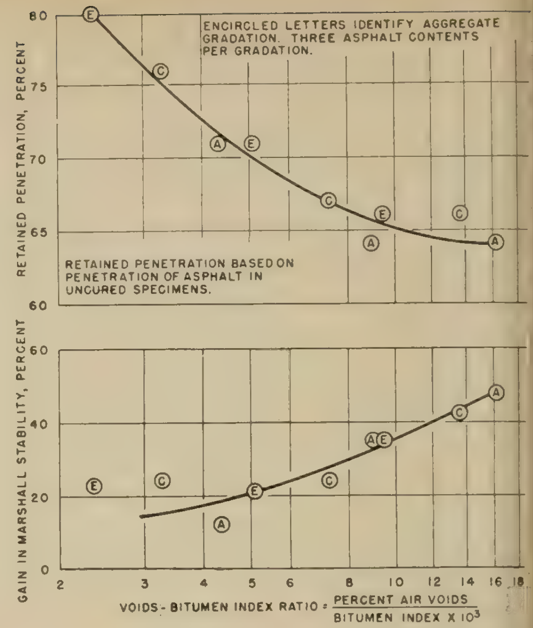



*Figure 9.—Effect of voids-bitumen ind ratio on asphalt hardening after 12 days oven curing at 140° F.*

The effect of air voids per se on the degree of asphalt hardening for each of the three aggregate blends is illustrated in figure 8. The specimens having high asphalt content are represented by the points to the left. The curves in the figure show that the effect of air voids on hardening was very similar to the effect on permeability. These data also indicate that at least one other factor affects the rate of asphalt hardening.

When aggregate gradation is a variable, as shown in figures 6, 7, and 8, none of the factors alone—bitumen index, permeability, or air void content—can be used to properly evaluate a compacted mixture as to its resistance to hardening of the asphalt during service. The results of the tests indicated that a combination of two or of all three factors was needed for a satisfactory evaluation. Several combined factors were tried. The one that appears to be most suitable on the basis of the data of the study consists of a simple ratio of the percentage of air void content divided by the product of the bitumen index $\times$ $10^3$.

Voids-bitumen index ratios for the mixtures containing the three blends of aggregate used in test series No. 2 are given in table 10. These ratios were plotted against the retained penetration and gain in Marshall stability, as shown in figure 9. The use of the ratio, which combines the void content and bitumen index factors, produces a single curve for all blends. The maximum deviations from the plotted curves were only 2 percentage points for retained penetration, and about 10 percentage points for gain in Marshall stability. These deviations are considered within or reasonal close to the precision of the tests. For the penetration test, accepted results of repea bility and reproducibility are 4- and 8-perce respectively. Attempts were made to inclu permeability in a combined factor but n logical formula was found that would prove a better fit of the plotted data than that show in figure 9.

In figure 10 the voids-bitumen index rat is used to compare asphalt hardening af 303 days of outdoor exposure for the 15 m tures of the first series of tests. As mention previously, it had become necessary to disca the results of tests on the asphalt recove from the uncured specimens because of e paraffin contamination of the specimens. T

**Table 10.—Results of tests on oven cured specimens, test series 2**

| Mix identification | Air voids | Bitumen index (multiply by $10^{-3}$) | Voids-bitumen index ratio [1] | Permeability [2] | Curing at 140° F. | Marshall [3] | | | Recovered asphalt [4] | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Stability | Flow | Increase in stability | Penetration | Retained penetration | Softening point |
| | *Percent* | | | *K* | *Days* | *Pounds* | | *Percent* | | *Percent* | *° F.* |
| AH | 4.4 | 1.02 | 4.3 | 5.5 | 0 | 2,200 | 10 | -------- | 66 | -------- | 120 |
| | | | | | 12 | 2,470 | 10 | 12 | 47 | 71 | 127 |
| CH | 4.1 | 1.25 | 3.3 | 5.5 | 0 | 1,920 | 10 | -------- | 66 | -------- | 120 |
| | | | | | 12 | 2,380 | 10 | 24 | 50 | 76 | 128 |
| EH | 4.0 | 1.70 | 2.4 | 7 | 0 | 1,440 | 10 | -------- | 71 | -------- | 120 |
| | | | | | 12 | 1,770 | 11 | 23 | 57 | 80 | 123 |
| AM | 7.3 | .82 | 8.9 | 46 | 0 | 1,680 | 8 | -------- | 64 | -------- | 121 |
| | | | | | 12 | 2,270 | 8 | 35 | 41 | 64 | 130 |
| | | | | | 63 | 2,540 | 10 | 51 | 32 | 50 | 136 |
| CM | 7.3 | 1.00 | 7.3 | 78 | 0 | 1,570 | 8 | -------- | 66 | -------- | 121 |
| | | | | | 12 | 1,950 | 9 | 24 | 44 | 67 | 130 |
| | | | | | 63 | 2,450 | 11 | 56 | 31 | 47 | 136 |
| EM | 7.0 | 1.38 | 5.1 | 115 | 0 | 1,460 | 9 | -------- | 66 | -------- | 121 |
| | | | | | 12 | 1,760 | 10 | 21 | 47 | 71 | 127 |
| | | | | | 63 | 2,160 | 12 | 48 | 34 | 52 | 135 |
| AL | 10.2 | .63 | 16.2 | 100 | 0 | 1,110 | 8 | -------- | 64 | -------- | 121 |
| | | | | | 12 | 1,640 | 9 | 48 | 41 | 64 | 129 |
| CL | 10.2 | .75 | 13.6 | 243 | 0 | 1,100 | 6 | -------- | 64 | -------- | 120 |
| | | | | | 12 | 1,560 | 9 | 42 | 42 | 66 | 131 |
| EL | 10.0 | 1.06 | 9.4 | 800 | 0 | 1,180 | 8 | -------- | 62 | -------- | 121 |
| | | | | | 12 | 1,590 | 8 | 35 | 41 | 66 | 127 |

[1] Percent air voids divided by ($10^3 \times$ bitumen index).
[2] Permeability per $cm.^2$, $cm.^3$ per sec., $K$ (multiply by $10^{-10}$).
[3] Averages of four tests.
[4] Averages of tests from two recoveries; average ductility was at least 210 cm.

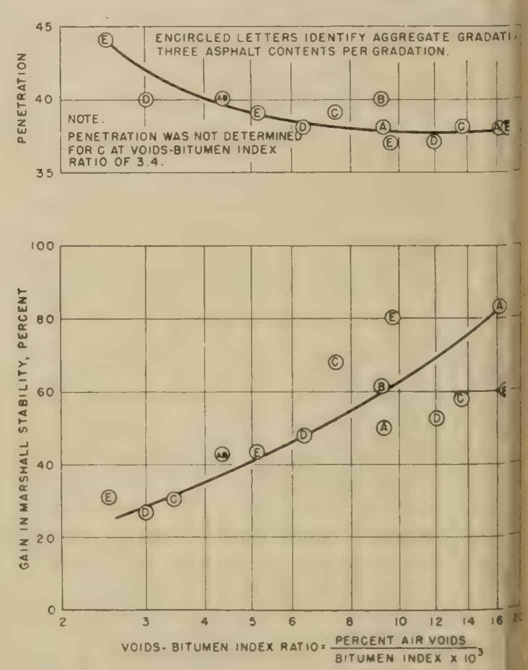



*Figure 10.—Effect of voids-bitumen in ratio on asphalt hardening after 303 of outdoor storage.*

sphalt recovered from specimens stored outoors was not contaminated as they had not een coated with paraffin. Because suitable ata were unavailable for computing retained enetration, the actual penetration was used n the plotting for the upper part of figure 0; as in the other figures, gain in Marshall ability was used to plot the lower part.

Penetration data in the upper part of figure 0, show somewhat more dispersion from the lotted curve than for the oven cured specinens of the second series of tests, as shown n figure 9. However, for this second series f tests, two extractions and recoveries were ade for each curing period compared to one ecovery for the outdoors weathered specinens of the first series of tests. In the upper art of figure 10, penetrations for two of the ixtures—the ones indicated by the first D nd second B from the left side—are shown s 2 to 3 points from the plotted curve, but hese penetrations are within the reproucibility of the penetration test method. If hese 2 points were disregarded, the plotted oints would be considered in excellent agreeent with the curve and this would indicate hat the voids-bitumen index ratio is a good easure for evaluating the resistance to sphalt hardening of a compacted mixture.

Information in the lower part of figure 10 so shows somewhat more dispersion of gain Marshall stability from the plotted curve an the data shown in figure 9. The greater spersion could be attributed to the operaonal factors associated with the long curing ocedure. Nevertheless, these data serve substantiate the fact that the voidstumen index ratio is a useful tool for evaluatg the effect of aging on the stability of wing mixtures.

### oids-Bitumen Index Ratio in Mix Design

In the discussion of data in table 8, aggreate blends A and B were mentioned as having o high a dust content for satisfactory use in tuminous paving mixtures. At design phalt contents, for both aggregates the tumen index was less than $1.23 \times 10^{-3}$ the inimum for satisfactory pavement performce based on the work of Campen and hers (*5*). This minimum bitumen index uld very well be included as a criterion in mix design procedures.

A substitute and possibly a better criterion r the 50-blow Marshall procedure might be e use of an upper limit on the voids-bitumen dex ratio and the elimination of the upper nit on percentage of air voids. From data figure 10 and table 9, a criterion of a maxium of 4.0 for the voids-bitumen index ratio ould eliminate the use of aggregate blends ch as A and B at all asphalt contents used the study reported here and would permit e use of the other three blends only at the gher asphalt contents. The degree of asphalt rdening was definitely less severe when the ids-bitumen index ratios were 4 or less gs. 9 and 10).

The use of a minimum air void content of 3 rcent and a maximum voids-bitumen index io of 4 in a mix design procedure and the compaction method used for the study discussed here—in which the density approximates the density of a 50-blow Marshall specimen—would permit differences in air void content according to the aggregate blend used. For example, the maximum air void contents that would be permitted for blends C, D, and E were interpolated from plotted curves of data from table 9 as 4.7, 5.1, and 6.0 percent, respectively. Data obtained from other plotted curves showed that the differences in air void content, from 3.0 percent to these maximum air void contents, corresponded to differences between maximum and minimum asphalt contents of 0.6, 0.7, and 1.1 percentage points, respectively. Thus, a wider range in asphalt content could be used with aggregate blend E than blends D or C. This seems logical as aggregate blend E was coarser and more open in gradation.

## DETAILS OF STUDY

### Preliminary Tests

Tables 1 and 2 list the properties of the asphalt and aggregates used in the study. Prior to the preparation of test specimens the crushed stone was separated into ½-inch to ⅜-inch, ⅜-inch to No. 4, and No. 4 to No. 8 sizes. Each of the three sands was thoroughly mixed but not sieved into smaller sizes. The cumulative system of weighing, starting with coarsest materials, was used in preparing the batches of aggregate for the study. Each batch was sufficient for only one test specimen.

Enough preliminary mixtures and test specimens were prepared and tested to establish, for each of the five aggregate blends: (1) effective specific gravity of the aggregate; (2) asphalt contents that would produce about 4-percent air void content in the compacted specimen; and (3) batch weights required for each mixture to produce the proper size of test specimen, 4 inches in diameter and 2½ inches high.

### Preparation of Mixtures and Specimens

The mixtures were prepared in a laboratory mixer from aggregate that had been heated to 325° F. and asphalt that had been heated to 300° F. The individual batches of aggregate were heated overnight in an oven. A gyratory compactor, of the type developed by the U.S. Army, Corps of Engineers, was used to mold the test specimens. Immediately after its preparation, each mixture was placed in a gyratory mold, heated to 200° F., and compacted by application of 30 gyrations at a 1° angle while it was under a foot pressure of 100 p.s.i. The same compactor and molding procedures were used in an earlier study (*12*) on aggregate gradations.

### First Series of Tests

#### Specimen grouping and testing

Fifteen different mixtures were used in the first series of tests: Each of the 5 aggregate blends was used in mixtures having high, medium, and low asphalt contents. A total of 180 specimens was molded to provide 3 specimens from each mixture and for each of 4 curing conditions.

The specimens having a high asphalt content, medium asphalt content, and low asphalt content were prepared and processed at intervals of about 2 weeks. The procedure used was as follows:

Four specimens from each of the 5 mixtures were molded on each of 3 consecutive days. Bulk specific gravities, determined by the procedure described in paragraph 4 of AASHO Designation: T 165-55 (*Effect of Water on Cohesion of Compacted Bituminous Mixtures*) were averaged for the specimens of each mixture. The 12 specimens of each mixture were then sorted into 4 groups so that each group of 3 contained a specimen from each day of molding and had approximately the same average bulk specific gravity. The 4 test groups of specimens were identified as:

*N*, No curing other than storage in laboratory air.

*P*, Storage in laboratory air plus 12 days in an oven at 140° F.

*Q*, Storage in laboratory air plus 63 days in an oven at 140° F.

*S*, Outdoor aging.

The *N* groups of specimens were tested for air permeability a few days after being molded. Paraffin was applied to their vertical faces before the permeability tests were made. On the ninth day after the first day of molding and after removal of paraffin, the *N* groups of specimens were tested for Marshall stability. These specimens were then sealed in cans so that later they could be extracted to determine the properties of the recovered asphalt. Upon completion of the oven curing, the specimens from the *P* and *Q* groups were tested for permeability and stability and sealed in cans in the same manner as the *N* groups.

The *S* groups of specimens were tested for permeability by using wet clay to seal their vertical faces. The specimens were then washed clean and air dried. Prior to their outdoor storage, aluminum foil was wrapped around the side of each specimen and folded down to completely cover the top surface. The wrapped specimens were then placed on a 1-inch bed of sand in a wooden frame that was on the flat roof of a concrete building. Sand was added to fill in the space between the specimens and was sealed on the surface by an application of a diluted asphalt emulsion. After the emulsion had set, the aluminum foil covering the tops of the specimens was removed. The specimens were stored outdoors for 303 days.

### Second Series of Tests

#### Specimen grouping and testing

Nine different mixtures were used in the second series of tests. Three different aggregate blends were used in mixtures having high, medium, and low asphalt contents. A total of 84 specimens was molded from these 9 mixtures: 4 from each mixture for each test condition. Specimens from each mixture were cured by the procedures $N$ and $P$, used in the first series of tests, and method $Q$ was used to cure only specimens from mixtures having medium asphalt contents. On each of 4 consecutive molding days, 2 specimens were prepared from each of the 6 high- and low-asphalt content mixtures, and 3 specimens were prepared from each of the 3 medium-asphalt content mixtures. Bulk specific gravities were determined for each of the specimens and all were averaged for each mixture. The specimens from each mixture were sorted into groups of 4 specimens so that each group contained a specimen from each day of molding and all groups had approximately the same average bulk specific gravity.

The 4-specimen test groups for the second series of tests were processed the same as the 3-specimen test groups in the first series of tests except that: (1) The vertical faces of the specimens were not coated with paraffin prior to permeability tests. (2) Upon completion of Marshall stability tests, the specimens and small amounts of steel wool, were placed in cans of distilled water and subjected to a vacuum of 27 inches of mercury for about 30 minutes. Earlier work had indicated that this was an effective means of preventing excessive hardening of the asphalt while the specimens were stored before extraction.

### *Effective Specific Gravity*

Several mixtures having different asphalt contents were prepared for each aggregate blend and tested by the procedure: *Maximum Specific Gravity of Bituminous Mixtures by Vacuum Saturation Procedure*, by J. M. Rice, ASTM Special Technical Publication No. 191, June 1953, pp. 43–61, or by test method: ASTM D 2041–64T. The maximum specific gravities were used to compute the effective specific gravities of the aggregates listed in table 2. The following formula was used:

$$\text{Effective specific gravity} = \frac{100}{\dfrac{100+P}{M} - \dfrac{P}{g}}$$

Where:

$P$ = Percent asphalt, aggregate basis.
$M$ = Maximum specific gravity of the mixture.
$g$ = Specific gravity of the asphalt.

### *Surface Area of the Aggregate*

Surface areas, corresponding to those determined by California, were computed from the gradations listed in table 2. The formula used was, as follows:

$$\text{Surface area (ft.}^2\text{ per lb. of aggregate)} = 2+0.02a+0.04b+0.08c+0.14d +0.30e+0.60f+1.60g$$

Where $a$, $b$, $c$, $d$, $e$, $f$, and $g$ are percentages of total aggregate passing sieve sizes Nos. 4, 8, 16, 30, 50, 100, and 200, respectively.

### *Bitumen Index*

California defines bitumen index as pounds of asphalt binder per square foot of surface area. The formula used for computing the bitumen index as given in tables 3, 8, 9, and 10 was:

$$\text{Bitumen index} = \frac{\text{Percent asphalt, aggregate basis}}{100 \times \text{surface area}}$$

### *Rate of Airflow*

Figure 3 shows the apparatus used in determining air permeability. The primary components are:

- A mold assembly for containing the test specimen.
- A sensitive needle valve for controlling the rate of airflow to provide a constant pressure differential. Airflow is produced by use of a laboratory vacuum or pressure line.
- A sensitive manometer for measuring pressure differentials of up to 2 inches of water.
- Three calibrated flowmeters of different capacities for measuring rates of airflow ranging from 2 to 9,100 milliliters per minute. Valves are provided for cutting off the two flowmeters not in use.

Use of the apparatus when a vacuum line is employed for producing airflow is illustrated in figure 4. The test mold consists of 3 parts:

- A metal cylinder 4 inches in diameter, open on both ends, that serves as a support for the test specimen.
- A metal cylinder 4 inches in diameter, open at the bottom, that rests on top of the test specimen; this cylinder is identified as the test cap.
- A metal cylindrical jacket slightly larger than 4 inches in diameter and containing a 4-inch diameter rubber triaxial sleeve as an inner liner. The support, specimen, and cap are confined by this inner liner. Connections are provided on the metal jacket for applying pressure or vacuum by a hand bulb to inflate or deflate the rubber liner. A pressure gage is also included.

The vertical face of the specimen normally is sealed with paraffin or some other suitable material prior to testing for permeability. To facilitate the placement of the sealed specimen into the test mold, the rubber membrane is deflated. After placing the support, specimen, and cap inside the jacket, a weight of 2,000 grams is placed on top of the cap, and air pressure of 2 p.s.i. is then applied to the rubber membrane to force it against the specimen and, finally, the hose connections shown in figure 4 are completed.

The procedure for measuring rate of airflow for a particular pressure differential is to open the valve for the most sensitive flowmeter and, keeping the other two closed, regulate the rate of airflow with the needle valve to maintain the desired pressure differential. When the desired pressure differential is obtained, the level of the floating ball is read and recorded and the rate of airflow can be determined from the calibration chart. If the level of the ball exceeds the maximum reading of the flowmeter, a flowmeter of higher capacity should be used.

### *Permeability*

Most of the rates of airflow were determined for pressure differentials of ½, 1, and 2 inches of water. Permeability, in fund- mental units, was computed from the con- ventional formula given by Hein and Schmi (9):

$$K = \frac{\mu \bar{Q} L}{A(p_1 - p_2)}$$

Where:

$K$ = Permeability per cm.², cm.³ per sec.
$\mu$ = Viscosity of air in poises.
$\bar{Q}$ = Rate of flow, cm.³ per sec.
$L$ = Height of sample, cm.
$A$ = Area of sample, cm.²
$p_1 - p_2$ = Pressure differential, dynes per cm

All specimens used in this study were inches in diameter and 2½ inches hig Laboratory temperatures during testing we not recorded but ranged from about 80° F. 90° F. Based on data from the *Handbook Chemistry and Physics*, 43d ed., 1961–196 the average viscosity of air was estimated be $1.853 \times 10^{-4}$ poises. Therefore, the formu reduces to:

$$K = \frac{0.974 \times 10^{-10} \times F}{P}$$

Where:

$F$ = Rate of airflow, ml. per min.
$P$ = Pressure differential, inches of water.

This simplified formula was used in computi the permeabilities shown in tables 4, 5, and

## REFERENCES

(*1*) *A Laboratory Oxidation Test for Aspha Bitumens*, by Victor Nicholson, The Assoc- tion of Asphalt Paving Technologists, vol., December 1937, pp. 208–214.

(*2*) *The Hardening of Asphalt with Relat to Development of Cracks in Asphalt Paveme*, by Prevost Hubbard, The Association of Asphalt Paving Technologists, vol. 9, Dece- ber 1937, pp. 165–194.

(*3*) *A Microviscometer Study of Road Asph Hardening in the Field and Laboratory*, by J. Heithaus and R. W. Johnson, Proceedings of the Asphalt Paving Technologists, vol. , February 1958, pp. 17–34.

(*4*) *A Laboratory-Field Study of Hot As- phaltic Concrete Wearing Course Mixtures*, by Joseph F. Goode and Elliott R. Owings, PUBLIC ROADS, *A Journal of Highway Re- search*, vol. 32, December 1961, pp. 221–2, and Papers on Road and Paving Materials and Symposium on Microviscometry, ASTM Special Technical Publication No. 309, June 19, pp. 1–21.

(*5*) *The Relationships between Voids, Sur Area, Film Thickness and Stability in Bi- minous Paving Mixtures*, by W. H. Camper J. R. Smith, L. G. Erickson, and L. R. Me, Proceedings of the Association of Asph Paving Technologists, vol. 28, January 19, pp. 149–178.

(*6*) *Field Measurement of Air-Permeabi for Control of Bituminous Mat Construction*, by Martin Ekse and Zung Teh Zia, Proceedi of the Association of Asphalt Paving Tech- ogists, vol. 22, January 1953, pp. 44–45.

(*7*) *Permeability, Void Content, and Dura- bility of Bituminous Concrete*, by J. F.

McLaughlin and W. H. Goetz, Purdue University, Proceedings of Highway Research Board, vol. 34, January 1955, pp. 274–286.

(8) *A Method for Measuring the Air Permeabilities of Asphalt Concrete Pavements,* by W. H. Ellis and R. J. Schmidt, Papers on Road and Paving Materials (Bituminous)—1960, ASTM Special Technical Publication No. 294, June 1960, pp. 85–91.

(9) *Air Permeability of Asphalt Concrete,* by R. C. Hein and R. J. Schmidt, Papers on Road and Paving Materials and Symposium on Microviscometry, ASTM Special Technical Publication No. 309, June 1961, pp. 49–62.

(10) *Control of Asphalt Concrete Construction by the Air Permeability Test,* by W. J. Kari and L. E. Santucci, Proceedings of the Association of Asphalt Paving Technologists, vol. 32, February 1963, pp. 148–170.

(11) *Permeability-Compaction Characteristics of Bituminous Mixtures,* by D. B. Warner and F. Moavenzadeh, Proceedings of the American Society for Testing and Materials, vol. 64, 1964, pp. 981–990.

(12) *A New Graphical Chart for Evaluating Aggregate Gradations,* by Joseph F. Goode and Lawrence A. Lufsey, Proceedings of the Association of Asphalt Paving Technologists, vol. 31, January 1962, pp. 176–207 and, *Aggregate Gradation for Highways, Simplication, Standardization, and Uniform Application and A New Graphical Evaluation Chart,* Bureau of Public Roads, 1962, pp. 13–26.

# NEW PUBLICATIONS

### *Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds*

The third issue of *Highway Research and Development Studies, Using Federal-Aid Research and Planning Funds* (May 1965), is now for sale by the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, for 75 cents. This publication is a compendium of information on Highway Research and Development studies financed with 1½-percent Planning and Research funds approved by the Office of Research and Development, Bureau of Public Roads. Information is included for each State for a 12-month period on projects approved for the calendar year 1965, or for the fiscal year 1965, which began July 1, 1964.

The information has been grouped according to seven broad study areas, and data are also presented on the objective of each study, the conducting agency, and the funding for each study. The seven areas are: Determination of the Future Role and Type of Facilities Needed for Highway Transportation; Reduction of Highway Accidents; Increase the Capacity of Urban Roads and Streets; Develop Method for Designing and Evaluating Highway Pavements and Structures; Reduction of Costs of Drainage Installations; Reduction of Construction, Maintenance, and Administration Costs; and Improve Capability for Conducting Research and Effect Rapid Utilization of Research Results.

This publication also contains a listing of the projects for the fiscal year 1965 in the American Association of State Highway Officials National Cooperative Highway Research Program (NCHRP). Sponsored by AASHO and administered by the Highway Research Board of the National Academy of Sciences, these projects are financed from a pool of funds consisting of 5 percent of the 1½-percent Federal-aid funds of the participating State highway departments.

### Urban Planning Publications

Two new publications prepared by the Urban Planning Division, Office of Planning, Bureau of Public Roads, are available from the Superintendent of Documents, Washington, D.C., 20402, at the price listed for each: *The Role of Economic Studies in Urban Transportation Planning*, by Joseph P. Meck, 45 cents; and *Traffic Assignment and Distribution for Small Urban Areas*, by the Urban Planning Division, $1.00.

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- Provide assistance and guidance to those responsible for reviewing economic forecasts required as part of the comprehensive transportation planning process in urbanized areas.
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### *Traffic Assignment and Distribution for Small Urban Areas*

The manual, *Traffic Assignment and Distribution for Small Urban Areas*, explains and illustrates the theory and use of a system of analytical procedures and computer programs for assigning and distributing trips to transportation systems with the IBM 1620 (60K) electronic computer. The procedures described are designed to process a basic set of trip cards. They allow the computation of surveyed trip length frequency, distribution of trips between zones by the gravity model formula or Fratar method, and assignment of trip interchanges to an existing transportation network.

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This battery of computer programs was developed by several State highway departments and the Bureau of Public Roads, and is generally applicable to urban areas having populations up to 150,000 persons. To utilize the computer programs for this system, the study area must be described by not more than 699 nodes, of which 200 may be zone centroids. The maximum traveltime that can be accommodated is 99 minutes.



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WASHINGTON

**Interstate 95–6, north of Old Belgrade Road, near Waterville, Maine.**

View is toward the south on southbound roadway and shows the use of split profile on side hill location.

IN THIS ISSUE: *Article on why drivers choose an expressway or primary route*

# Public Roads

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**Muriel P. Worth,** ***Editor***
**Jessie W. Dean,** ***Assistant Editor***

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# Attitudes of Drivers Determine Choice Between Alternate Highways

*BY THE OFFICE OF RESEARCH AND DEVELOPMENT BUREAU OF PUBLIC ROADS*

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***The research information presented in this article is based on a study of the factors that influence a driver's choice of alternate routes. Through the study in which the attitudes of drivers toward two highways were measured, an attempt was made to determine the utility of attitude scaling methods for predicting the choice. Establishment of such a subjective measure was sought for use in highway design, traffic planning in general, and predicting the use that will be made of new and improved highways. The author believes that the data collected show that this subjective method of evaluating route choice is a simple and effective means of predicting use of highway facilities.***

***In addition to the attitudes of the drivers, traffic characteristics of the routes were measured and the tension generated on each was determined. Nine test drivers were used for the tension tests. The routes employed were 47-mile sections of an expressway design toll road and a parallel rural primary highway. Drivers were sampled entering and exiting on both highways. A summated rating attitude scale was administered to a sample of 3,259 drivers. Descriptive information was obtained about the driver, his trip, and travel habits. Analysis of results showed that these drivers held stable attitudes that clearly differentiated between the routes. Direct measurement of driver attitudes seems to be a far better predictor of route choice than any descriptive information about the drivers or their driving habits.***

***In addition, the results provide a means of rationalizing the attraction of traffic to an expressway on the basis of drivers who seek to minimize tension in driving. The data suggest that total stress incurred in driving is a more important determinant of route choice than either operating costs or traveltime costs. A model of route choice and attraction of traffic is proposed based upon tension generation that can be related to traveltime data. Analysis of this research shows that drivers evaluate the use of alternate highways in a rational, though subjective, fashion. Such evaluation, however, seems to be very independent of the usual monetary plans often used to measure highway benefits and costs.***

## *Introduction*

WHENEVER a driver is provided alternate routes, he must make an evaluation of the benefits and costs of using each in order to make a choice. If he knew nothing about available alternate highways or did not make an evaluation of them, his choice would be random. Because drivers do not operate in a random manner, it seems reasonable to assume that they learn the characteristics of the highways and out of this learning develop a basis for evaluation of alternate routes. A driver's choice thus becomes dependent on the diverse characteristics of the alternates relative to his trip objectives, and these determine stable choice behavior. This behavior is of considerable significance both in determining the use of highway facilities and the benefits a driver derives from them.

Three major factors have been developed to account for the patterns of choice that a driver makes between alternate highways. The first is the time savings obtained by taking one route instead of the other. The second is the direct and indirect operating cost savings obtained by taking one route instead of the other. The third is the comfort and convenience savings obtained by taking one route instead of the other.

In general, traveltime savings have been the dominant criterion of use of alternate facilities; the best predictor being the traveltime ratio. In both rural (*1, 2*)[3] and urban studies (*3, 4*) a driver seems to choose routes that provide significant time savings, even though he may have to drive a longer distance. Discussions in all these studies imply that the driver values time directly and, hence, scales that variable. From an economic standpoint, a considerable effort has been made to determine the dollar equivalent of this time scale. For passenger car drivers these attempts have not been particularly successful (*5*). The relation of operating cost to choice by a passenger car driver seems to be weak (*6*). Either the driver does not evaluate operating cost differences or these differences are insignificant. When related to the total costs of a trip, operating cost differences between alternate routes may be very trivial for the passenger car driver.

In addition to these physical measurements, the purely subjective concept of comfort and convenience has been developed. This has generally been described qualitatively as the ease of driving or freedom of movement. Claffey (*6*) has scaled this factor in terms of the changes in speed imposed on the driver and, hence, counted the impedances to movement. Michaels (*7*) has differentiated among highways on the basis of the tension aroused in a driver from traffic and geometric design features. His results indicate that tension reduction is the greatest single saving accruing to a driver who chooses an expressway over a parallel uncontrolled-access highway, and the driver seems to subjectively evaluate alternates in conformity to the tension induced on each.

Although the research reports on the problem of use of alternates have described what traffic does, little research has been carried out on driver perception of alternate routes available (*3*). Further, no attempts have been made to measure on a quantitative scale the evaluations a driver makes or his relation of these evaluations to choice of routes. Thus, no reliable way now exists to predict usage of facilities except by empirical studies of traffic.

Regarding any benefit in analysis of highway facilities, obviously, drivers evaluate on a predominantly subjective basis. No economic determination seems feasible unless the

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[3] References indicated by italic numbers in parentheses are listed on page 236.


*Figure 1.—Map locating study routes.*

scale of value drivers use and its relation, if any, to dollars is known.

Considering the problem of selection of alternate routes, a reasonable assumption is that choice will be based upon what the driver has learned about the alternate. Either directly or indirectly, a driver must develop some stable evaluations. That is, he must have some predisposing views toward the routes or his choices would be random. These predisposing views are, by definition, the attitudes an individual holds toward some object or process. If route choice is rational, then a direct measure of a driver's evaluation should be his attitudes toward the alternate. By determining the intensity of these attitudes toward a pair of highways, it should be possible to determine how these attitudes are related to the characteristics of the highways and the choices drivers make.

To achieve these objectives, however, it is first necessary to determine whether a stable set of attitudes exists toward highways of different characteristics. Second, it is necessary to determine whether these attitudes depend on the characteristics of the drivers, which are relatively permanent, or upon the characteristics of a particular trip that would cause highly variable attitudes. In this context, the study discussed here was developed. The aim was to test the hypothesis that drivers on each of two highways had significantly different attitudes toward the two highways and that these attitudes were based on the more enduring characteristics of the routes and the drivers.

## *Development of the Attitude Scale*

The attitude scaling technique employed in this study was the Method of Summated Ratings. It employs a series of direct statements to which the respondent expresses the extent of his agreement. An example of such a statement might be, "A road with many hills and curves is interesting to drive." The test subject then responds in one of five categories ranging from "strongly agree" to "strongly disagree." A score of 0, 1, 2, 3, or 4 is given to his response, according to the category chosen, a score of 2 being neutral. Thus, by using a set of such items, a total attitude score can be obtained for any test subject toward the road under study.

The general procedure for preparing such an attitude battery is described by Edwards (*8*). In the study reported here, it was decided to compare attitudes on a toll road and a rural primary road as these are two of the more common that a driver has to choos between, and yet they have radically differen design characteristics. To develop the fin items for the attitude scale, 61 statemen were initially prepared. They described variety of characteristics of a rural primar road and an expressway, both positive an negative. They were presented to 260 sta members of the Bureau of Public Road Instructions given were:

"Place yourself in a hypothetical situatio of having the choice of two routes for hon to work trips: (1) a controlled-access to road, and (2) a parallel free-access primar roadway. The toll on the turnpike is $ The trip is 30 miles on both routes. Assum that the primary route is similar to U.S. between Baltimore and Washington, between Alexandria and Woodbridge.

"The attached questionnaire is design to elicit attitudes toward these two types highways. You should respond to ea statement in terms of your own persor feelings, checking one of the five categori that range from strongly agree to strong disagree."

Some basic objective information w obtained about the respondents, includi age, sex, and the percentage of time th would choose the toll road. Adding the la item permitted an initial check on the valid of the final scale, for it was hypothesized th those responding most positively to expre way items would be those most likely to u that facility. All items were scored in ter of favorability toward the expressway. T returns were then analyzed according to t standard procedure in which the high scoring quarter of the sample was compa with the lowest scoring quarter; well o half the items significantly differentiated tween the two highways. The final batt was composed of 18 items, from the origi group of 61, that were the most discriminat between the groups having high and l scores.

A further analysis was made on this pret group. The attitude scores were correla with the respondents' percentage of choice the toll road. The two distributions w dichotomized and a phi coefficient was co puted. The correlation coefficient was +( between attitude scores and choice of rou Thus, it was reasonable to conclude that this hypothetical situation, a stable set attitudes existed toward the two types highways that was significantly related to choice of routes the respondents would ma

In addition to the final attitude batter questionnaire was included to obtain s basic descriptive information about respondent's trips so that the attributes of driver and his trips could be related to attitudes. These items were to provid means for testing the stability of the attitu and fell into three basic categories. The was the characteristics of the driver and vehicle, including age and sex of driver, age of car. The second was the charac istics of the trip, including purpose, num of car occupants, the driving time alr completed, and driving time to be comple he third was the descriptive information of he driver's estimate of the frequency with hich he made this kind of trip and the frequency with which he used the alternate route.

The item on driving time was included ecause no statements relating to traveltime lone were in the attitude battery. In the ample used to develop the scale, time was not discriminating factor between the groups coring high and low. By treating traveltime s an independent variable, subjective estimates of driving time could be related to the espondent's attitudes toward the routes. bviously, if traveltime were a dominant riterion of choice, then a correlation should xist between the driver's attitude toward the oute and the duration of the trip that he was ndertaking. By using this approach, an ndependent test could be made of a driver's hoice of routes and of traveltime.

### *Selection of Test Location*

In considering a pair of roads of sharply ifferent characteristics between which a river might choose, the ideal would be a air that had a common beginning and a comon terminus. In addition, the pair should e long enough to permit a meaningful choice y the driver. A pair of highways that t these requirements is the Maine Turnpike etween Kittery and South Portland and the arallel rural primary, U.S. 1, which has been udied extensively over the past decade (1, 2). The sections are approximately the ame length, about 45 miles. At the Kittery nd, the choice of route is a simple one for the river, for the connection is a Y. At the outh Portland end, U.S. 1 and the Turnpike oin again. A map of the two roads is shown n figure 1.

The characteristics of both routes are ypical of a modern toll road and a rural rimary. The Turnpike is a 4-lane divided ighway on which interchanges are spaced - to 15-miles apart; they generally have een built to Interstate design standards. .S. 1 varies from 2- to 4-lanes and passes hrough several small towns and undeveloped ountryside. Access is not controlled, and he route has a variety of traffic control devices.

### *Procedure*

A survey team of nine men was used. he sampling schedule was set for daylight ours between 8 a.m. and 5 p.m., and was ffected at both ends of each highway. uring the first 4 hours, vehicles were stopped s they entered the test sections; during the ext 4 hours, they were stopped as they left he test sections. Samplings were obtained om north and south ends of both routes, ut drivers were not stopped twice on the ame trip. By counterbalancing the order, n approximately equal sampling of drivers ntering and exiting at both ends of the two ighways was obtained.

To obtain the most stable attitudes toward he routes under study, only Maine or New ampshire drivers were stopped. No fixed rocedure was established for stopping a particular vehicle. The complexities of traffic and the fact that only two interviewers were at each station precluded any formal sampling procedure. However, by extending the sampling period for more than 30 days, it is believed that most biases were eliminated.

When a driver was stopped, a common set of instructions was given:

"Good morning. We are doing research on why drivers pick particular roads for their trips and would like to enlist your assistance. We have a questionnaire that we would like you to complete, which will take about 5 minutes of your time. If you can spare that time, we would appreciate it."

If the driver agreed, the attitude form was handed to him and the instructions for filling it out were read with him. When the interviewer and the driver were satisfied as to what was wanted, the interviewer withdrew and the driver completed the attitude questionnaire. When finished, he handed the form back to the interviewer who then asked the objective questions and marked the verbal replies on a coding sheet. The two parts of the form had a common number so that both parts of the survey could be combined subsequently.

#### Speed and volume measurements

In addition to the attitude survey, traffic measures were taken on the two routes. Rather complete volume counts were made daily for both the Turnpike and U.S. 1. On U.S. 1, volume counters were placed at three locations for hourly traffic counts. On the Turnpike, volume was sampled at four locations during several different time periods. In addition, a radar speed meter recorded daily samples of traffic speed on both routes. Thus, a fairly complete record of the traffic characteristics on both test sections was obtained during the period of the study.

#### Tension measurements

The galvanic skin reflex (GSR) test was employed to obtain tension measurements on both the Turnpike and U.S. 1. During the 1-month study each of the interviewers was used as a test subject and drove both routes twice in both directions. The procedure outlined in previous reports (7, 9) was employed.

### *Results*

During the 4 weeks of surveying on both routes, a total sampling of 3,259 different drivers was obtained. No significant differences were noted between drivers sampled at the two ends of the test routes. Also, no differences were noted between drivers sampled on entering the test sections and those leaving them. Hence these data were pooled. As shown in table 1, approximately the same

**Table 1.—Attitudes of drivers toward the Maine Turnpike and U.S. 1**

| Sex of drivers | Maine Turnpike | | | U.S. 1 | | |
|---|---|---|---|---|---|---|
| | *Number sampled* | *Mean attitude score* | *Standard deviation* | *Number sampled* | *Mean attitude score* | *Standard deviation* |
| Male | 1,138 | 41.33 | 9.40 | 1,039 | 32.09 | 9.56 |
| Female | 482 | 38.52 | 9.54 | 600 | 30.26 | 8.65 |
| Total | 1,620 | ---------- | ---------- | 1,639 | ---------- | ---------- |

**Table 2.—Distribution of drivers sampled on Maine Turnpike and U.S. 1, by sex**

| Sex of drivers | Maine Turnpike | | U.S. 1 | | Total | |
|---|---|---|---|---|---|---|
| | *Number sampled* | *Percent* | *Number sampled* | *Percent* | *Number sampled* | *Percent* |
| Male | 1,138 | 70.4 | 1,039 | 63.4 | 2,177 | 66.7 |
| Female | 482 | 29.6 | 600 | 36.6 | 1,082 | 33.3 |
| Total | 1,620 | 100.0 | 1,639 | 100.0 | 3,259 | 100.0 |

**Table 3.—Age of vehicles on the Maine Turnpike and U.S. 1, by sex of driver**

| Vehicles | | Vehicle distribution by drivers sampled on— | | | |
|---|---|---|---|---|---|
| Age | Sample | Maine Turnpike | | U.S. 1 | |
| | | Male | Female | Male | Female |
| *Year* | *Percent* | *Percent* | *Percent* | *Percent* | *Percent* |
| Less than 1 | 18.6 | 22.5 | 22.1 | 16.7 | 13.2 |
| 1–3 | 39.3 | 45.9 | 39.6 | 34.2 | 35.9 |
| 4–6 | 26.2 | 20.1 | 27.1 | 28.1 | 32.8 |
| More than 6 | 15.7 | 11.4 | 11.1 | 21.3 | 18.0 |

Table 4.—Analyses of variance of attitudes of male and female drivers toward Maine Turnpike, based on age of drivers and vehicles

| Source of variance | Sum of squares | Degree of freedom | Mean squares | $F$, ratio | Probability ($F$) |
|---|---|---|---|---|---|
| MALE DRIVERS | | | | | |
| Driver age | 656.53 | 3 | 215.51 | 2.468 | <0.05 |
| Vehicle age | 996.48 | 2 | 498.24 | 5.706 | <0.01 |
| Age × vehicle | 243.35 | 6 | 40.56 | ---------- | (1) |
| Residual | 99,454.14 | 1,139 | 87.32 | ---------- | ---------- |
| Total | 101,341.50 | 1,150 | ---------- | ---------- | ---------- |
| FEMALE DRIVERS | | | | | |
| Driver age | 464.30 | 3 | 154.80 | 1.543 | (1) |
| Vehicle age | 263.10 | 2 | 131.55 | 1.312 | (1) |
| Age × vehicle | 418.42 | 6 | 69.74 | ---------- | (1) |
| Residual | 48,342.20 | 482 | 100.30 | ---------- | ---------- |
| Total | 49,488.02 | 493 | ---------- | ---------- | ---------- |

[1] Not significant.

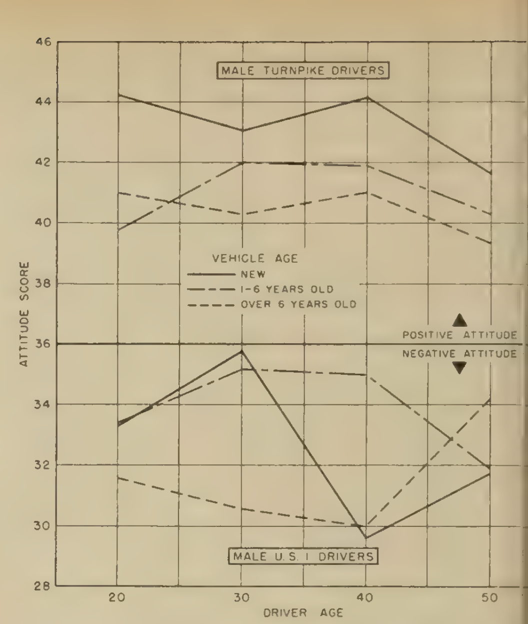


*Figure 2.—Male driver attitudes tow turnpike as function of driver and vehi age and travel route.*

Table 5.—Analyses of variance of attitudes of male and female drivers toward U.S. 1, based on age of drivers and vehicles

| Source of variance | Sum of squares | Degree of freedom | Mean squares | $F$, ratio | Probability ($F$) |
|---|---|---|---|---|---|
| MALE DRIVERS | | | | | |
| Driver age | 2,532 | 3 | 844.0 | 9.58 | 0.01 |
| Vehicle age | 629 | 2 | 313.5 | 3.56 | 0.05 |
| Age × vehicle | 1,390 | 6 | 231.7 | 2.62 | 0.05 |
| Residual | 86,299 | 980 | 88.1 | ---------- | ---------- |
| Total | 90,850 | 991 | ---------- | ---------- | ---------- |
| FEMALE DRIVERS | | | | | |
| Driver age | 1,148 | 3 | 382.7 | 5.50 | 0.01 |
| Vehicle age | 755 | 2 | 377.5 | 5.42 | 0.01 |
| Age × vehicle | 722 | 6 | 120.3 | 1.73 | (1) |
| Residual | 42,605 | 604 | 69.5 | ---------- | ---------- |
| Total | 45,230 | 615 | ---------- | ---------- | ---------- |

[1] Not significant.

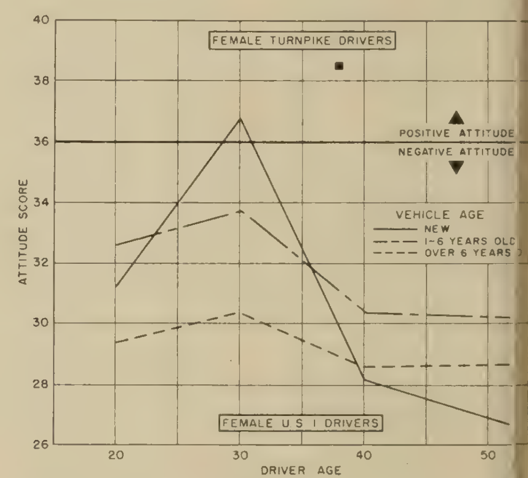


*Figure 3.—Female driver attitudes tow turnpike as function of driver and veh age and travel route.*

number of observations were taken on both routes. This, of course, does not represent the distribution of traffic but only the method of sampling on the two highways.

Fourteen percent of the drivers stopped declined to participate in the survey. This percentage was the same on both routes. In addition, approximately 6 percent of the drivers stopped had been interviewed before. As might have been expected, the percentage of repeats from the first week to the last week rose on U.S. 1 from 1.9 percent at the end of the first week to 5.7 percent the third week. On the Turnpike, the figures rose from 0.8 percent, at the end of the first week, to 10.3 percent the third week.

## *Attitude Survey*

The Turnpike was used as a reference for assigning a quantitative score to the responses when the attitude questionnaires were scored. Thus, all statements about U.S. 1 that reflected a positive attitude toward it were given a 0 score for the category of "strongly agree" and a score of 4 for the response of "strongly disagree." For those items that were unfavorable statements about U.S. 1, strong agreement was scored as 4 and strong disagreement as 0. Statements about the Turnpike were scored in the obvious reverse manner. Thus, the total score of a respondent was interpreted to reflect his attitude toward the Turnpike. The scores on each of the items and the descriptive information obtained from the interview were placed on punchcards, and all of the basic analyses of the attitude sampling was performed by a computer.

A summary of the attitudes of drivers on each route is shown, by sex, in table 1. The higher the score, the more positive the feelings of the drivers toward the Turnpike. A score of 36 indicated a neutral attitude toward the Turnpike. As shown in table 1, significant differences were stated for choosing between the two highways. Drivers on U.S. 1 had negative attitudes toward the Turnpike, and Turnpike users had positive attitudes toward it. Also, the differences stated by the sexes were significant. The male drivers on the turnpike were significantly more positive toward the Turnpike than the female driver. On U.S. 1, the male driver, although hav a negative attitude toward the Turnpike, less negative than the female driver. attitudes of male and female drivers on b routes were significantly different from tral. Thus, it is reasonable to conclude use of the attitude scale showed a differe ation between the users of the two highw

The sex distribution of the drivers on two routes was analyzed and, as show table 2, two-thirds of the total sampling drivers was male. More significant, howe is the difference between the proportio male or female drivers on the two ro Significantly more female drivers trav U.S. 1 than the Turnpike. Compariso this sex distribution with attitudes tow the Turnpike (table 1) indicates a significa less positive attitude of the females than males toward the Turnpike. Therefor was concluded that a correlation exi between the attitudes held by the two s toward the highways and the actual ch they made.

The third category under the driver vehicle characteristics concerns that of ve age. The percentages of vehicles on ute, by their age, and by sex of their drivers shown in table 3. Two inferences may be ade from this table: First, in this sample, hicles driven by females were older than ose driven by males. Second, and more gnificant, the percentage of older vehicles the Turnpike was considerably less than ose on U.S. 1.

Drivers in the sampling on both routes were mpared for age differences. In relation to titudes toward the two highways, rather arcut differences existed. An analysis of riance was performed for both driver age d vehicle age, the attitude scores being the pendent variable. The summary tables for ales and females using the Turnpike are shown table 4 and for those on U.S. 1, in table 5. oth driver age and vehicle age were statisti-lly significant in every analysis except for e female drivers on the Turnpike. In gures 1 and 2 the mean attitude scores as a nction of age are shown for all conditions. ehicle age is the parameter in these curves. s shown for the male drivers, attitudes ward the Turnpike became less positive as eir age increased. Vehicle age also had a ear effect on the attitudes. Thus, the ewer the automobile, the more positive was e attitude toward the Turnpike. In general, e same results were obtained for the female ivers on U.S. 1; that is, there was a definite dering of attitudes by age of vehicle and iver. A peak in attitudes toward the Turn-ke seemed to occur in the age range of 25 to , after which drivers' attitudes became ore negative toward the Turnpike. No gnificant differences were noted for the male driver on the Turnpike. From these alyses it was concluded that attitudes ward the alternate highways were signifi-ntly dependent on the stable characteristics the driver and his vehicle. Analyses of ese results further indicate that attitudes ward alternate routes were very stable, olving partially out of the enduring charac-ristics of the driver and his vehicle.

## Attitudes and Trip Characteristics

The second class of relations to a driver's titude concerned the characteristics of the ecific trip during which the driver was mpled. The objective of this analysis was determine whether the attitudes toward e two highways as markedly modified by e purpose of the trip, the number of occu-ants in the vehicle, and the traveltime sociated with the trip. Analysis showed at no significant relations existed between ther the trip purpose or the number of cupants in the vehicle and the driver's titude toward the Turnpike. Similarly, the lation between subjective estimates of trip ration was unrelated to driver's attitude ward the Turnpike. Thus, the results of is analysis on the characteristics of the ecific trip indicate that a driver's attitude as independent of the specific trip. The oice, then, between alternates was made on basis of stable and preexisting attitudes ward the different types of highways.

The results relevant to traveltime should not be interpreted to mean that there were no differences in the distribution of trip durations on the two highways. Table 6 contains the frequency distributions for the sample. These time vaules are subjective estimates of the time already spent driving as well as being estimates of the time required to complete the trip. Therefore, the longer the trip, the more likely it was to be made on the Turnpike. Thus, approximately 32 percent of all drivers sampled on the Turnpike had been traveling for less than one-half hour and 54 percent had more than 1 hour left to drive. But on U.S. 1, 70 percent of the drivers had been driving for less than one-half hour and only 25 percent needed more than another one-half hour of driving to complete their trip. A slightly different presentation in figure 4 shows the percentage distribution of remaining triptime for drivers who had just started their trips. Only 15 percent of those on U.S. 1 expected to be driving for more than one-half hour, whereas 71 percent of the drivers starting their trips on the Turnpike expected to drive for more than one-half hour. Thus, the drivers on longer trips were the ones that tended to gravitate toward the Turnpike

A clearer understanding of the effects of triptime and attitudes can be obtained by examining reports of only those travelers on both routes who had approximately common origins and destinations. If only those Turnpike drivers are selected who had been traveling for less than one-half hour and who had between one-fourth hour and 1 hour left to travel, they could be compared with U.S. 1 drivers who also had been traveling for less than one-half hour but who had between one-half hour and 2 hours more to drive. Obviously, drivers who chose U.S. 1 sacrificed time. The attitudes of male drivers of different ages who chose the Turnpike were compared; the scores are shown in table 7. There were no significant differences among the ages of Turnpike drivers; whereas on U.S. 1, choice of the Turnpike decreased significantly when the drivers were older. However, the U.S. 1 driver always had a significantly negative attitude toward the Turnpike. Thus, it is concluded that for trips having common origin and destination, the driver's choice between the two routes was related mostly to his attitude toward the alternate. For drivers on U.S. 1, this showed that they chose the rural primary route instead of the expressway although this choice increased traveltime 30 percent.

The sample was also analyzed in relation to the frequency with which drivers made trips between South Portland and Kittery. Trip frequency was defined in three categories: Less than 1 trip a year, 1 to 12 trips a year, or more than 1 trip a month. The distribution was computed for both the Turnpike and U.S. 1 and for the two sexes. The percentage of the total sampling on each route for the two sexes and the trip frequencies are shown in table 8. In the Turnpike sampling, the majority of the drivers made the trip more than once a month. On U.S. 1, however, the

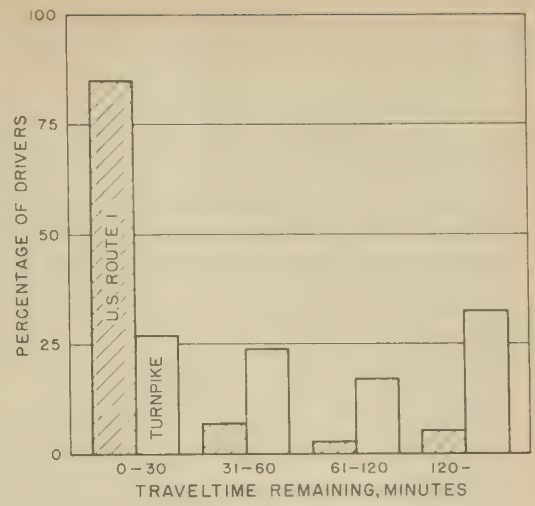


*Figure 4.—Remaining trip time after driving less than 30 minutes, percentage distribution.*

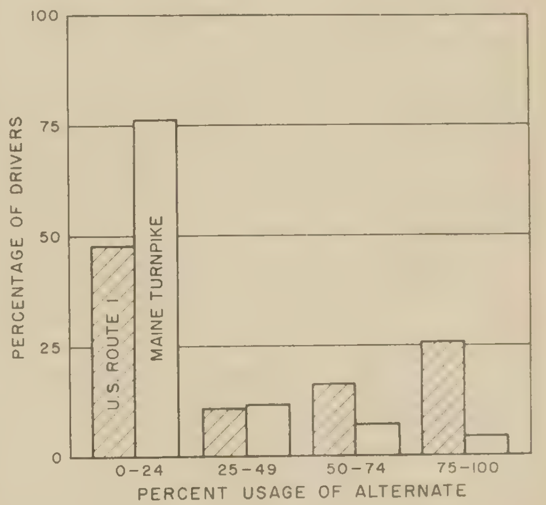


*Figure 5.—Frequency of usage of alternate routes by drivers sampled, both routes.*

majority of the drivers made the trip between once a year and once a month. A chi-square test was used to test the differences between the number of trips made on the Turnpike and those on U.S. 1, and the differences between the distributions were significant. When trip frequency increased to more than one trip a month, the proportion of these trips made on U.S. 1 decreased and the proportion on the Turnpike increased. This may indicate that the Turnpike exerted an attraction for drivers as the frequency with which they traveled between Kittery and South Portland increased.

The attitudes of drivers toward the two routes were also analyzed as a function of frequency with which trips were made between South Portland and Kittery. The mean attitude scores are shown in table 9. Because of the significant differences among ages of drivers, the data also are separated by that variable. Two inferences may be made: First, the influence of age is the same as discussed previously. Second, as a function of trip frequency, a consistent and significant increase occurred in the average attitude score of both male and female drivers toward the Turnpike. In addition, the drivers on U.S. 1, although having negative attitudes toward the Turnpike, tended to have a change in attitude,

**Table 6.—Distribution of driving times for drivers traveling on Maine Turnpike and U.S. 1**

| Driving time completed, minutes | Maine Turnpike: Driving time left, minutes — Less than 15 | 15-30 | 31-60 | 61-120 | More than 120 | Total | U.S. 1: Driving time left, minutes — Less than 15 | 15-30 | 31-60 | 61-120 | More than 120 | Total |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Less than 15: | | | | | | | | | | | | |
| Male | 55 | 12 | 45 | 35 | 68 | 215 | 190 | 146 | 29 | 16 | 29 | 410 |
| Female | 4 | 2 | 22 | 11 | 23 | 62 | 136 | 96 | 17 | 4 | 7 | 260 |
| 15-30: | | | | | | | | | | | | |
| Male | 16 | 40 | 32 | 32 | 66 | 186 | 91 | 146 | 31 | 14 | 19 | 301 |
| Female | 8 | 23 | 15 | 24 | 25 | 95 | 94 | 92 | 20 | 13 | 6 | 195 |
| 31-60: | | | | | | | | | | | | |
| Male | 35 | 43 | 27 | 34 | 42 | 181 | 41 | 51 | 12 | 11 | 14 | 129 |
| Female | 13 | 21 | 12 | 8 | 13 | 67 | 12 | 34 | 9 | 5 | 6 | 66 |
| 61-120: | | | | | | | | | | | | |
| Male | 31 | 44 | 33 | 59 | 85 | 252 | 25 | 27 | 17 | 16 | 20 | 105 |
| Female | 10 | 28 | 20 | 35 | 35 | 128 | 7 | 20 | 7 | 8 | 3 | 53 |
| More than 120: | | | | | | | | | | | | |
| Male | 36 | 64 | 38 | 72 | 173 | 383 | 14 | 16 | 10 | 26 | 48 | 114 |
| Female | 12 | 23 | 22 | 32 | 64 | 153 | 8 | 16 | 4 | 1 | 16 | 45 |
| Cumulated total: | | | | | | | | | | | | |
| Male | 173 | 203 | 175 | 232 | 434 | 1,217 | 361 | 386 | 99 | 83 | 130 | 1,059 |
| Female | 47 | 97 | 91 | 110 | 160 | 505 | 227 | 266 | 57 | 31 | 38 | 619 |

**Table 7.—Mean attitude scores for male drivers whose trips had approximately common origins and destinations**

| Driver age | Mean attitude scores for male drivers on— Maine Turnpike | U.S. 1 |
|---|---|---|
| Less than 24 | -------- | 35.37 |
| 24–34 | 42.47 | 34.22 |
| 35–44 | 43.32 | 33.72 |
| More than 44 | 41.73 | 29.90 |

approaching neutrality, toward the Turnpike as trip frequency increased. Thus, as trip frequency increased, a general shift to more positive attitudes toward the Turnpike occurred. This result offers further evidence that a driver's attitude toward the two highways shifted, on the basis of his driving experiences on both of the routes, toward favoring the expressway.

A final general analysis was made concerning the extent of utilization of the alternate routes by drivers. Each driver sampled was asked what percentage of time he used the other route for his trips. The percentage of the drivers sampled, who used the alternate route a specific percentage of the time, is shown in figure 5. Because of no differences in data for male and female drivers, all the data were combined. The drivers sampled on the Turnpike rarely used U.S. 1—only 12 percent of the sampling of Turnpike drivers used U.S. 1 for more than half their trips. But drivers sampled on U.S. 1 frequently used the Turnpike—42 percent used it for more than 50 percent of their trips. This usage also indicates an attraction of drivers toward the Turnpike.

### *Attitude Scale*

The attitude scale employed in this study was composed of two classes of items. One classification of the statements was by their reference to either the Turnpike or U.S. 1, and the other was according to whether they were favorable or unfavorable. Hence, the the items in the attitude scale can be classified in a 2 by 2 matrix. In addition, the total attitude score was arbitrarily scored in relation to the Turnpike—a negative statement about U.S. 1 was interpreted as being favorable toward the Turnpike; conversely, a positive statement toward U.S. 1 was interpreted as being negative toward the Turnpike. An item analysis of the attitude scale was made to determine the effects of these different kinds of statements. A sampling of data on the respondents was selected at random on the basis of the percentage of the time they used the alternate route. Each item was classified as to whether it referred to the Turnpike or U.S. 1 and as to whether it was a favorable or unfavorable statement. In these classes, the score value was determined by the extent of agreement with the item itself by the respondent. Thus, a score value of more than 2 indicates agreement with the item, regardless of whether it is favorable or unfavorable. Conversely, a score value of less than 2 indicates disagreement with the statement. In tables 11 and 12, the data are shown for the male drivers.

As shown in table 10, regardless of t route upon which they were sampled, a regardless of the percentage of their trips the Turnpike, drivers responded positively favorable statements about the Turnpike. response to unfavorable statements, driv sampled on the Turnpike, regardless of th frequency of use, disagreed with the statements and, hence, provided a positive respon toward the Turnpike. Drivers on U.S. however, strongly agreed with the negati Turnpike statements if they were infreque users of the Turnpike and strongly disagre if they were frequent users. Thus, there w a significant shift in response to the negati statements by U.S. 1 drivers as a functi of the frequency with which they used t Turnpike.

Conversely, as shown in table 11, driv sampled on the Turnpike were essentia neutral in their responses to favorable statements about U.S. 1, regardless of whet they were frequent or infrequent users of Turnpike. Drivers sampled on U.S. 1, sponded to the favorable items positiv but less so if they used the Turnpike most the time. On unfavorable statements ab U.S. 1, agreement was consistent am drivers sampled on the Turnpike when qu tions were independent of the frequency w which the Turnpike was used. The U.S driver, however, had a definite shift fr disagreement with unfavorable statements if he were an infrequent user of the Turnpi to a positive response if he were a frequ user.

The significant aspect (tables 10 and 11) is the fact that drivers sampled on the Turnpike made consistent responses to statements about both routes, whether they were frequent or infrequent users of the Turnpike. The drivers on U.S. 1, however, shifted significantly in response to both types of statements, according to whether they were frequent or infrequent users of the Turnpike, but the major shift was in response to the unfavorable type of statement. These responses were to items that seemed to be the most discriminating type in the scale. Accordingly, drivers sampled on the Turnpike showed significant stability in their responses, regardless of the frequency of their usage of the Turnpike. The drivers sampled on the Turnpike consistently agreed with positive statements about the Turnpike and disagreed with unfavorable statements. He also significantly agreed with statements about the unfavorable characteristics of U.S. 1. Drivers sampled on U.S. 1, however, showed an adaptability to change in their responses, which was a function of experience with the Turnpike. Conclusion from the foregoing analysis is that the negative characteristics experienced by drivers on U.S. 1 in relation to the Turnpike caused drivers to shift to the Turnpike and minimized the probability of Turnpike drivers shifting back to U.S. 1.

## *Speed Volume and Traveltime Results*

On the Turnpike, speed and volume were determined on a sampling basis. Speed and volume measurements were made at 10-mile intervals, both northbound and southbound. A radar speed meter was mounted in the rear of a stationwagon that was parked on the shoulder. The speed meter was aimed at the approaching traffic at an angle of about 10°. This angle was larger than is recommended for the most accurate speed measurements, so some error is in these measurements. Normally, a sample of 100 vehicles was counted, and the time required for them to pass the counting station was also determined. Thus, it was possible not only to determine the speed distribution but also to estimate the hourly volume passing that point. The same procedure was followed on U.S. 1.

The cumulative speed distributions for the Turnpike are shown in figure 6—similar data on U.S. 1 are also included. Data were kept separate for the two directions in morning and afternoon sampling periods. The mean speed of these samples (Turnpike) was approximately 51.9 miles per hour, and the standard deviation was 9.1 miles per hour. The speed distribution is slightly negatively skewed. These speeds should be considered cautiously for, as has been shown by Shumate and Crowther (*10*), there is nonhomogeneity among spot speed samples. For U.S. 1, the cumulative speed distributions also are shown in figure 6. The mean of this sample was 43.7 miles per hour and the standard deviation was 10.3 miles per hour. This speed distribution is also negatively skewed but not so much as that for the Turnpike. The variability of speeds, from

**Table 8.—Relative frequency of trips of drivers sampled on Maine Turnpike and U.S. 1**

| Frequency of trips | Male drivers on— | | Female drivers on— | |
|---|---|---|---|---|
| | Maine Turnpike | U.S. 1 | Maine Turnpike | U.S. 1 |
| *Year* | *Percent* | *Percent* | *Percent* | *Percent* |
| Less than 1 | 4.7 | 7.3 | 12.4 | 13.6 |
| 1–11 | 44.1 | 53.4 | 42.6 | 47.3 |
| More than 12 | 51.1 | 39.4 | 45.1 | 39.0 |

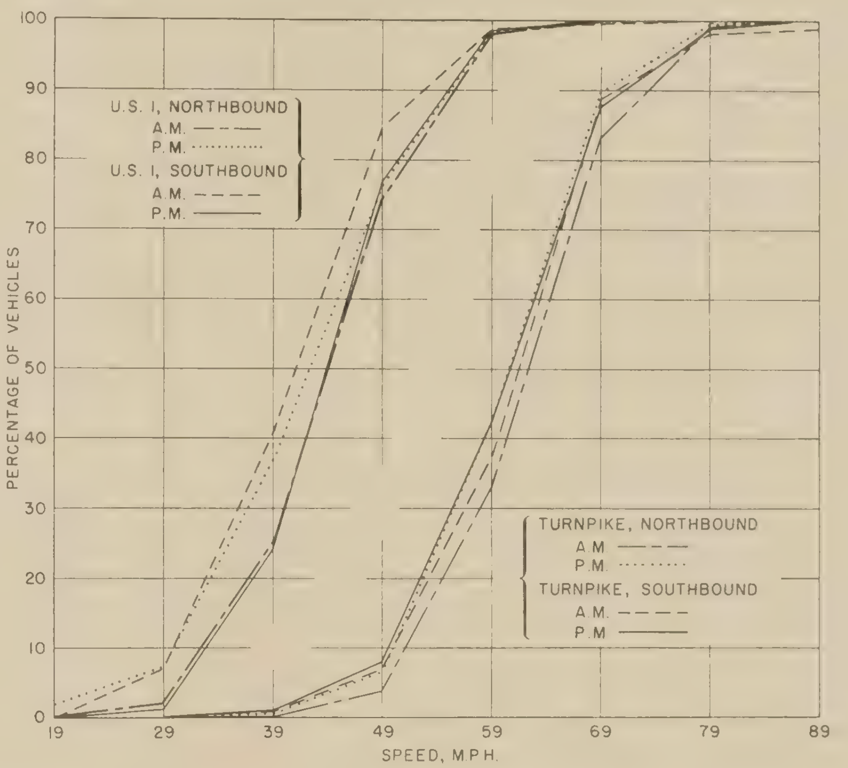


*Figure 6.—Vehicle speeds on both routes, cumulative distribution.*

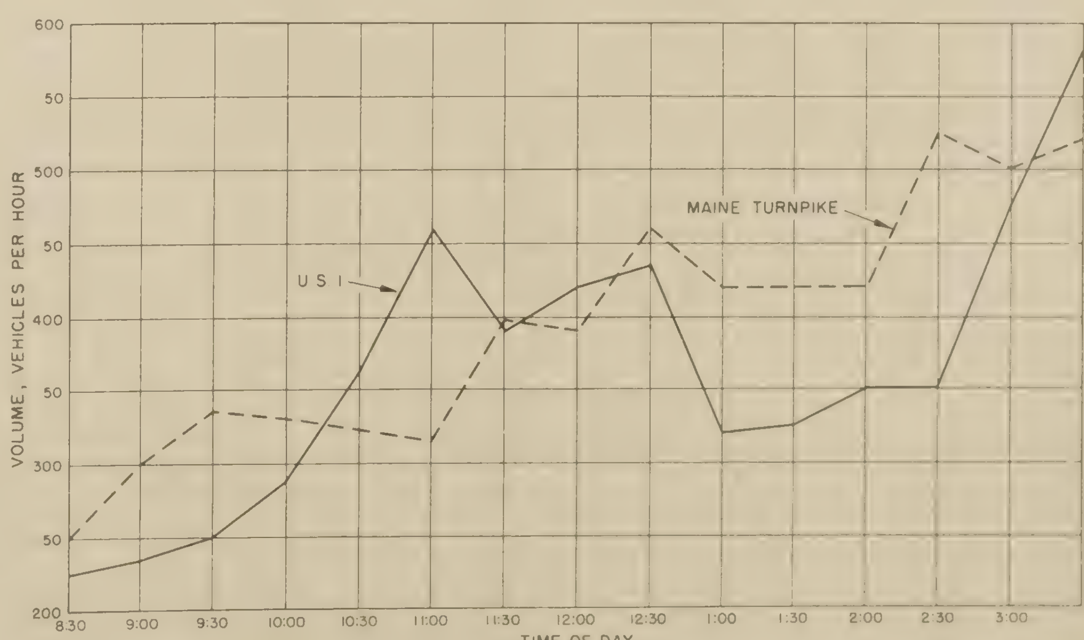


*Figure 7.—Calculated average hourly volumes on both routes.*

Table 9.—Mean attitudes toward the two highways as a function of the frequency of trips between South Portland and Kittery

| Trip frequency, per year | Attitudes by age and sex of driver: Less than 24, Male | Less than 24, Female | 24–34, Male | 24–34, Female | 35–44, Male | 35–44, Female | More than 45, Male | More than 45, Female |
|---|---|---|---|---|---|---|---|---|
| Maine Turnpike: | | | | | | | | |
| Less than 1 | | | 40.07 | 39.00 | 41.18 | | 36.93 | |
| 1–11 | 38.92 | 38.02 | 40.25 | 36.87 | 41.31 | 38.00 | 39.13 | 39.25 |
| More than 12 | 43.21 | 33.78 | 43.23 | 40.33 | 42.93 | 41.10 | 41.77 | 38.24 |
| U.S. 1: | | | | | | | | |
| Less than 1 | 32.65 | 28.48 | 34.54 | 30.32 | 32.08 | 26.33 | 29.98 | 27.19 |
| 1–11 | 32.96 | 31.15 | 33.05 | 32.29 | 31.34 | 29.63 | 30.00 | 28.47 |
| More than 12 | 31.32 | 31.54 | 34.97 | 32.79 | 33.68 | 29.12 | 30.72 | 30.36 |

Table 10.—Average item score of favorable statements for Maine Turnpike, by male drivers who use the Turnpike, either rarely or frequently

| Percent drivers use Maine Turnpike | Favorable statements: Maine Turnpike drivers | Favorable statements: U.S. 1 drivers | Unfavorable statements: Maine Turnpike drivers | Unfavorable statements: U.S. 1 drivers |
|---|---|---|---|---|
| Less than 24 | 2.45 | 2.14 | 1.71 | 2.58 |
| More than 75 | 2.54 | 2.44 | 1.70 | 1.70 |

Table 11.—Average item score of favorable statements for U.S. 1, by male drivers who use the Maine Turnpike, either rarely of frequently

| Percent drivers use Maine Turnpike | Favorable statements: Maine Turnpike drivers | Favorable statements: U.S. 1 drivers | Unfavorable statements: Maine Turnpike drivers | Unfavorable statements: U.S. 1 drivers |
|---|---|---|---|---|
| Less than 25 | 2.09 | 2.60 | 2.46 | 1.61 |
| More than 75 | 2.00 | 2.20 | 2.42 | 2.13 |

sample to sample and location to location, was much more on U.S. 1 than on the Turnpike. Therefore, the reliability of these summary statistics is questionable.

Volume of traffic was calculated for both the Turnpike and U.S. 1 on the basis of the same samples of the speed distribution. The average calculated hourly volume between the hours of 8 a.m. and 4 p.m. are shown for both routes in figure 7. The volume on U.S. 1 was not uniform over its entire 47-mile length; it was consistently larger at the more populous northern end. In addition, on U.S. 1, three counting stations were set up: One at each end of the study section and a third—a permanent counting station—about the middle of the test section. The calculated hourly volumes shown in figure 7 are approximately the same as those obtained at the counting stations. The volumes on the two routes were comparable and generally were parallel in their variations throughout the day.

Traveltime data were obtained from the trips made by the nine test drivers used for the GSR study. In these runs, the drivers were instructed to float with the traffic. This was done four times on each highway. Thus, 36 observations of traveltime were made on each route. Summary statistics are shown in table 10. The standard deviations indicate that on both routes the coefficient of variation in traveltime was 7 percent. This implies a variation for travel speed of approximately 17 percent on U.S. 1 and 14 percent on the Turnpike. Actually, the mean traveltime on U.S. 1 closely approximated the traveltime predicted from the mean speed of traffic on U.S. 1. On the Turnpike, however, the average speed of the test drivers was nearly 7½ miles per hour faster than that of traffic sampled on the Turnpike. This would indicate that the mean traveltime on the Turnpike for normal traffic may be up to 4½ minutes more than that shown in table 12. Finally, the maximum difference in time saved by selecting the Turnpike was calculated on the basis of the confidence intervals shown in table 12. In traveling between South Portland and Kittery a driver could obtain a maximum traveltime savings of 35 percent ±4 percent by driving on the Turnpike.

## *Tension Measurements*

The data for the nine test subjects were analyzed by determining the peak magnitude of GSR for observed interferences that caused

Table 12.—Traveltime between South Portland and Kittery on the Maine Turnpike and U.S. 1

| | Maine Turnpike | U.S. 1 |
|---|---|---|
| | *Minutes* | *Minutes* |
| Mean traveltime | 41.1 | 63.9 |
| Standard deviation | 3.61 | 4.31 |
| 95 percent confidence interval | ±1.25 | ±1.51 |

the driver to change his speed or positio[n] on the roadway. These interferences were (1) Other vehicles traveling in the same di[-] rection, (2) vehicles merging into path o[f] driver, (3) vehicles turning out of path o[f] driver, (4) traffic control devices, (5) pedestrian[s] on or near path of driver, (6) grades, (7[)] curves, (8) shoulder objects, and (9) opposin[g] vehicles. The fourth—traffic control de[-] vices—appeared on the Turnpike runs as we[ll] as those on U.S. 1 because highway mainte[-] nance operations were continually performe[d] on the Turnpike during the period in whic[h] GSR data were taken. Normally, advisor[y] speed signs were placed on the highway t[o] protect the maintenance crew, and these wer[e] included in the definition of traffic contro[l].

The magnitude of GSR per minute, whic[h] is the defined measure of driver tension, wa[s] statistically analyzed by the analysis o[f] variance. A summary of this analysis [is] shown in table 13. Significant difference[s] were recorded between the routes and sub[-] jects but not direction. These results a[re] similar to those reported previously (7[).] The comparison of tension between the tw[o] routes is shown for each subject in figure [.] The average tension differed considerabl[y] between subjects, but U.S. 1 generated si[g-] nificantly more tension for each driver tha[n] the Turnpike. The range of reduction [of] tension among this group of subjects o[n] the Turnpike was from 22 to 61 percen[t.] The overall average saving of tension b[y] taking the Turnpike was 46 percent.

Each route was divided into four, 10½-mi[le] sections. The tension data were analyzed [to] determine whether differences in tension we[re] generated between the sections of the t[wo] routes. As had been expected, no significa[nt] variations from segment to segment we[re] recorded on the Turnpike. Nor were signi[fi-] cant differences recorded between the sectio[ns] on U.S. 1. This was an unexpected findi[ng] because the highway and traffic from secti[on] to section of U.S. 1 had different charact[er-] istics and land use adjacent to the highw[ay] varied considerably. One reason for the la[ck] of difference was that the predomina[nt] interference in generating GSR arose direc[tly] from other vehicles in the driver's pa[th,] rather than differences in sections of the hi[gh-] way. Furthermore, when driving throu[gh] the more complex environments, all driv[ers] reduced their speed and thus reduc[ed] the probability of unexpected interferenc[e.] These compensatory changes may well ha[ve] eliminated any differences in GSR from t[he] different sections.

Table 13.—Analysis of variance of GSR data

| Source of variance | Sum of squares | Degree of freedom | Mean squares | F, ratio | Probability (F) |
|---|---|---|---|---|---|
| Routes | 308.65 | 1 | 308.65 | 305.59 | <0.01 |
| Subjects | 421.11 | 8 | 52.64 | 52.12 | <0.01 |
| Direction | 0.89 | 1 | 0.89 | 0.89 | (1) |
| Routes and subjects | 62.60 | 8 | 7.83 | 7.75 | <0.01 |
| Routes and directions | 28.59 | 1 | 28.59 | 28.31 | <0.01 |
| Subjects and directions | 34.87 | 8 | 4.36 | 4.32 | <0.01 |
| Residual | 44.38 | 44 | 1.01 | ---------- | ---------- |
| Total | 901.14 | 71 | ---------- | ---------- | ---------- |

[1] Not significant.

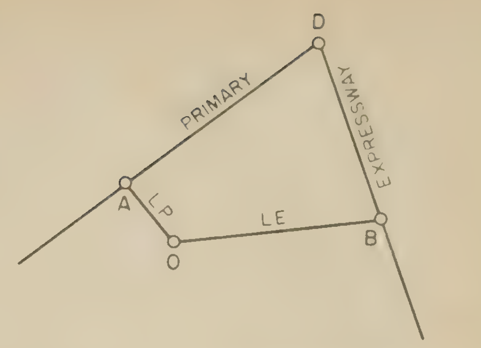


*Figure 9.—Geometry of diversion situation.*

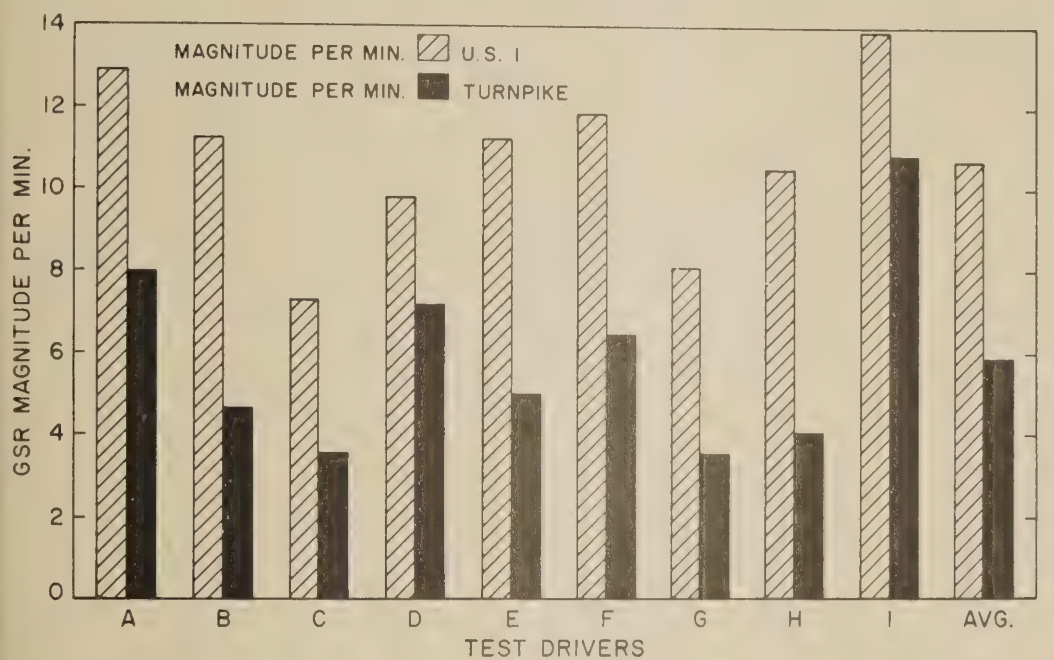


*Figure 8.—Mean tension generation on both routes.*

## *Interpretation of Results*

One of the main objectives of this study was to determine whether drivers had stable attitudes that correlated their choices between alternate highways. The results clearly established that they do. The attitudes of the users of the one highway differed significantly from the attitudes of the users of the other. Furthermore, the users of the Turnpike had significantly positive attitudes toward that controlled-access highway, and users of the rural primary had significantly negative attitudes toward the Turnpike. On the basis of the results, only a small proportion of drivers who hold a positive attitude toward the Turnpike actually will drive on the primary. Furthermore, in the alternate choice situation studied, an attitude scale appears to be strongly related to choice, much more so than any descriptive information about the characteristics of the drivers or their trips.

The results of the study clearly showed that drivers do evaluate their experiences on different highways. This evaluation is developed from a variety of elements in the highways they travel. Whether consciously or unconsciously, drivers weigh the different features of highways and combine subjective experiences into an overall evaluation. This is reflected in attitudes and predisposes drivers toward the choice of one highway instead of another. As a matter of fact, it is these attitudes that overwhelm all the specific short-term aspects of a particular trip and dictate the choice of route.

A third aspect of the study concerned the problem of attraction of traffic to an expressway. In several of the analyses it was very evident that attitudes shifted toward favoring the Turnpike. The most clear-cut example is the one in which the individual items on the scale were analyzed according to the route sampled. The significant finding was that the more drivers use the two highways, the more the primary suffers by comparison. The learning experience apparently increases drivers' awareness of the negative characteristics of the primary, so they become more dissatisfied with it. The direct experiences obtained in driving the primary-type of highway seem to force drivers onto a turnpike. Thus, the overall problem of the attraction of traffic to an expressway may be considered to arise from the direct experiences drivers have in driving it and any alternate. Because the expressway is perceived by drivers to have fewer negative effects than an alternate primary, a slow shift to the expressway occurs that seems to be motivated by a desire to escape the characteristics of the highway of older design.

Three major factors inherent in this type of situation may motivate a shift in favor of an expressway. First is the reduction in travel-time obtained by choosing the expressway. However, the results of the study showed no significant shifts in attitudes as a function of driving time. Drivers have the same attitude about both routes whether they are traveling for one-fourth hour or more than 2 hours even though, as a proportion of the total trip, savings in time gained from taking the expressway are decreased for long trips.

Second, in the original validation study, an item relative to the time savings to be obtained on an expressway was nondiscriminating; that is, regardless of whether people have positive or negative attitudes toward a turnpike they all agreed that time could be saved on it. Thus, although all drivers knew there was a time saving, it had no influence on their attitudes. As drivers know this to start with, time savings cannot be the basic cause of the shift in attitudes favoring an expressway. Some more subtle aspect of driving must be the source and it seems to be most sensitive to the negative characteristics of the primary.

Third, the direct cost of travel to the user is a factor. However, this does not seem reasonable, as the shift is in the wrong direction. That is, if cost of travel were a significant determinant of choice, a shift of attitudes away from a turnpike would occur, especially as trip frequency increased. However, the results clearly showed that, as the frequency of trips increased, there was an increasingly positive attitude toward the Turnpike and an even more likelihood that a driver would choose it. Also, two items were added to the scale that directly affect economic evaluation by the driver. These two items were actually the same except that one dealt with direct out-of-pocket cost, whereas the other dealt with cost per vehicle-mile.

The two statements read, "I would always travel the Turnpike between South Portland and Kittery if the cost were no more than" and alternatives were provided; for example, one increased the cost from 25 cents to $4, doubling over each of the five categories and another increased the cost from one-half cent a mile to 8 cents a mile. As might have been expected, the cost per mile item was nondiscriminating. Very few drivers had any



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idea of per mile cost. The result was that estimates on both routes were randomly distributed; a small proportion of drivers omitted a reply to the item. More surprising, actual out-of-pocket cost was also nondiscriminating. The reliability on the Turnpike was a little higher, possibly because the drivers had just received a toll ticket. Further, drivers sampled on both highways consistently reported to the interviewers that the cost of the Turnpike was irrelevant to their choice. This finding may simply mean that most drivers in this sampling were very indifferent to the expense of traveling the Turnpike at current cost levels.

Neither time savings nor direct costs seem to be dominant in determining the attraction of traffic to the turnpike. What seems to be required is something that drivers must learn by direct experience: Something related primarily to the negative characteristics of the rural primary type of highway. This leads inevitably to the consideration of the stresses arising in driving on the two routes. From the results of the GSR phase of the study discussed here, the tension aroused in the test drivers on the Turnpike was approximately one-half that generated on the primary. This tension was caused by interferences that had purely negative effects. It seems reasonable that shifts in traffic to an expressway facility is actually a forcing of drivers away from the primary route so that they can avoid its stress inducing characteristics. Stated more generally: Drivers make choices between routes to minimize the total stress to which they are subjected in driving. Thus, for the passenger car driver, the basis for scaling the benefits to be obtained from using an expressway are neither economic nor timesaving, but they are stress saving.

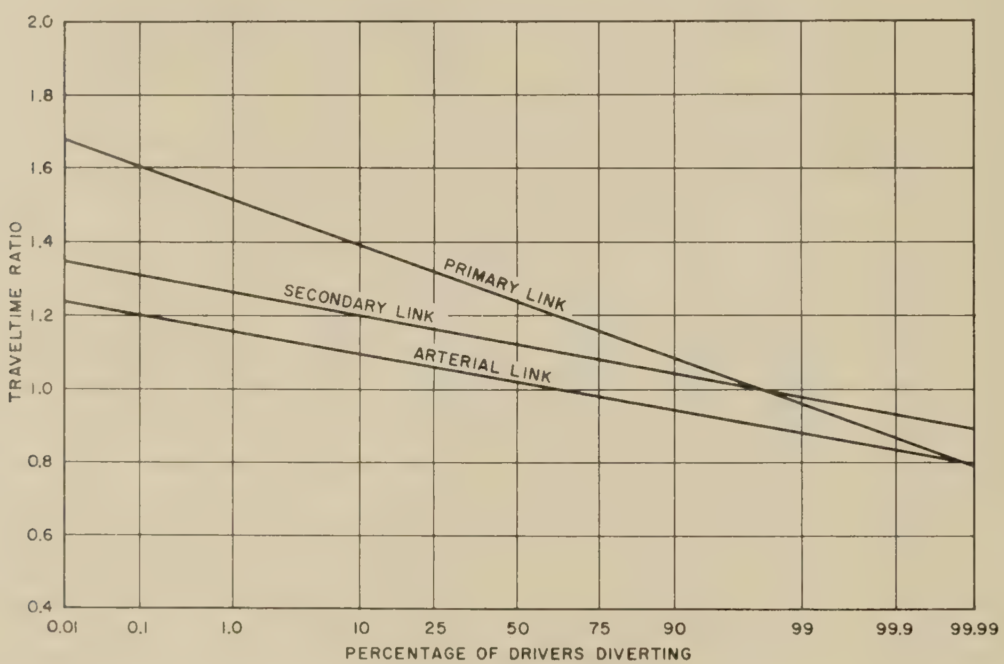


*Figure 10.—Theoretical diversion distributions, different connections from primary to expressway.*

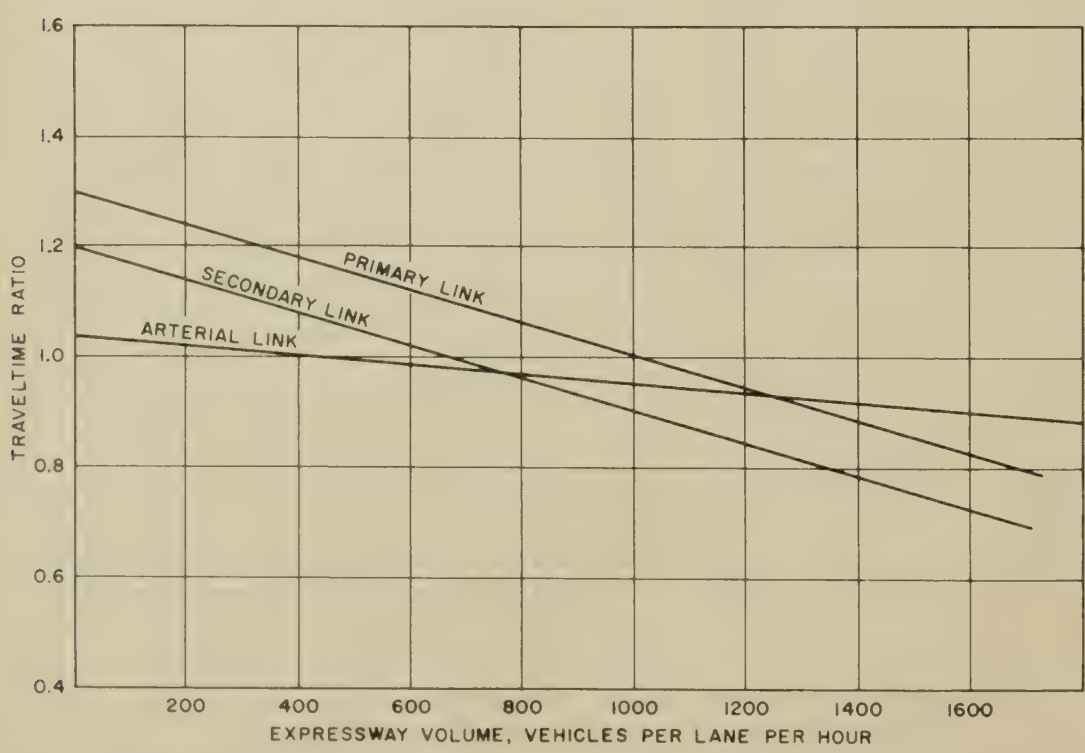


*Figure 11.—Expected traveltime ratio, 50 percent diversion, as function of expressway volume.*

The objective of minimizing the stress level in driving may explain two characteristics of the distribution of trips in the study results. First, the more frequent a trip, the more likely the drivers were to take the Turnpike. Second, the longer the duration of the trip, the more likely it was to be made on the Turnpike. Obviously, the total stress experienced on either route was a function of the particular properties of the route and the duration of the trip. That is, the total tension incurred is the integration of the unit stress over the duration of the trip. These tension inducing interferences occur randomly in time, the mean value being more on the primary highway than on the Turnpike. Because the variance in rate of occurrence of tension inducing interferences is high, the differences between the stress experienced on two highways in any short time interval will be unpredictable. Frequent repetitions or an increased sampling interval—that is, longer trips—will be required for the driver to reliably detect the difference between the alternates. By making frequent repetitions or longer trips, drivers will more likely detect the differences in tension on the alternate routes and thereby modify their choice behavior. The traveltime distribution and trip frequency data collected for this study conform to this hypothesis.

In simplest terms, the tension generated on any trip is some function of total traveltime and the frequency and intensity of stressing interferences. Using a relative measure of tension, a dimensionless constant is obtained. The relative stress obtained on any trip on a highway may be defined:

$$S=\frac{T_n}{T_R}\ (t) \qquad (1$$

Where,

$T_n$=magnitude of GSR per minute on highway $n$.

$T_R$=magnitude of GSR per minute on reference highway.

$t$=trip duration.

Thus, if tension generated on a freeway is used as a reference, a numerical value of relative stress can be calculated when the type of highway on which travel is done and trip duration are known. In this and previous studies (*6, 7*) it was shown that tension generated relative to the controlled-access highway was approximately 1.8 for a primary highway and 3.3 for an urban arterial. For a rural secondary highway having a low volume of traffic, the ratio probably is intermediate between these two, or about 2.5. Similarly the relative stress for any set of routes may also be computed by summing the stress for the components and the minimum stress route determined.

Relative to the problem of diversion to an expressway, this model suggests that: Drivers will divert to an expressway if the total stre experienced in reaching the expressway and on the expressway to the destination does not exceed that of the trip from origin to the alternate highway and on the alternate to the destination.

A general situation is shown in figure 9. Assume that an expressway, $E$, and a primary $P$, have a common terminus. Also, assume that the origin of a trip is located in the space bounded by the two routes so that there is a direct connection to either by link $L$. According to the hypothesis proposed herein, a driver will divert to the expressway to reach his destination if the total tension generated on the link, $L_E$, and the expressway, $E$, is equal to or less than the tension generated on the link, $L_P$, and the primary, $P$. When the origin lies on the primary and link $L$ is a perpendicular connection to the expressway, $E$ (fig. 9), then an inequality is obtained, as shown in equation (2), which defines the minimum separation between the primary and expressway for which 50-percent diversion will occur:

$$K_L \sin\theta + K_E \cos\theta \leq K_P \qquad (2)$$

The constants are the relative stress developed on each of the links. The solution of equation (2) is simply derived. Solving in terms of the cos $\theta$, a quadratic equation is obtained, the real root of which is shown in equation (3):

$$\cos\theta = \frac{\left(\frac{T_P}{T_E \cdot V_P \cdot V_E}\right) + \left(\frac{T_L}{T_E \cdot V_L}\right) \mathrm{A}}{\left(\frac{T_L}{T_E \cdot V_L}\right)^2 + \left(\frac{1}{V_E}\right)^2} \qquad (3)$$

Where,

$$\mathrm{A} = \sqrt{\left(\frac{T_L}{T_E \cdot V_L}\right)^2 + \left(\frac{1}{V_E}\right)^2 - \left(\frac{T_P}{T_E \cdot V_P}\right)^2}$$

$\frac{T_P}{T_E}$ = ratio of stress developed on a primary highway to that developed on an expressway.

$\frac{T_L}{T_E}$ = ratio of stress developed on the link between primary and expressway.

$V$ = mean speed in m.p.h. on appropriate highway.

It is further possible to define the travel distance ratio and the traveltime ratio. The equations are:

$$\frac{d_L + d_E}{d_P} = \sin\theta + \cos\theta \qquad (4)$$

Where,

$d_L$ = distance on link.
$d_E$ = distance on expressway.
$d_P$ = distance on primary.

and,

$$\frac{t_L + t_E}{t_P} = \frac{V_P}{V_L} \sin\theta + \frac{V_P}{V_E} \cos\theta \qquad (5)$$

Where,

$t$ = traveltime on each link.
$V$ = mean travel speed on each link in m.p.h.

By using the values for relative stress for three different types of highways and the travel speeds, equations (3), (4), and (5), may be solved, and the results will be as shown in table 14. The mean traveltime ratio decreases consistently as the stress inducing characteristics of the link increases.

Two other aspects may be considered by using this model. One is the variance in tension. In this analysis the relative stress is treated as a constant, although it is, of course, a mean value. On the basis of the data collected in this study, the variance of this ratio was 0.42. Using this ratio, it is possible to calculate the percentage of drivers diverting to an expressway, using equations (3) and (5). Normit plots are shown in figure 10 for the three examples. The other aspect concerns the volumes of vehicles the highways are carrying. As has been stated previously (*7*), the mean tension on an expressway increases linearly to about 1,400 vehicles per lane per hour. Beyond that volume tension increases very rapidly. On urban arterials (*9*) volume seems to have relatively little overall effect on tension generation. For primary highways, however, no data are available on the effect of increasing volume. If it is assumed that the effect of volume on the primary highway is similar to that on arterials, it is obvious that diversion to an expressway will vary solely with volume on that type of highway. The effect of increasing expressway volume on the traveltime ratio for 50-percent diversion is shown for the three types of links in figure 11. These curves were derived from equations (3) and (5). In all three examples, the traveltime ratio for 50-percent diversion decreases until, as volumes exceed 1,000 vehicles per lane per hour on the expressway, an actual time savings must occur before half the traffic diverts.

Note that the diversion curves developed from this special example do not conform to those developed from origin and destination studies in this corridor (*1*). The model predicts much more attraction than actually occurred; this was caused partly by the assumptions about the connection between primary and expressway routes. The choice points are not very direct for drivers within the Maine Turnpike and U.S. 1 corridor. Furthermore, a significant proportion of trips in that corridor are very short. For this kind of traffic, essentially trapped on U.S. 1, diversion to the Turnpike would gain the driver no detectable reduction in stress and, hence, little diversion would be expected.

However, for corridor trips of more than 10 miles and north-south oriented, considerably more diversion should occur than is shown in the general diversion curves (fig. 11). In this respect, Carpenter (*2*) examined through trips between Wells and Saco and reported that 30 percent of them diverted to the Turnpike, even though the traveltime ratio was approximately 1.22. However, on the basis of the link characteristics, the tension ratio for the alternate routes may be calculated and is approximately 1.09. This yields expected diversion of approximately 35 percent of these trips.

A reasonable conclusion is that whenever the alternates available are equally stress inducing, drivers will always choose the route that takes the least time. Therefore, it is not surprising that most drivers, when questioned as to why they chose the route, commonly used traveltime as a response. Not only is total stress directly related to traveltime but also, many of the alternates available offer no significant stress reduction. Furthermore, such trips are often so short that stress differences are hardly detectable. It is evident from results of the study reported, however, that drivers will actually tolerate a time loss, as well as a distance loss, if the total stress to which they may be subjected is perceptibly reduced. On the basis of this model, measures that reduce stress should cause both increases in trip length and trip frequency. As driving is a stressful and energy consuming task, each driver has a tolerance or limit beyond which the subjective cost of driving becomes excessive. The satisfactions to be gained by a trip are less than the energy required to achieve it. If trips are predominantly goal oriented, the stress imposed on a driver becomes the equivalent of a cost, the value of which is determined in part by the desirability of the goal. Conversely, reduction of this subjective cost by the addition of improved highways not only makes any given trip easier, but also makes lower priority goals more attainable. Thus, new travel is generated.

It would seem that the value of these subjective costs of driving could be determined experimentally, either: (1) by subjective scaling of simulated trips, which is a variation of game theory techniques, or (2) by subjective evaluation of actual trips made under well-defined conditions. However, a significant problem would remain: The measurement of the value a driver places on the need to make the trip. It is the ease with which the highway transportation satisfies this need that is the measure of the subjective benefits of the highway transport system. It would seem, then, that methods exist for quantifying the subjective costs of travel but not for subjective benefits. One thing, however, becoming increasingly clear is that, although passenger car drivers make rational evaluations of transportation, their benefit-cost ratio appears to have little in common with the economic criteria normally used in highway transport.

**Table 14.—Theoretical solution of expected diversion from a primary highway to an expressway**

| Link type | Separation between primary and expressway | Trip distance | Travel-time |
|---|---|---|---|
| | *Radians* | *Ratio* | *Ratio* |
| Primary | 0.99 | 1.39 | 1.24 |
| Secondary | .34 | 1.28 | 1.12 |
| Arterial | .13 | 1.12 | 1.02 |

## REFERENCES


(*1*) *Traffic Usage of Maine Turnpike*, by Glenn E. Brokke, Public Roads, vol. 28, No. 10, October 1955, pp. 224–230.

(*2*) *Proportionate Use of Maine Turnpike by Traffic Through Portsmouth-Portland Corridor*, by J. C. Carpenter, Highway Research Board Proceedings, 32d annual meeting, 1953, vol. 32, pp. 452–463.

(*3*) *Objective and Subjective Correlates of Expressway Use*, by E. Wilson Campbell and Robert S. McCargar, Highway Research Board Bulletin 119, *Factors Influencing Travel Patterns*, 1956, pp˙ 17–38.

(*4*) *The Effect of Travel Time and Distance on Freeway Usage*, by Darel L. Trueblood, Public Roads, vol. 26, No. 12, February 1952, pp. 241–250.

(*5*) *The Value of Travel Time for Passenger Cars: A Preliminary Study*, by Dan G. Haney, Stanford Research Institute, SRI Project No. IMU–3869, January 1963.

(*6*) *Characteristics of Passenger Car Travel on Toll Roads and Comparable Free Roads*, by Paul J. Claffey, Highway Research Board Bulletin 306, *Studies in Highway Engineering Economy*, 1961, pp. 1–22.

(*7*) *The Effect of Expressway Design on Driver Tension Responses*, by Richard M. Michaels, Public Roads, vol. 32, No. 5 December 1962, pp. 107–112.

(*8*) *Techniques of Attitude Scale Construction*, by Allen L. Edwards, Appleton-Century-Crofts, Inc., 1957.

(*9*) *Driver Tension Responses Generated on Urban Streets*, by Richard M. Michaels, Public Roads, vol. 31, No. 3, August 1960, pp. 53–58, 71.

(*10*) *Variability of Fixed-Point Speed Measurements*, by Robert P. Shumate and Richard F. Crowther, Highway Research Board Bulletin 281, *Traffic Volume and Speed Studies* 1961, pp. 87–96.


# NEW PUBLICATIONS

### *New Highway Map of the United States*

The Bureau of Public Roads has recently published a new highway systems map of the United States showing the National System of Interstate and Defense Highways, the Federal-Aid Primary Highway System, and the U.S. Numbered Highway System. The eight-color map is printed on a single sheet, measuring 42- by 65-inches. The scale of the map is 1:3,168,000; that is, 1 inch equals 50 miles, and it is drawn on the Albers equal-area projection. The actual map compilation was made by the U.S. Geological Survey, with the cooperation of the Bureau of Public Roads and the State highway departments.

The new map may be ordered under the short title, *Federal-Aid Highways*, from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C., 20402, at $1.50 per copy.

In addition to the three highway systems, the map also shows national forests, parks, and Indian reservations; all information represented is as of March 1, 1965. It should be noted that the map shows highway routes without regard to condition or completion, and many of the Interstate System routes are not yet built. Although the map will serve many useful purposes, it is not a touring or road-condition map.

For the 41,000-mile National System of Interstate and Defense Highways, commonly called the Interstate System, the locations of all routes are shown on the map, but only about one-half of the mileage is open to traffic at present. The System is scheduled for completion by 1972. The Federal-Aid Primary System totals about 227,000 miles (exclusive of the Interstate System); the majority of its routes are parts of the State highway systems. The U.S. Numbered System, 170,000 miles in extent, was devised by the American Association of State Highway Officials as a means for guiding travelers; it does not designate Federal-aid highways. However, most U.S. numbered routes are on the road systems eligible for Federal aid.

# Illumination Variables in Visual Tasks of Drivers

By[1] H. RICHARD BLACKWELL, Director, The Institute for Research in Vision, The Ohio State University; RICHARD N. SCHWAB,[2] Electrical Engineer, Office of Research and Development, U.S. BUREAU OF PUBLIC ROADS; and B.S. PRITCHARD,[3] formerly Research Associate at the Institute.

*The research reported in this article originally was conceived as a very limited study of the illumination levels needed for adequate performance of certain types of visual tasks that might be required of drivers. The authors originally planned to apply a general method, previously developed for visual tasks related to interior environments, to problems related to determining the illumination requirements for visual performance of driving tasks at night. However, the general method was not entirely satisfactory and new techniques for studying these problems had to be developed. The new techniques are explained.*

*Data developed from this study show that drivers experience many different degrees of difficulty in performing visual tasks that might be encountered in night driving—the degree of difficulty experienced being dependent to a large extent on the factors that influence the background luminance and the contrast of the task. A very comprehensive study of illumination and visibility variable would be required before any general understanding of the problems related to seeing while driving could be achieved, according to the authors. They note that the study reported in this article is not such a comprehensive work but that the results obtained should be useful for defining variables of interest for further research on highway lighting requirements. Some of the pitfalls that should be avoided in this further research are discussed.*

*On the basis of the data presented and the assumptions made, the authors estimate that 1.30 footcandles of illumination would be required for a driver to see a small black dog 200 feet away in the driving lane, and that 1.85 footcandles of illumination would be required for the driver to see a manikin of a young girl dressed in a long gray coat in the same location as the dog. An analysis of the data compiled suggests that contrast is more important than luminance in defining visual tasks.*

## *Introduction*

THE RESEARCH reported here is based primarily on the *visual task evaluator* (VTE) technique described by Blackwell (*1*)[4] in an earlier publication. In the work discussed in this article the technique has been extended from the earlier work on illumination levels required to perform certain types of visual tasks, occurring in interior environments, to those types of tasks that a driver might encounter in a street or roadway environment. The technique leads to an index of visibility based on the extent to which a practical visual task exceeds the borderline point between barely seeing the task and not seeing it at all. This borderline point is called the threshold of visibility, and the visual task may be that of seeing any object in the visual field that may be of interest to the observer when it is viewed against its normal background environment. An example might be seeing a pedestrian standing by the side of the road.

The degree to which the practical task exceeds the threshold point is measured by using the VTE to reduce the contrast that the object has with its background until the object is no longer distinguishable when viewed through the VTE. The amount that the contrast between any object and its background must be reduced to reach threshold may be used as an index of the extent to which that object exceeds the visibility threshold.

In the original use of the VTE technique, Blackwell used this measure of contrast reduction to define a value $\tilde{C}$ for each task studied. $\tilde{C}$ is defined as the physical contrast of a 4-minute, luminous, disk target having a visibility level equivalent to that of the task of interest—equivalence being specified as an equal amount of contrast reduction required to bring each task to its visibility threshold. The 4-minute disk target can be used, therefore, as a comparison standard, and the contrast ($\tilde{C}$) of this target can be used to determine the illumination level required for a selected level of task performance based on laboratory performance data (*2*). In the study reported here, the VTE defines illumination levels in terms of a performance criterion adopted as a standard by the Illuminating Engineering Society (*3*, *4*). Several special procedures were required in applying this method to the roadway environment; they are described herein and their validity established.

In addition, the VTE technique specified the relative visibility of a task under different roadway conditions, thereby allowing the examination of the effect of different aspects of illumination upon visibility. The particular aspects of the roadway illumination examined include the type of light source, the type of pavement, the spacing between light sources, the location of the task on the roadway, and the distance between the observer and the task. The relative visibility also has been related to background luminance and task contrast—the two physical parameters that determine task visibility within the roadway situation.

This study data showed a wide range in the degree of difficulty of different visual tasks that might be encountered on roadways at night. Indeed, different tasks require levels of illumination that range from moonlight to full daylight. The difficulty of a task depends, to a most significant extent, upon the factors influencing background luminance and task contrast, and these include all the factors that affect the amount of illumination striking a vertical object and its horizontal background. This implies that a very comprehensive study of illumination and visibility variables in roadway visual tasks is required before any

[1] This article is based on research conducted under Ohio HPS-HPR 1(32), *A Study of Highway Lighting*, by the Transportation Engineering Center, Engineering Experiment Station, The Ohio State University under sponsorship of the Ohio Department of Highways, and in cooperation with the U.S. Bureau of Public Roads. The project also was supported by the Illuminating Engineering Research Institute. A complete technical presentation of this research is available in reference (*5*).

[2] Mr. Schwab was formerly Research Assistant at The Institute for Research in Vision.

[3] Now dead.

[4] References indicated by italic numbers in parentheses are listed on page 248.

great understanding of the problem of seeing while driving can be achieved. The research reported in this article does not represent such a comprehensive study. It should be useful however, in defining the variables of interest for a more comprehensive study.

The primary data were collected at one test site in Hendersonville, N.C., where lighting variables could be controlled and changed readily. Other test sites also were used and the results obtained were very similar. For this article, only data from the Hendersonville test site were used to derive average values of the illumination required for roadway tasks because these were the most complete data (*5*). The lighting at the test site was assumed to be reasonably representative of general practice.

## *Summary*

Field tests were conducted on the visibility of a series of realistic objects located on a test roadway having lighting that could be changed. Visibility was assessed through the VTE technique. It was necessary to develop special techniques when applying the VTE to the study of roadway visual tasks. One technique involved the evaluation of the visual effect of disability glare. A special attachment for a photoelectric photometer was developed to do this, and the results were analyzed on the basis of laboratory visibility data. Another technique involved the use of a small part of the visual field when evaluating the state of visual adaptation. Physical measurements of the contrast in several roadway tasks were used to demonstrate that visual adaptation should be measured over a small part of the field next to the most visible detail of the object, rather than over a much larger area as previously had been used with the VTE procedure.

Visibility assessments were used first to evaluate the influence of such variables as objects, illuminants, viewing distance, location of object on the roadway, location of object to luminaires, luminaire spacing, and pavement material. Roadway tasks were concluded to vary grossly in difficulty and all the variables studied had important effects upon target visibility. The relative significance of the different roadway variables developed from the test data obtained should be estimated with caution, until a more complete, theoretical understanding of the causative factors involved has been obtained.

The data have also been used to determine the illumination needed to bring roadway visual tasks to a level of performance currently used in defining standards by the Illuminating Engineering Society. Average values required for visibility of objects at a distance of 200 feet were 1.30 footcandles to see a toy black dog and 1.85 footcandles to see a little girl manikin. Frequency graphs were prepared to illustrate the number of locations on the roadway providing this criterion level of task visibility for different levels of roadway illumination. In 99 percent of the locations on a lighted roadway, about 4 footcandles were required to see the dog and about 5 footcandles were required to see the manikin. Data were also prepared to illustrate the relative levels of illumination required to increase visibility to the criterion level at distances of more than 200 feet. When the distance was increased to 400 feet, an increase in illumination of about 2.5 times was required to see the dog and about 15 times to see the manikin. Care must be exercised in interpreting these illumination requirements. First, illumination levels depend critically upon the geometry involved. Hence, the illumination levels derived from the test data can refer only to roadway lighting installations of the same geometry. Second, many of the conditions encountered at the test site may not apply to real roadways. For example the pavement surfaces at the test site were unusually clean and unmarked. Third, the visibility criterion adopted by the IES for indoor tasks may not be applicable to roadway tasks. Analysis of the data, however, illustrates the value of studying the roadway visual problem by using the VTE technique. These tests produced useful information on the relative influence of different roadway variables and required illumination for selected objects at a selected level of visibility.

## *Equipment and Calibration*

The basic instrument used for the tests discussed herein was the original laboratory model of the VTE, which is shown mounted on the table at the right in figure 1; a Pritchard photometer is mounted on the tripod at the left. The extra lens beside the photometer control box is the *disability glare lens*, which is described subsequently. A schematic optical diagram for the VTE is figure 2. An observer looking through the VTE sees an image of the real world beyond the objective lens centered in the photometric comparator cube. Surrounding this central circular image of the external world is a doughnut-shaped *annulus* of uniform luminance produced by a lamp within the instrument. This annulus luminance is adjusted to equal the external world by a neutral absorbing wedge, labelled *annulus wedge*. This same lamp also illuminates a variable contrast wedge that is used to reduce the contrast of the image of the external world by a superimposed uniform light veil over the entire image seen through the instrument. The effect is similar to having a fog between the observer and the object viewed. The variable contrast wedge is constructed so that, at any given point on it, the total of the light transmitted through it from the external world and the light reflected from the internal light source are approximately a constant. For calibration purposes, a mirror, $M1$, is inserted to block the beam from the external

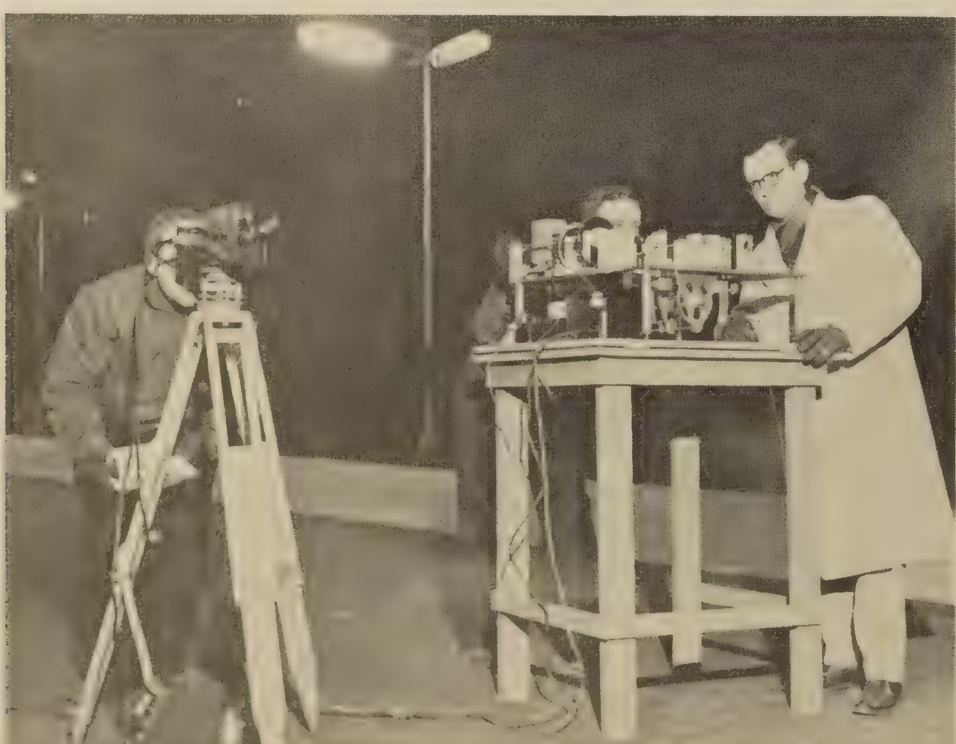

*Figure 1.—Equipment for outdoor visibility test. Pritchard photometer, left; visual task evaluator, right.*

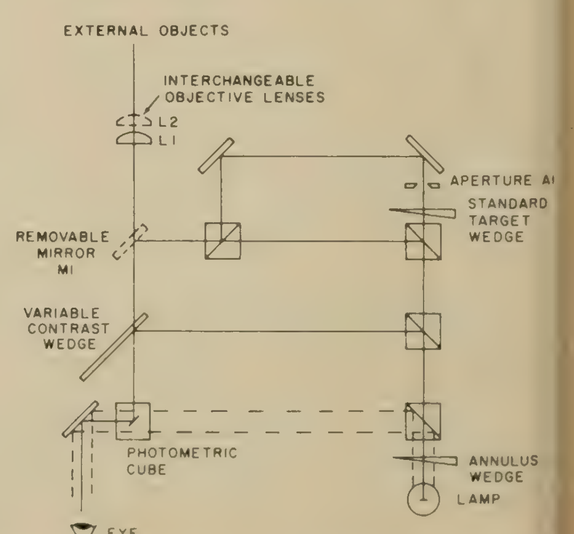


*Figure 2.—Diagram of original visual task evaluator.*

world and reflect a standard 4-minute disk target, the size of which is controlled by aperture $A1$; the contrast is controlled by the standard target wedge.

The calibration method used for the study reported here is summarized: In a photometric laboratory the transmission of the annulus wedge, $T_A$, was measured for all possible wedge settings. The luminance of an external object, when viewed through the VTE, was adjusted so that its luminance exactly matched that of the annulus when the annulus wedge was set for maximum transmittance. The luminance of the external object was then measured. This luminance, $B_o$, varied with lamp output and, therefore, had to be measured periodically. The next calibration determined the extent to which each setting of the variable contrast wedge reduced the contrast of the external scene. The extent of this reduction was termed *contrast rendition*, $CR$, and was measured by setting up an external object of equal luminance to the annular field when the variable contrast wedge was set for maximum transmittance. The transmittance, $T$, and the reflectance, $R$, were then measured photoelectrically by successively blocking the reflected and transmitted beam at different settings of the variable contrast wedge. The contrast rendition was defined as:

$$CR=\frac{T}{T+R} \qquad (1)$$

The final calibration measured threshold contrast for the standard target at several settings of the annulus wedge; this determined the background luminance against which the standard target was seen. With the removable mirror, $M1$, in place, the contrast of the standard target was varied by adjusting the standard target wedge until the 4-minute disk target was at the threshold of visibility. This process was repeated several times at each of several settings of background luminance by either reducing the contrast so that a visible target became invisible or by increasing the contrast of an invisible target until it became visible. Values of threshold contrast, $C_m$, thus obtained were plotted for different background luminances, and the smooth curve shown in figure 3 was drawn through them.

The values of $C_m$ used represent the average of three sets of calibration data obtained by Pritchard between the fall of 1958 and the spring of 1960 and the original calibration data obtained during the 1957–58 VTE work on interior tasks. It was originally believed that data should be analyzed in terms of calibration data obtained at the same time each practical task was measured (*6*). Because it was subsequently learned that, for a practiced operator, use of an average calibration curve was preferable, average calibration data were used in the study discussed here. Also, for a second, untrained observer, this average calibration curve seemed to apply very well. In fact, when two observers attempted to make readings on the same practical tasks, the average calibration data (fig. 3) applied more reasonably to information obtained by the second observer than the calibration curves obtained directly by him. Therefore, the Pritchard calibration data were used in analyzing all VTE measurements, regardless of the observer.

## *Field Procedures*

The original procedure for use of the VTE consisted of the following steps. First, the operator viewed the practical visual task through the VTE and centered the image of the task in the field of view. The variable contrast wedge was then set for maximum transmittance, the objective lens, $L2$, was inserted, and lens $L1$ was removed. Lens $L2$ produced a completely out-of-focus image of the external world, subtending exactly 2 degrees of visual angle. The resultant blurring of the external world image integrates the luminances within the field of view and produces the appearance of uniform brightness over the central circular area of the photometric cube. The brightness of the surrounding annulus, controlled by the annulus wedge, was then easily adjusted to match the average brightness of the central area. The average luminance, $\overline{B}$, of the task was defined from the calibration described earlier as:

$$\overline{B}=B_O\times T_A \qquad (2)$$

The annulus wedge was left in the position of the photometric match. Objective lens $L1$ was substituted for $L2$ to form an in-focus image of the external world. The variable contrast wedge was adjusted until the visual task of interest was reduced to threshold visibility and the contrast rendition, $CR$, read for that setting of the variable contrast wedge. The equivalent contrast, $\tilde{C}$, was then defined as,

$$\tilde{C}=\frac{C_m}{CR} \qquad (3)$$

where, $C_m$ is the value read from figure 3 at a background luminance equal to $\overline{B}$. $\tilde{C}$ measures the intrinsic visual difficulty of the task because of physical variables such as object size, shape, luminance contrast, and chromatic contrast. $\tilde{C}$ does not reflect the difficulty of the task related to the background luminance present because its use in establishing the illumination requirements of different visual tasks requires $\tilde{C}$ to be independent of the illumination level present at the time the visual task is assessed.

After a value $\tilde{C}$ for a task has been obtained, the background luminance, $B_r$, that is required for performance of the task at a selected level of adequacy can be determined. As mentioned in the introduction, the performance criterion used in this article was based on certain assumptions of what constitutes adequate performance. These assumptions were: (1) That the task—detecting the presence of the standard object—be performed 99 percent accurately by trained laboratory observers; and (2) that information about the task be derived at the rate of five assimilations per second. To compensate for the difference between use of laboratory observers and so-called *commonsense seeing* and other variables, such as lack of complete information as to where and when the object was to appear, a field factor of 15 was introduced to adjust the laboratory performance data upwards. Justification for using these assumptions has been discussed by Blackwell (*1*, *4*).

Based on the preceding assumptions, laboratory performance data can be obtained to relate contrast threshold to background luminance, $B_r$, required to reach a certain performance level. Such a curve is shown as the solid line in figure 4. The ordinate corresponds to the logarithm of $\tilde{C}$ and the abscissa to the logarithm of $B_r$; therefore, after $\tilde{C}$ was measured, $B_r$ was obtained by reading the curve in figure 4

The required illumination, $E_r$, was computed from the value of $B_r$. In the roadway study, the relationships were solved:

$$E_r=B_r\times\frac{\overline{E}_h}{\overline{B}} \qquad (4)$$

Where,

$\overline{E}_h$ = the average horizontal illumination provided by the roadway lighting system.

$\overline{B}$ = the average luminance of the task as defined in equation (2).

The logic of equation (4) is explained in the rest of this paragraph. The roadway lighting system producing average illumination, $\overline{E}_r$, provides luminance $\overline{B}$ for a particular task at some point along the roadway. If the visual task assessment showed that a luminance, $B_r$, was required to perform the task at the selected level of adequacy, the ratio $B_r\div\overline{B}$ represents the extent to which the lighting system produced an adequate luminance. *Assuming no change in illumination geometry*, the required average illumination, $E_r$, would equal the actual average illumination, times the ratio $B_r\div\overline{B}$. It cannot be overemphasized that no change in illumination geometry must be assumed. Obviously, in a three-dimensional situation such as in roadway lighting and viewing, unless the illumination geometry is maintained precisely, a change in illumination level could alter task contrast and, hence, task visibility. The assumption used in writing equation (4) is that, in effect, the system of roadway illumination is on a dimming control. The illumination could, therefore, be set at $E_r$ to provide a selected level of visual performance for any task of interest by adjustment of the illumination up or down to the required level.

## *Disability Glare*

In order to apply the VTE technique to a roadway environment, a special method was employed to allow for the deleterious effects of disability glare on task visibility. The field of view of the VTE was limited to the central 2-degree area around the object.

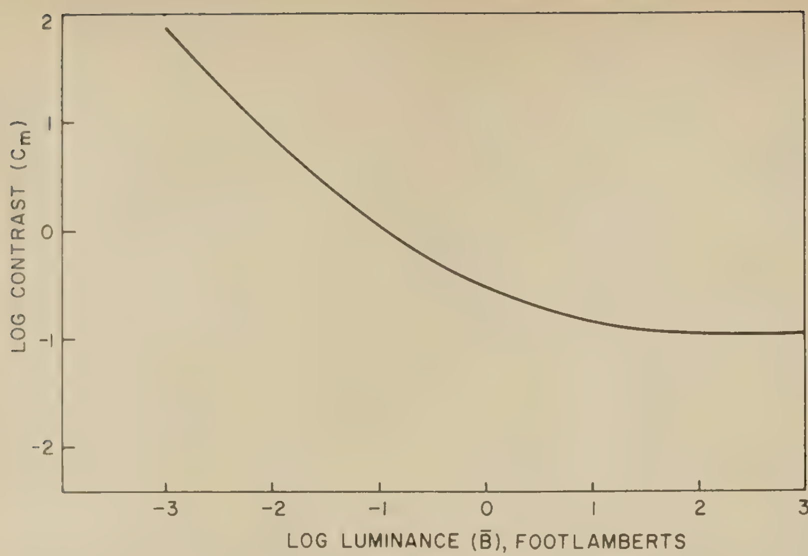


*Figure 3.—Variation in threshold contrast as a function of background luminance.*

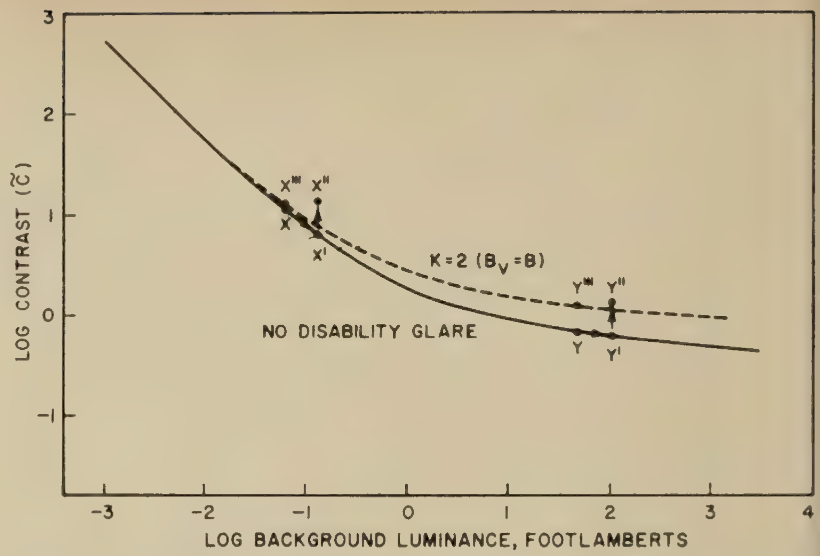


*Figure 4.—Background luminance as a function of target contrast for standard level of visual performance: No disability glare solid curve; disability glare, dashed curve.*

Because the main sources of disability glare were the luminaires located outside this area, these effects were not included in the initial visibility assessment. It might have been possible to enlarge the area viewed through the VTE by changing lenses; however, because of the physiological differences of individual observer's reactions to glare, it seemed preferable to use a calculation method.

The method used depended on the effects of disability glare described in an earlier publication by Blackwell (*2*). Disability glare can be assessed in terms of a uniform luminance veil, $B_v$, that is superimposed over the entire field of view and is equivalent in its effect on visibility to all the discrete sources of luminance in the field. The effects of disability glare are shown in terms of the standard performance curve in figure 4. The value of veiling luminance, $B_v$, that is equal to the disability glare effect increases the direct luminance, $B$, to $B_e$, the effective luminance, where:

$$B_e = B + B_v \qquad (5)$$

The increase in luminance produced by disability glare is shown for two initial values of $B$, designated $X$ and $Y$. The task contrasts required for the standard level of performance are indicated by the location of the points $X$ and $Y$ on the solid curve. At the corresponding value of $B_e$, the contrast required for the eye to see at a selected level of visual performance is decreased by an amount equal to the differences between points $X'$ and $Y'$ and the original points $X$ and $Y$.

Disability glare has a second effect, that of reducing the task contrast present; this effect may be described as:

$$C' = C \times \frac{B}{B + B_v} \qquad (6)$$

Where,

$C'$ = the apparent task contrast in the presence of the disability glare, $B_v$.

$C$ = the initial contrast of the task.

Because disability glare decreases task contrast, the physical value of task contrast must be increased to provide the contrast needed for adequate performance. This effect is shown in figure 4 by a comparison of the location of the points $X''$ and $Y''$ with those of points $X'$ and $Y'$. The horizontal displacements of $X'$ from $X$ and $Y'$ from $Y$ are precisely the same on a double logarithmic plot as the vertical displacements of $X''$ from $X'$ and $Y''$ from $Y'$, equations (5) and (6). The values of $X''$ and $Y''$ are the contrasts required at the luminance values $B$ rather than the values $B_e$, so they must be plotted at the locations $X'''$ and $Y'''$. The constructions used in locating the points $X'''$ and $Y'''$ may be used for all points falling on the standard performance curve. The dashed curve (fig. 4) represents the resultant effect of disability glare on the standard performance curve when $B_v$ is equal to $B$. A disability glare constant, $K$, was used to define the amount of glare present as:

$$K = \frac{B_v + B}{B} \qquad (7)$$

In figure 4, $B_v$ was assumed to be equal to $B$, so $K$ is equal to 2.

The method for determining the value of $B_r$ in the presence of disability glare requires use of the dashed curve in figure 4, rather than the solid curve. Obviously, for a specified ordinate value of $\tilde{C}$, the luminance required to attain a specific level of performance is higher when disability glare is present than when it is absent. For convenience, the background luminance required when disability glare is present is referred to by the notation $B_r'$. Similarly, $E_r'$ is used to refer to the required illumination in the presence of disability glare. For a fixed value of $K$, the larger the value $B_r$ and $E_r$ were originally, the more $B_r'$ will exceed $B_r$ and $E_r'$ will exceed $E_r$.

To compute the values of $E_r'$, a measure of the value of $B_v$ in each roadway situation was required. Individual values of the illumination produced at the eye by each glare source could have been measured for each situation and a value for $B_v$ computed; however, the work for this type of approach seemed prohibitive. A photometric device for direct measurement of $B_v$ was required.

Some years ago, Fry (*7*) described a device consisting of a wide-angle lens that forms an image of the entire world out to 90 degrees on either side of straight ahead and an absorbing photographic mask that selectively transmits illumination coming from different points in the field in different proportions to satisfy an empirical formulation for disability glare. Such a device could be utilized as the objective lens of a photometer so that the summation could be performed photometrically. The device, although simple in principle, was exceedingly difficult to construct. The image produced by the wide-angle lens was distorted

*Figure 5.—Night view of three targets at test site.*

and the photographic mask, therefore, had to have the same spatial distortion built into it. Furthermore, the transmission of light through the mask was required to change over a range of from 10,000 to 1. It was not possible to achieve this range in one piece of photographic material. Therefore, two separate masks were required, each having a central opaque spot that excluded all light from the central 2-degree area and symmetrically graduated density that radiated out from the central spot. An improved design for a disability glare lens has been described by Fry, Pritchard, and Blackwell (*8*).

In actual use, the Pritchard photometer was pointed at a visual task of interest and a value of average task luminance was obtained. The ordinary objective lens was then removed, and the disability glare lens was substituted for it, without moving the photometer. A photometric reading was made using each of the two masks. The effective luminances obtained were added to equal $B_v$. The value of $K$ was computed from equation (7). After computing the value of $K$, allowance was made for the 7-percent component of disability glare when the eye was exposed to a field of uniform luminance, as shown by Moon and Spencer (*9*). The visual performance data represented by the solid curve in figure 4 contain this magnitude of disability glare. Thus a value of $K'$ was computed from the relation:

$$K'=\frac{B_v+B}{B}-0.07$$
$$=K-0.07 \qquad (8)$$

The value of $K'$ was used to construct contours such as the dashed curve (fig. 4), because the solid curve represents a baseline with the 7-percent disability glare already present. These procedures suffice for the computation of values of $E_r$ and $E_r'$ in practical roadway situations.

## *Relative Visibility Calculations*

To arrive at an understanding of how different illumination variables affect visibility, it was necessary to obtain a measure of the relative visibility of a specific task under different conditions. Such a measure is the relative visibility factor ($RVF$), which is defined as:

$$RVF=\frac{\tilde{C}}{\overline{C}} \qquad (9)$$

Where,

$\tilde{C}$ = the equivalent contrast of the standard target.

$\overline{C}$ = the value of target contrast for the standard level of visual performance at the luminance $\overline{B}$ (solid curve in fig. 4).

The value of $RVF$ is an indication of the difficulty of the visual task in terms of object size and shape, luminance and chromatic contrast, and average task luminance. $RVF$ thus differs from $\tilde{C}$ only in the significance of the absolute values of the two quantities and in the fact that it reflects the effect of the level of background luminance, whereas $\tilde{C}$ does not. A value of $RVF$ equal to unity signifies that the roadway illumination provides exactly the level of visual performance represented by the standard performance curve. $RVF$ values larger than unity show that the task is more visible than required to meet this performance criterion, whereas $RVF$ values less than unity show that the task is not as visible as is required.

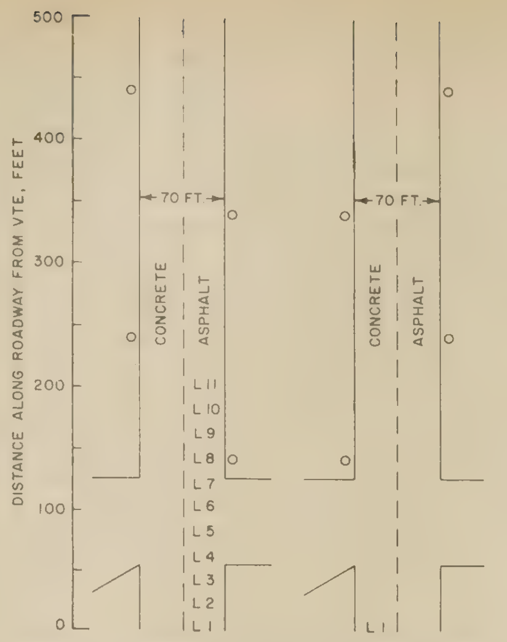


*Figure 6.—Plan of layout at test site.*

As for the required illumination values, allowance for disability glare can be accomplished by adjusting the standard performance curve. The relative visibility factor in the presence of glare ($RVF'$) is defined as:

$$RVF'=\frac{\tilde{C}}{\overline{C}'} \qquad (10)$$

Where,

$\overline{C}'$ = the value of $\overline{C}$ adjusted for disability glare.

## *Visual Tasks*

It was believed desirable to utilize realistic roadway tasks that might be fairly representative of collision type situations rather than simplified tasks such as black disks that frequently have been used in similar studies. For primary use, a toy black dog and a manikin of a 12-year-old girl were selected. The manikin was outfitted in a loose-fitting, full-length gray coat having 20-percent reflectance. In addition, one series of measurements was made on seven other objects: (1) Black disk, 1 foot in diameter; (2) manikin wearing a coat having 60-percent reflectance; (3) toy, pink poodle dog; (4) black automobile without lights or retro-reflectors; (5) yellow highway cone marker; (6) bicycle lying flat on the roadway; and (7) red brick. The manikin, in the coat having 20-percent reflectance, and the two dogs are shown in figure 5, at the test site.

## *Roadway Installations*

The plan layout of the test facility is shown in figure 6. The right half of the roadway was paved with asphalt and the left with concrete. The surrounding ground sloped off toward the right and toward the far end of the road. The area was wooded, particularly toward the right side. A white frame house was in the woods at the far end of the street. Luminaire poles were spaced at 100-foot intervals on each side of the roadway, as illustrated. Five poles were used for the first two series of tests; for the third series, a sixth pole was added at the end of the roadway, 230 feet beyond the last luminaire on the left side. Each pole had a 4-lamp fluorescent luminaire mounted transverse to the curb, a 400-watt mercury lamp, and an incandescent luminaire that could accommodate either 6,000- or 15,000-lumen lamps. Only one type of luminaire was used on a specified series of measurements.

The VTE was set up in the middle of the driving lane on the appropriate side of the pavement to be used in the particular series of measurements, as shown in the elevation layout in figure 7. The first operating luminaire, shown as a small circle (fig. 6), was always on the same side of the roadway as the measurement booth. The luminaires were spaced 200 feet apart on each side of the roadway, in staggered locations. In one test, only half the luminaires were used, and the spacing between them on one side of the road was 400 feet. Because the basic arrangement of luminaires was staggered, the spacing for this one test was designated as 200 feet.

## *Experimental Data*

Three series of tests were made at Hendersonville, N.C. For convenience, these tests have been designated as test series I, II, or III. Each series is described separately because several important changes in the experimental technique were made as the work proceeded. The results have been analyzed for all three series of measurements and are included in the analysis of data.

### Test series I

In the first series of measurements made at Hendersonville in the spring of 1959, the visual task, type of light source, type of pavement, spacing between luminaires, and lane in which the task was located were varied in a systematic way. After the data had been analyzed, certain questions arose as to the validity of the method previously described for using the VTE. It seemed from the data that the tasks became more visible as the viewing distance was increased. This effect was opposite to that expected on the basis of the object's decreasing angular size. The annulus brightness had been matched to the average brightness of an out-of-focus image, thereby equating the average luminance of the field of view to the luminance of the internal light source. It was suggested that the method used might have distorted the experimental data and produced these unexpected results. This would have been true if the eye had been adapted to the brightness

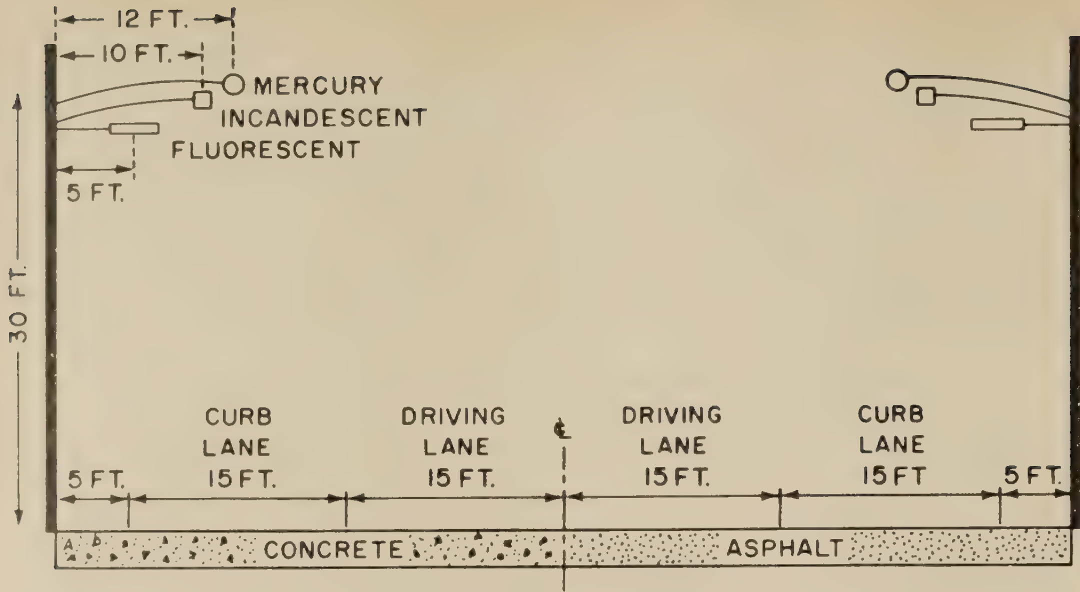


*Figure 7.—Elevation layout of Hendersonville test site.*

at a point in the visual field near the object rather than to an average field brightness. The second series of measurements were designed to investigate whether the procedure for measuring field brightness had introduced an error into the results.

### Test series II

Two approaches were used in the investigation of the VTE procedure. First, a new procedure was developed that would be free of the suspected error. On the basis of earlier work (*2*), measurements were made of the physical luminances expected to influence visibility under the different conditions. Then predictions were made as to the relative visibility of the several objects viewed at different distances, and the two VTE procedures were used to make measurements of these objects. These measurements were analyzed in relation to the predicted visibility values. Most of the variables studied in test series I also were employed in use of the new procedure. An observation distance of 180 feet was employed.

The new VTE procedure, designated the *final* procedure, included several steps: Before setting the annulus wedge carefully, the variable contrast wedge was adjusted to ascertain the part of the object that disappeared last and, hence, was initially most visible. The VTE then was directed so that the background adjacent to the most visible part of the object was at the edge of the circular inner field of the photometric comparator in juxtaposition to the annular field. Objective lens $L1$ was left in place so that the image of the external world was in focus. Then, the annulus wedge was set to match the brightness of the selected area of the background, and the variable contrast wedge was set for maximum transmittance. From this point on, the procedure followed was exactly the same as in test series I. Because the *blurring lens* was not used in the *final* procedure and because of the resultant nonuniformity of the background luminance, it was somewhat difficult to obtain a photometric match between the small part of the background and the surrounding annular field. Otherwise, the procedure for test series II caused no difficulties.

Physical measurements were made of the luminance of the most visible part of the object and its adjacent background, as determined by the *final* VTE procedure. The Pritchard photometer was used; its aperture restricted the field to a diameter of 10 minutes of circular arc. Photographs were taken of the target under each different condition, so the visual area of the relevant element of the target could be computed with precision. Comparison of the values of $\tilde{C}$ obtained from the two VTE procedures showed equivalent results for all targets at distances of less than 220 feet. At longer distances, the value of $\tilde{C}$ obtained from the original procedure was substantially larger than the values obtained from the *final* procedure.

The relations of $RVF$, $RVF'$, and $\tilde{C}$ were judged from the data shown in figure 8; the ordinate scale on the left of the figure shows the values of $RVF$ and $RVF'$ and the ordinate scale on the right shows the values of $\tilde{C}$. The role of disability glare at different locations in the roadway installation can be ascertained

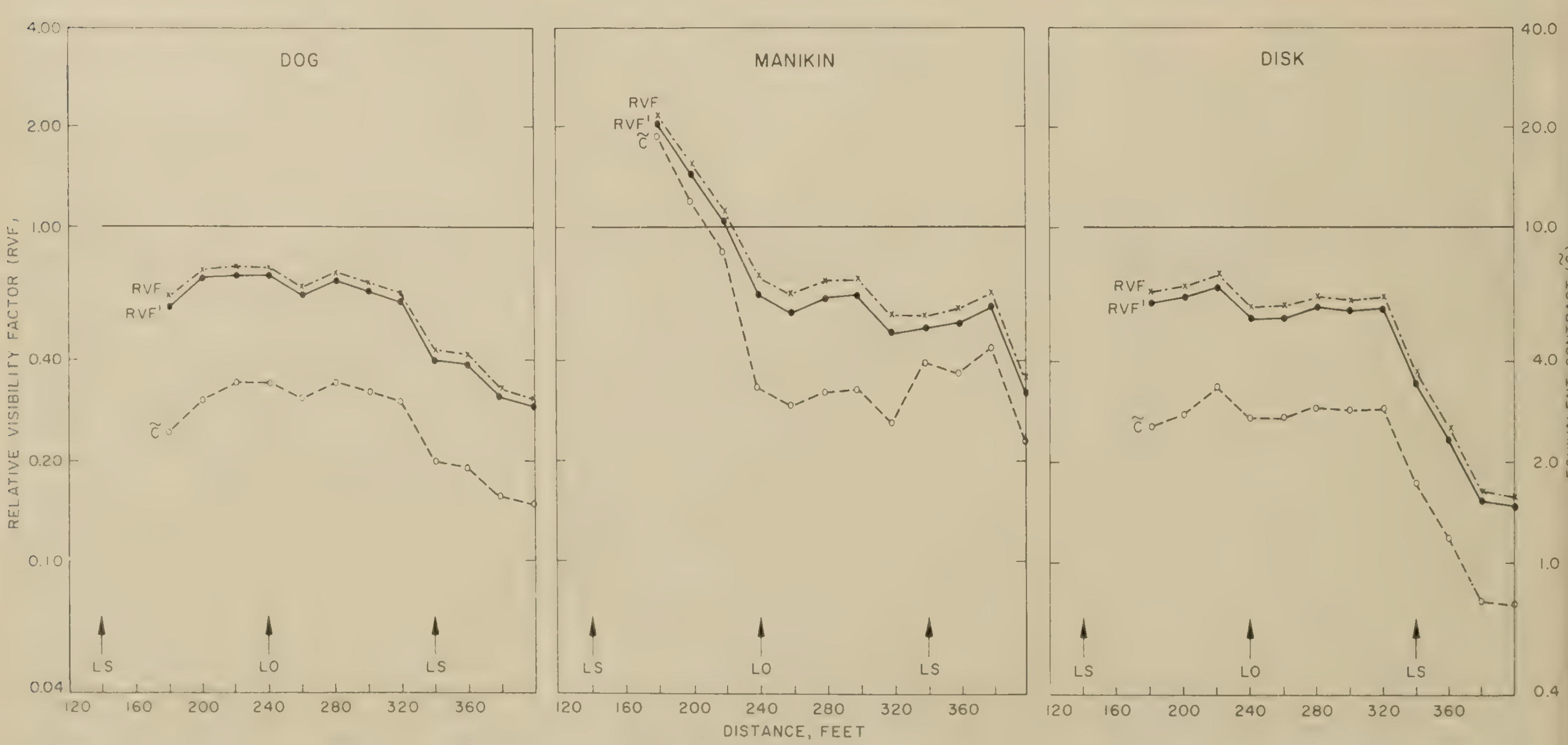


*Figure 8.—RVF, RVF′, and $\tilde{C}$ as functions of visibility distance in relation to location of luminaires. Horizontal line represents level of visibility required for standard performance level.*

by comparing the values of $RVF$ and $RVF'$. As stated previously, the role of variations in pavement luminance may be evaluated by comparing values of $\tilde{C}$ and $RVF$. In figure 8, the luminaire locations are indicated by: $LO$, luminaire on the opposite side of the roadway, and $LS$, luminaire on the same side of the roadway as the task. Values of $RVF$, $RVF'$ and $\tilde{C}$ change according to distance in much the same way (they are parallel), thus establishing that these variations in background luminance as a function of distance were not important causative factors in determining object visibility at different distances. In almost no test did the values of $RVF'$ exceed unity. Therefore, the lighting system was not producing a level of visibility sufficient to satisfy the performance criterion.

**Test series III**

The third series of test measurements were made because of a desire to obtain additional data under the VTE procedure used in test series II. In particular, it seemed desirable to study the relationships between visibility indices and distance for illumination geometrics other than those obtained in the earlier measurements, in which the VTE was always located at position $L1$, as shown in figure 6. During test series III, the VTE was located at each of the 11 positions, $L1$ to $L11$. At each position, the dog and manikin were moved so that the distance between the object and the observer ranged from 180 to 400 feet. All these measurements were made on asphalt pavement, under 15,000-lumen incandescent luminaires at 100-foot spacings, and the objects were located in the driving lane.

## *Analysis of Data*

The focal point of interest for the test series II was to test the extent to which the original and *final* VTE procedures yielded visibility indices in agreement with expectations based upon physical measurements. The luminances of the most visible detail of each object and its background for each of several distances were measured. These data were used to compute a measure of the target visibility expected to exist. The procedure involved the following described steps: The luminance contrast was computed from the relation proposed earlier by Blackwell (*10*):

$$C=\frac{B_t-B_b}{B_b} \tag{11}$$

Where,

$B_t$=object luminance.
$B_b$=background luminance.

Then the contrast was adjusted by a factor to allow for the fact that the area of the object differed under different conditions. The factor $F$ was defined as:

$$F=\frac{\bar{C}_s}{\bar{C}_a} \tag{12}$$

Where,

$\bar{C}_s$=threshold contrast for a 4-minute luminous disk.
$\bar{C}_a$=threshold contrast for a target having the angular size of the element of greatest visibility.

Values of $\bar{C}_s$ and $\bar{C}_a$ were read for the particular background luminance, $B_b$, from the visual threshold curves of Blackwell (*2*) for 1-second exposure duration. These threshold data are for circular objects. In making these calculations, noncircular elements were considered to have the same threshold contrast as circular objects of equal area. A value of contrast obtained in equation (11) was then adjusted by using equation (12) to allow for differences in target size as:

$$C'=CF \tag{13}$$

Generally, good agreement was obtained between the physically measured value of $B_b$ obtained with the Pritchard photometer and the value of $\bar{B}$ obtained from the annulus wedge settings on the VTE. However, there were tests in which the two values disagreed considerably. This was particularly true of the data obtained under the *original* VTE procedure. It seemed more reasonable to conclude that the value of $\bar{B}$ was in error because of the comparative difficulty and uncertainty in visual photometric measurements. Errors in $\bar{B}$ would be expected to alter values of $\tilde{C}$ as related to $C'$. When $\bar{B}$ was too large, $\tilde{C}$ would be reduced because the veiling luminance would be larger than it should be. Conversely, when $\bar{B}$ was too small, $\tilde{C}$ would be spuriously large. A correction factor $F'$ was developed where:

$$F'=\frac{\bar{C}_{\bar{B}}}{\bar{C}_{B_b}} \tag{14}$$

Where,

$\bar{C}_{B_b}$=threshold contrast for an object having the area of most visibility at $B_b$.
$\bar{C}_{\bar{B}}$ =threshold contrast for the same element at $\bar{B}$.

These threshold values were also read from the same threshold curves used for equation (12). Then the corrected computed equivalent contrast of a traget element was determined:

$$C''=C'\ F' \tag{15}$$

The correction factor $F'$ reduced $C'$ whenever $\tilde{C}$ was spuriously small, or increased $C'$ whenever $\tilde{C}$ was too large. Thus, in effect the correction was being made in the wrong quantity. This should be remembered when considering values of $\tilde{C}$ as related to $C''$.

Values of $\tilde{C}$ obtained under the original and *final* VTE procedures were than evaluated. These values were compared with corresponding values of $C''$. Data for various objects under different luminaires and pavement combinations are presented in Part A of figure 9 for the original and Part B for the *final* procedure. Double logarithmic plots are used. All of these data represent a fixed distance of 180 feet. Thus, there is no parameter along which to order values of $C''$ and, hence, figure 9 contains only a simple regression line. The solid line in each part of the figure has a 45-degree slope representing that $\tilde{C}$ is proportional to $C''$. Because the line does not pass through the (0, 0) origin, $\tilde{C}$ is proportional to a constant times $C''$. This is, of course, acceptable because there was no satisfactory way to relate the threshold data and the measurements made with a VTE. The data seems to cluster more closely about the regression line in Part B than in Part A, particularly the data for the manikin. This was interpreted to mean that the values of $\tilde{C}$ obtained under the *final* VTE procedure agree more closely with the computed indices of visibility than do corresponding data obtained under the original procedure.

A better procedure for evaluating the values of $\tilde{C}$ obtained at various distances in terms of corresponding values of $C''$ can be achieved by plotting the values of both $\tilde{C}$ and $C''$ as a function of distance. The data obtained for the dog from test series II and III are shown in figure 10 and for the manikin in figure 11. The data for the black disk from test series II are plotted in figure 12. There was no evidence that results from either VTE procedure agreed better with values of $C''$ in the tests of the dog and of the black disk. However, data obtained for the manikin under the *final* VTE procedure agreed with the predicted visibility indices better than data obtained under the original VTE procedure.

The data shown by solid lines in figures 10, 11, and 12 were of considerable intrinsic interest because they represented the expected variation in visibility as a function of distance. The variations in $C''$ with distance are explained in the following terms: For the dog and black disk, visibility decreased slowly as a function of distance because of decreased angular size. In addition, visibility increased somewhat whenever the object was nearer than a luminaire on the same side. At this location, the objects received little illumination and, therefore, were very dark and had comparatively high negative contrast. The test in which the manikin was used produced a large sinusoidal variation in visibility as a function of distance and a superimposed general decrease in visibility as a function of distance because of the decrease in size. The locations having peak visibility corresponded to locations in which the manikin was slightly beyond a luminaire on the same side. In this location the manikin had a high degree of illumination, was very bright, and had high positive contrast to the background.

## *Required Illumination for Roadway Visual Tasks*

On the basis of the preceding analysis, the values of $\tilde{C}$ obtained under the *final* VTE procedure seemed at least somewhat more valid than those obtained under the original procedure. Also, the two VTE procedures effected equivalent results for the shorter distances between observer and task. In

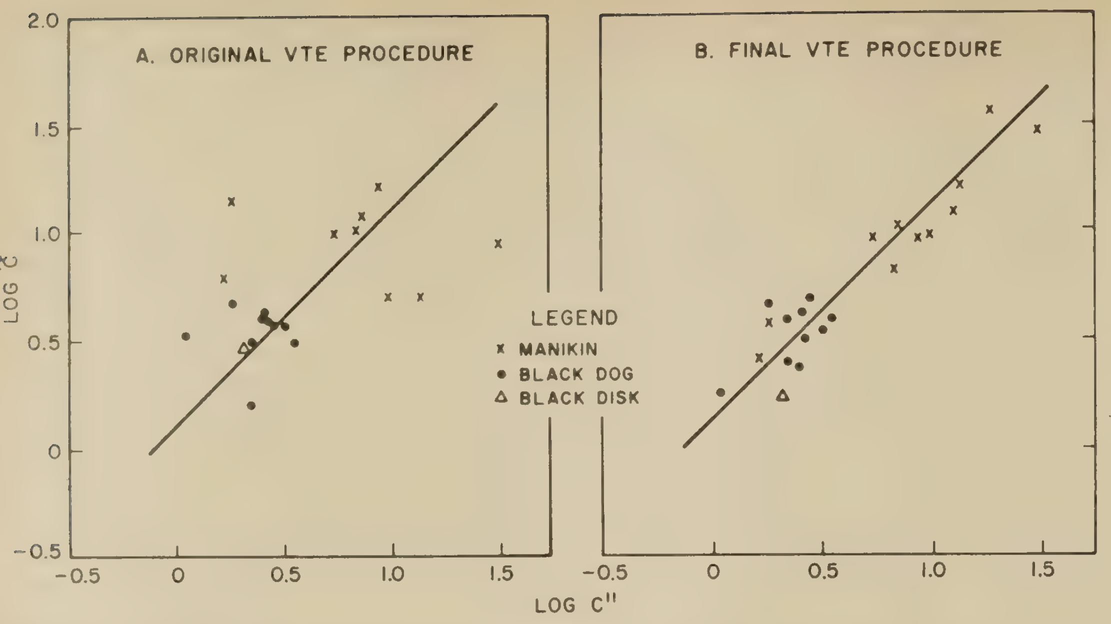


*Figure 9.—Variation log $\tilde{C}$ as a function of corrected, computed equivalent contrast, log $C''$.*

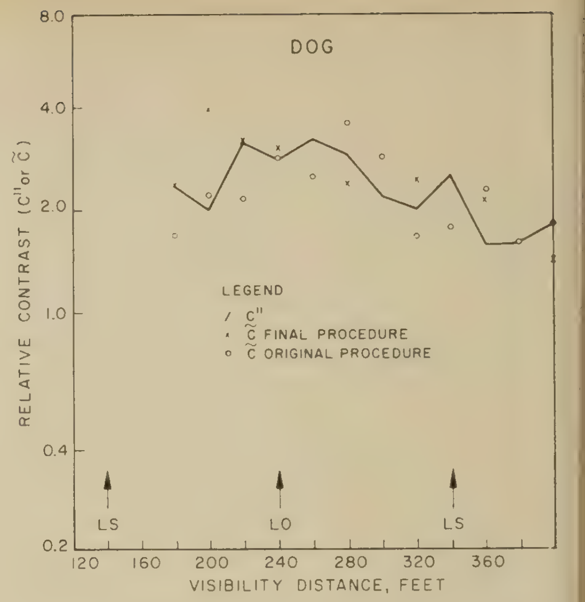


*Figure 10.—Relative $C''$ and $\tilde{C}$ as functions of visibility distance, for a dog.*

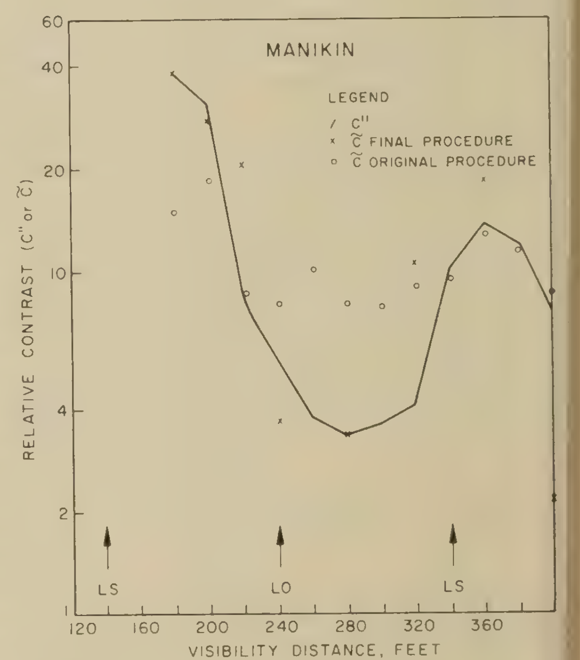


*Figure 11.—Relative $C''$ and $\tilde{C}$ as functions of visibility distance, for a manikin.*

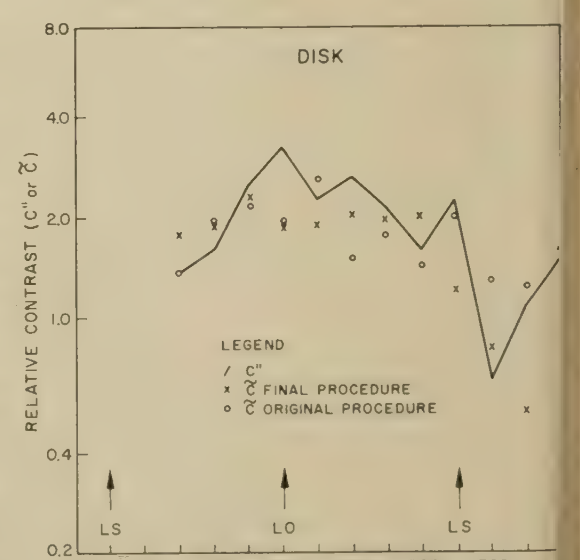


*Figure 12.—Relative $C''$ and $\tilde{C}$ as functions of visibility distance, for a disk.*

the test made with the manikin, the two VTE procedures produced approximately equivalent data for distances less than 220 feet. In the tests made with the dog and disk, the cutoff point was about 320 feet. By keeping these two findings in mind, it was then possible to sort through all the data obtained in the three series of measurements and attempt to determine what illumination ($E_r$ and $E_r'$) would have been required to bring the performance of these tasks to the assumed criterion level.

### Test series I and II

Because it was concluded that the experiments of test series I, in which the original VTE procedure was used, distorted the visibility indices, at least for the longer distances, it was decided to restrict the use of test series I data to distances of less than 220 feet. The comparison between the original and *final* VTE procedures indicated that these two yielded equivalent results under these conditions; therefore, data at two distances—180 and 200 feet—were used.

For all experiments of test series II, the *final* VTE procedure was used, so all distances were suitable in computing values of $E_r$ and $E_r'$. The different roadway conditions used during test series I were studied in test series II for a distance of 180 feet only. In addition, the dog, manikin, and black disk were studied at various distances for one illuminant-pavement combination.

Several analyses involving the data from test series I and II can be presented before discussing data from test series III because in test series III only the dog and manikin were studied under one illuminant-pavement combination. Therefore, test series I and II data contain the only information on other tasks, illuminants, and pavement. Values of $E_r$ and $E_r'$ for these tasks are summarized in table 1, and values for the dog and manikin obtained in the same tests are presented for comparison. The results show that different visual tasks occurring on the roadway require illumination that ranges from 0.3 to nearly 1,000 footcandles. The presence of some high values was not surprising because the more difficult roadway tasks seem at least as difficult as some of the tasks that were studied indoors and produced equally high values. From among the tasks studied, the two chosen for major emphasis—the dog and manikin—were analyzed as being a fair representation of the task of mean difficulty. All the tasks were chosen as being typical of collision obstacles.

The amount of disability glare for different roadway conditions was analyzed, and values of $K'$ are shown in table 2. Disability glare differed significantly with the type of illuminant, being least for incandescent, a little worse for mercury, and considerably worse for fluorescent illumination. Disability glare was considerably worse on asphalt than on concrete pavements. This difference was expected because the luminaires, relative to the visual environment, seemed to be brighter when seen against the pavement material having the lower reflectance. Disability glare was also worse in the tests on the manikin than on the dog; this could have been predicted because the line of sight was elevated more for viewing the manikin than the dog.

Data on the effect of luminaire spacing is presented in table 3. To see the dog, more footcandles were required when luminaires were spaced 200 feet rather than 100 feet apart, but markedly lower illumination was required to see the manikin where the luminaires were farther apart. The differences in the effect on the illumination required to see the dog were probably not significant, but the differences required to see the manikin were. These results are explained in these terms:

A luminaire was located 40 feet in front of the object in each test. The difference in luminaire spacing, therefore, caused a difference in the distance to the first luminaire *behind* the object. The manikin was seen as an object brighter than its background because the luminance contrast was larger when the uminaires were spaced farther apart. The manikin's luminance was unaffected by spacing but its background was darker when the luminaires were farther apart. The dog was seen as an object darker than its background because the wider spacing of luminaires reduced luminance contrast by reducing background luminance. This analysis of luminaire spacing has no generality beyond the situation tested and depends decisively upon the fact that in each test a luminaire was located 40 feet in front of the object. Had the VTE and object positions been altered, a very different result might have been obtained. This analysis demonstrated the danger of generalizing from data based on tests in only one location beneath the luminaires. Data on the task at several locations within a single cycle of the luminaires was very necessary, and the need for such data was part of the reason for conducting test series III.

Required illumination for the two major objects located in the driving and curb lanes were computed, and the results are given in table 4. In the curb lane, the object was located 5 feet to the left of the right pavement edge and was viewed from the same location in the driving lane. Had parking been allowed, this would have been the parking lane. In this test, however, no cars were parked and the lane could have been used to drive in. The values of illumination for curb and driving lanes refer particularly to the lighting needed in the respective lanes. Thus, in interpreting requirements for illumination in the curb lane, it was necessary to consider how much was produced by the lighting system in the curb lane and how much was needed.

Values of the illumination required for each object, considering disability glare, $E_r'$, were approximately 3 times higher for the curb lane than the driving lane. Values of the required illumination, not considering glare, $E_r$, were approximately 2.2 times higher in the curb lane than in the driving lane. Analysis of these data, therefore, showed that a portion of the difference in requirements for illumination under the two conditions was the result of a difference in disability glare But other factors must have been at work.

The values of $B_r$ and $B_r'$ were higher in the curb lane when the dog was the object than in the driving lane, thus indicating that the task was more difficult in the curb lane. No consistent differences in $B_r$ and $B_r'$ were produced by the data about the manikin. Therefore, the visual tasks studied were at least as difficult, if not more so, in the curb lane. Also, the luminaires were less effective in producing luminance in the curb lane than in the driving lane. Together, these three factors probably account for the apparent requirement for more illumination in the curb lane for the same performance level as in the driving lane. Because more illumination was required in the curb lane, the resultant lighting problem becomes doubly difficult as in most conventional lighting systems the curb lane will have less illumination than the driving lane.

**Table 1.—Illumination required to see objects 180 to 200 feet away in driving lane [1]**

| Object | Illumination required: No disability glare ($E_r$) | Illumination required: Disability glare ($E_r'$) | Disability glare constant ($K'$) |
|---|---|---|---|
| | *Footcandles* | *Footcandles* | |
| Auto | 0.312 | 0.341 | 1.39 |
| Manikin, light coat | .349 | .358 | 1.13 |
| Manikin, gray coat | .387 | .414 | 1.35 |
| Cone marker | .415 | .436 | 1.21 |
| Dog, light | 1.30 | 1.52 | 1.20 |
| Dog, black | 1.64 | 1.80 | 1.18 |
| Bicycle | 7.23 | 10.8 | 1.23 |
| Brick | 782 | >926 | 1.13 |

[1] Data shown are the mean values of results obtained from tests I and II, on asphalt pavement, 100-foot spacing of luminaires.

**Table 2.—Disability glare constant ($K'$) when observed objects are in driving lane [1]**

| Pavement and objects | Illuminant: Incandescent, 6,000 lumen | Illuminant: Incandescent, 15,000 lumen | Illuminant: Mercury | Illuminant: Fluorescent |
|---|---|---|---|---|
| Asphalt: | $K'$ | $K'$ | $K'$ | $K'$ |
| Dog | 1.18 | 1.18 | 1.12 | 1.78 |
| Manikin | 1.23 | 1.35 | 1.38 | 2.00 |
| Concrete: | | | | |
| Dog | 1.02 | 1.05 | 1.15 | 1.38 |
| Manikin | 1.12 | 1.12 | 1.35 | 1.48 |
| Mean [2] | 1.16 | | 1.25 | 1.66 |

[1] Data from test series I, 100-foot spacing of luminaires.
[2] Means for the disability glare constant for type of pavement and objects are: Pavement—asphalt, 1.40; concrete, 1.21; object—dog, 1.23; manikin, 1.38.

**Table 3.—Illumination required to see object when luminaires are at two different spacings [1]**

| Test series | Visibility distance | 100 feet between luminaires: No disability glare ($E_r$) | 100 feet between luminaires: Disability glare ($E_r'$) | 200 feet between luminaires: No disability glare ($E_r$) | 200 feet between luminaires: Disability glare ($E_r'$) |
|---|---|---|---|---|---|
| DOG | | | | | |
| | *Feet* | *Footcandles* | *Footcandles* | *Footcandles* | *Footcandles* |
| I | 180 | 0.649 | 0.664 | 1.30 | 1.42 |
| I | 200 | .649 | .664 | 1.37 | 1.50 |
| II | 180 | 1.27 | 1.33 | .991 | 1.06 |
| II | 200 | .986 | 1.03 | .589 | .646 |
| Mean | | 0.889 | 0.922 | 1.06 | 1.16 |
| MANIKIN | | | | | |
| I | 180 | 0.443 | 0.463 | 0.135 | 0.144 |
| I | 200 | .461 | .471 | .283 | .311 |
| II | 180 | 1.17 | 1.32 | .235 | .240 |
| II | 200 | .601 | .672 | .379 | .417 |
| Mean | | 0.669 | 0.732 | 0.258 | 0.278 |

[1] Source of light, 15,000-lumen incandescent illuminants on concrete pavement.

The illumination required by use of the different illuminants studied is given in table 5. Analysis of the $E_r$ values shows that the illuminants may differ in complex ways even without the disability glare being a factor. Fluorescent illuminants seemed to be superior to incandescent, and mercury illuminants were inferior to incandescent illuminants for objects such as the manikin. Because both mercury and fluorescent illuminants produce more disability glare than incandescent, a study of the values of $E_r'$ shows that for both the mercury and fluorescent illuminants more illumination was required than for the incandescent. The data were somewhat erratic and the differences should be applied with considerable caution, especially as only one type of fixture for each illuminant was compared in this study.

The results of an analysis of the illumination required on asphalt and concrete pavement surfaces are given in table 6. A study of the values of both $E_r$ and $E_r'$ shows that less

**Table 4.—Illumination required to see objects in curb lane and driving lane: Source of light, 15,000-lumen incandescent illuminants**

| Pavement type and test series | Visibility distance | Illumination required: No disability glare ($E_r$) — Driving lane | Illumination required: No disability glare ($E_r$) — Curb lane | Illumination required: Disability glare ($E_r'$) — Driving lane | Illumination required: Disability glare ($E_r'$) — Curb lane | Disability glare constant ($K'$) — Driving lane | Disability glare constant ($K'$) — Curb lane |
|---|---|---|---|---|---|---|---|
| **Dog** | | | | | | | |
| Asphalt: | *Feet* | *Footcandles* | *Footcandles* | *Footcandles* | *Footcandles* | | |
| I | 180 | 1.21 | 2.00 | 1.32 | 2.58 | | |
| I | 200 | 1.05 | 1.81 | 1.10 | 2.28 | 1.18 | 1.35 |
| II | 180 | 2.65 | 11.1 | 2.98 | 17.6 | | |
| Concrete: | | | | | | | |
| I | 180 | .649 | .721 | .664 | .810 | | |
| I | 200 | .649 | .810 | .664 | .930 | 1.05 | 1.13 |
| II | 180 | 1.27 | .939 | 1.33 | .985 | | |
| Mean | -------- | 1.25 | 2.89 | 1.34 | 4.20 | 1.12 | 1.24 |
| **Manikin** | | | | | | | |
| Asphalt: | | | | | | | |
| I | 180 | 0.360 | 1.32 | 0.395 | 1.75 | | |
| I | 200 | .349 | 2.83 | .374 | 4.29 | 1.35 | 1.59 |
| II | 180 | .451 | .455 | .472 | .498 | | |
| Concrete: | | | | | | | |
| II | 180 | 1.17 | .381 | 1.32 | .399 | 1.12 | 1.20 |
| Mean | -------- | 0.582 | 1.25 | 0.640 | 1.74 | 1.24 | 1.40 |

**Table 5.—Illumination required to see objects in driving lane under different illuminants**

| Pavement type and test series | Visibility distance | No disability glare ($E_r$): Incandescent 6,000 lumens | No disability glare ($E_r$): Incandescent 15,000 lumens | No disability glare ($E_r$): Mercury | No disability glare ($E_r$): Fluorescent | Disability glare ($E_r'$): Incandescent 6,000 lumens | Disability glare ($E_r'$): Incandescent 15,000 lumens | Disability glare ($E_r'$): Mercury | Disability glare ($E_r'$): Fluorescent |
|---|---|---|---|---|---|---|---|---|---|
| **Dog** | | | | | | | | | |
| Asphalt: | *Feet* | *Foot-candles* | *Foot-candles* | *Foot-candles* | *Foot-candles* | *Foot-candles* | *Foot-candles* | *Foot-candles* | *Foot-candles* |
| I | 180 | 1.84 | 1.21 | 1.53 | 0.876 | 2.16 | 1.32 | 1.83 | 1.40 |
| I | 200 | .984 | 1.05 | .763 | .739 | 1.10 | 1.10 | .875 | 1.02 |
| II | 180 | .982 | 2.65 | 2.30 | .754 | 1.10 | 2.98 | 2.96 | 1.17 |
| Concrete: | | | | | | | | | |
| I | 180 | .762 | .649 | .616 | .917 | .780 | .664 | .692 | 1.21 |
| I | 200 | .558 | .649 | .483 | 1.08 | .558 | .664 | .517 | 1.46 |
| II | 180 | .476 | 1.27 | .580 | .685 | .481 | 1.33 | .636 | .873 |
| Mean | ------ | 1.09 | | 1.05 | 0.842 | 1.18 | | 1.25 | 1.19 |
| **Manikin** | | | | | | | | | |
| Asphalt: | | | | | | | | | |
| I | 180 | 0.213 | 0.360 | 0.403 | 0.318 | 0.218 | 0.395 | 0.432 | 0.429 |
| I | 200 | .443 | .349 | 1.26 | .636 | .474 | .374 | 1.48 | .920 |
| II | 180 | .460 | .451 | .603 | .435 | .482 | .472 | .654 | .601 |
| Concrete: | | | | | | | | | |
| I | 180 | .311 | .443 | .708 | .295 | .318 | .463 | .852 | .339 |
| I | 200 | .268 | .461 | .444 | .562 | .274 | .471 | .498 | .677 |
| II | 180 | .540 | 1.17 | 1.18 | .423 | .598 | 1.32 | 1.31 | .509 |
| Mean | ------ | 0.456 | | 0.766 | 0.445 | 0.488 | | 0.871 | 0.579 |

**Table 6.—Mean values of illumination required to see objects on different types of pavement—based on data in table 5**

| Object | No disability glare ($E_r$): Asphalt | No disability glare ($E_r$): Concrete | Disability glare ($E_r'$): Asphalt | Disability glare ($E_r'$): Concrete |
|---|---|---|---|---|
| | *Footcandles* | *Footcandles* | *Footcandles* | *Footcandles* |
| Dog | 1.31 | 0.727 | 1.58 | 0.822 |
| Manikin | .494 | .567 | .577 | .636 |

illumination was required on concrete than on asphalt pavement when the dog was the object, but when the manikin was the object more illumination was required on the concrete pavement. This finding is explained by the relative reflectances of the objects and the pavement surfaces. The dog was dark and matched the asphalt considerably better than the concrete in reflectance. Therefore, the dog was more difficult to see on asphalt and considerably more illumination was required. Because the manikin was comparatively light and matched concrete somewhat better than asphalt, the manikin was somewhat more difficult to see on concrete and somewhat more illumination was required. This analysis explains clearly that the illumination required on the two different pavements depends intrinsically upon the object on the roadway, and that no general statement comparing the two types of pavement can be made accurately.

### Test series III

Values of $E_r$ and $E_r'$ for the measurements of test series III are given in tables 7 and 8. These values represent all the data from the third series of tests; the *final* VTE procedure was used exclusively. Illumination required is presented for each object and each distance; these data represent averages for the 11 different locations of the VTE as related to the luminaires. The values given, however, are restricted to the tests of incandescent luminaires and asphalt pavement. The same conclusion—that the manikin was somewhat less difficult to see on the asphalt pavement than the dog—can be drawn from the test series III data for distances of less than 300 feet. At longer distances, however, the manikin was no longer seen against the pavement in most tests. The small, white, frame house in the woods at the far end of the roadway may have been a critical factor.

Expressing data on roadway requirements for lighting in terms of pavement luminances rather than in illumination units, as has been done in the study reported here, is of considerable contemporary interest. Although the eye is concerned with luminances and not illumination requirements, the data herein are not presented in terms of luminances because:

First, although it is possible to design a lighting installation in terms of the illumination, it is difficult, if not impossible, to design it to provide specified luminances because of the lack of complete knowledge of the reflectance characteristics of pavement surfaces. Second, use of luminances could influence illuminating engineers so that they might forget that illumination has two functions in roadway lighting: (1) To produce pavement luminance; and (2) to produce object contrast. An analysis of the data compiled for this article suggests that contrast is more important than luminance.

The values of $E_r$ and $E_r'$ given in tables 7 and 8, the authors believe, are the best evaluations of the illumination needed to see the dog and manikin under incandescent luminaires and on asphalt pavement. These data can be used to provide a basis for establishing suitable illumination levels for roadway lighting. Consider first the test at a 200-foot distance—the data from test series I and II can be used with confidence for this distance or shorter distances. The data from test series III applied only to objects seen on the asphalt pavement under incandescent illumination. The data from test series I and II were used to define ratios relating illumination requirements for the other illuminant-pavement combinations to the incandescent illuminant-asphalt pavement condition. These ratios were then used to adjust the data from test series III to apply to other illuminants and/or pavements. The factors are summarized in table 9 for useful combinations of illuminant and pavement. The factors for the dog and manikin are maintained separately for $E_r$ and $E_r'$ values, respectively. For test series III, the average values of $E_r$ and $E_r'$ for the incandescent-asphalt combination were taken from tables 7 and 8.

Using the factors, estimated values of $E_r$ and $E_r'$ were computed for each combination of illuminant and reflectance and are given in table 9. The values given in this table represent the estimates of the illumination required for targets located in the driving lane, and each combination of illuminant and reflectance is given equal weight. The $E_r'$ values of 1.30 footcandles required to see the dog and 1.85 footcandles required to see the manikin represent the summary result of the entire study of roadway visual tasks. Of course, as was pointed out, all that can be suggested is to specify the illumination required for adequate visual performance for a *particular* illumination geometry and location of object and observer. Thus, the illumination units have no generality and cannot be used except in terms of similar conditions of illumination and viewing. Where geometry is different, as at other sites tested (*5*), the illumination required also was different, and average illumination could *not* be used as a reliable indicator of visibility.

An adequate understanding of the extent to which objects may be seen anywhere on the roadway when they appear without warning cannot be obtained only from the average values of illumination. To obtain some estimate of this aspect of the roadway lighting problem, values of $E_r$ and $E_r'$, for each of the 11 locations of the VTE used in test series III were computed for each combination of illuminant and pavement surface. The factors presented in table 9 were used to compute these data from the values of $E_r$ and $E_r'$ given for individual locations in tables 7 and 8. These calculations produced 66 values of $E_r$ and $E_r'$ for each target. They were used to generate the cumulative frequency graphs in figure 13. The ordinate in this figure represents the percentage of locations along the roadway in which the target in question was predicted to be adequately visible at 200 feet. Values of average illumination provided by the hypothetical lighting system of the same geometry are shown on the abscissa. Values of $E_r'$ are of primary intererst; however, the values of $E_r$ are given only to show how much

**Table 7.—Illumination required for observer at different locations to see objects at various distances, no disability glare, test series III**

| Observer location | Illumination required | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | Visibility distance, feet— | | | | | | | |
| | 180 | 200 | 220 | 240 | 280 | 320 | 360 | 400 |
| **Dog** | | | | | | | | |
| | *Footcandles* | *Footcandles* | *Footcandles* | *Footcandles* | *Footcandles* | *Footcandles* | *Footcandles* | *Footcandles* |
| 1 | 1.33 | 0.552 | 0.965 | 1.16 | 2.30 | 1.86 | 1.66 | 2.71 |
| 2 | 1.03 | 1.32 | 1.04 | 1.27 | 3.16 | 1.92 | 2.34 | 2.85 |
| 3 | 1.60 | 1.02 | 2.51 | 1.92 | 1.82 | 1.78 | 2.20 | 3.01 |
| 4 | 1.05 | 1.45 | 1.42 | 1.96 | 2.24 | 1.51 | 2.13 | 1.88 |
| 5 | 1.27 | .890 | 1.38 | 1.18 | 1.25 | 1.15 | 1.34 | 1.44 |
| 6 | .815 | .774 | 1.60 | .980 | 1.14 | 2.08 | 1.77 | 1.90 |
| 7 | 1.09 | 1.34 | 1.06 | .975 | 1.48 | 1.65 | 2.50 | 3.37 |
| 8 | .772 | 1.10 | 1.30 | 2.00 | 2.75 | 2.38 | 4.07 | 4.55 |
| 9 | 1.58 | 1.62 | 3.02 | 3.90 | 3.25 | 3.03 | 4.11 | 1.14 |
| 10 | 1.59 | 1.42 | 3.14 | 2.35 | 2.76 | 2.70 | 2.88 | 4.29 |
| 11 | 1.76 | 2.22 | 2.18 | 2.00 | 3.16 | 3.89 | 5.19 | 5.21 |
| Mean | 1.26 | 1.25 | 1.78 | 1.79 | 2.30 | 2.18 | 2.74 | 2.94 |
| **Manikin** | | | | | | | | |
| 1 | 0.312 | 0.600 | 0.791 | 1.23 | 1.56 | 1.51 | 0.773 | 3.08 |
| 2 | .834 | 1.05 | 1.23 | 1.41 | 1.20 | 1.42 | .956 | 3.58 |
| 3 | .842 | .840 | 1.23 | 3.12 | 2.60 | 5.66 | 2.32 | 2.27 |
| 4 | .806 | 1.06 | 2.32 | 1.28 | 1.08 | .724 | 2.68 | 2.58 |
| 5 | .696 | 1.61 | 1.12 | 1.51 | .495 | 1.58 | 3.20 | 12.2 |
| 6 | 1.06 | 1.40 | 1.83 | .914 | 1.12 | 3.00 | 3.00 | 2.66 |
| 7 | .825 | 1.22 | 1.26 | .449 | 1.45 | 2.89 | 14.8 | 24.8 |
| 8 | 1.96 | 1.05 | .775 | 1.17 | 2.56 | 8.03 | 7.08 | 3.88 |
| 9 | 1.02 | .721 | 1.20 | 1.17 | 2.30 | 8.90 | 34.6 | 2.44 |
| 10 | .845 | 1.57 | 2.16 | 2.20 | 2.20 | 24.6 | 10.2 | 34.6 |
| 11 | .510 | .893 | 2.26 | 2.10 | 2.11 | 1.92 | 1.33 | 2.20 |
| Mean | 0.882 | 1.09 | 1.47 | 1.50 | 1.70 | 5.48 | 7.35 | 8.57 |

less illumination could be used if disability glare could be entirely eliminated from roadway lighting.

It may be of interest to evaluate the extent to which these average required illuminations depend on the distance at which objects must be seen. The average values of $E_r$ and $E_r'$ from tables 7 and 8 may be expressed as a ratio of the average value for the 200-foot distance. Such ratios are plotted in figures 14 and 15. It is clear that the illumination required differs only a little between 180 and

**Table 8.—Illumination required for observer at different locations to see objects at various distances, disability glare ($K'$), test series III**

| Observer location | $K'$ | Illumination required | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Visibility distance, feet— | | | | | | | |
| | | 180 | 200 | 220 | 240 | 280 | 320 | 360 | 400 |
| **Dog** | | | | | | | | | |
| | | *Foot-candles* | *Foot-candles* | *Foot-candles* | *Foot-candles* | *Foot-candles* | *Foot-candles* | *Foot-candles* | *Foot-candles* |
| 1 | 1.20 | 1.56 | 0.591 | 1.11 | 1.40 | 2.84 | 2.24 | 1.99 | 3.56 |
| 2 | 1.29 | 1.30 | 1.75 | 1.34 | 1.60 | 4.27 | 2.42 | 3.02 | 3.76 |
| 3 | 1.41 | 2.21 | 1.32 | 3.79 | 2.90 | 2.51 | 2.40 | 3.56 | 5.11 |
| 4 | 1.66 | 1.42 | 2.30 | 2.25 | 3.41 | 4.17 | 2.24 | 3.78 | 3.50 |
| 5 | 1.59 | 1.97 | 1.23 | 2.09 | 1.82 | 1.77 | 1.73 | 2.06 | 2.18 |
| 6 | 1.62 | 1.15 | 1.09 | 2.80 | 1.26 | 1.68 | 3.70 | 3.00 | 3.22 |
| 7 | 1.62 | 1.54 | 2.03 | 1.37 | 2.32 | 2.46 | 2.74 | 4.76 | 4.15 |
| 8 | 1.32 | .930 | 1.26 | 1.56 | 2.63 | 3.88 | 3.36 | 6.60 | 4.98 |
| 9 | 1.32 | 2.03 | 2.04 | 4.79 | 7.61 | 5.15 | 4.80 | 4.39 | 2.93 |
| 10 | 1.29 | 1.96 | 1.75 | 4.59 | 3.40 | 4.09 | 3.91 | 3.09 | 4.60 |
| 11 | 1.32 | 2.22 | 2.99 | 3.09 | 2.82 | 4.68 | 4.79 | 6.25 | 7.89 |
| Mean | ------ | 1.66 | 1.67 | 2.62 | 2.83 | 3.41 | 3.12 | 3.86 | 4.17 |
| **Manikin** | | | | | | | | | |
| 1 | 1.20 | 0.312 | 0.600 | 0.800 | 1.44 | 1.88 | 1.54 | 0.781 | 3.96 |
| 2 | 1.29 | .841 | 1.26 | 1.52 | 1.78 | 1.52 | 1.45 | .969 | 4.95 |
| 3 | 1.41 | .853 | .840 | 1.26 | 4.73 | 3.84 | 5.66 | 3.52 | 3.35 |
| 4 | 1.66 | .845 | 1.12 | 4.64 | 1.51 | 1.19 | .758 | 6.13 | 5.60 |
| 5 | 1.59 | .730 | 2.44 | 1.57 | 2.39 | .506 | 1.80 | 6.38 | 52.3 |
| 6 | 1.62 | 1.57 | 2.22 | 3.18 | 1.62 | 1.20 | 3.28 | 4.15 | 4.21 |
| 7 | 1.62 | .883 | 1.77 | 1.38 | .449 | 2.24 | 5.38 | 64.4 | 86.0 |
| 8 | 1.32 | 2.58 | 1.08 | .791 | 1.22 | 3.62 | 13.3 | 13.5 | 5.25 |
| 9 | 1.32 | 1.04 | .729 | 1.25 | 1.22 | 3.11 | 13.4 | 104. | 3.08 |
| 10 | 1.29 | .885 | 1.73 | 2.92 | 2.83 | 2.83 | 57.4 | 16.2 | 45.6 |
| 11 | 1.32 | .510 | .904 | 3.20 | 2.77 | 2.16 | 2.01 | 1.36 | 2.30 |
| Mean | ------ | 1.000 | 1.33 | 2.05 | 1.94 | 2.19 | 9.64 | 20.2 | 19.7 |

Table 9.—Illumination required to see each of two objects 200 feet away on different types of pavement under different illuminants

| Pavement and illuminant | No disability glare ($E_r$) | | Disability glare ($E_r'$) | |
|---|---|---|---|---|
| | Multiplication factor [1] | Illumination required | Multiplication factor [1] | Illumination required |
| **Dog** | | | | |
| Asphalt: | | *Footcandles* | | *Footcandles* |
| Incandescent | 1.00 | 1.25 | 1.00 | 1.67 |
| Mercury | .964 | 1.20 | 1.06 | 1.77 |
| Fluorescent | .772 | .965 | 1.01 | 1.69 |
| Concrete: | | | | |
| Incandescent | .556 | .696 | .519 | .867 |
| Mercury | .536 | .670 | .548 | .915 |
| Fluorescent | .429 | .535 | .523 | .873 |
| Mean | NA | 0.886 | NA | 1.30 |
| **Manikin** | | | | |
| Asphalt: | | | | |
| Incandescent | 1.00 | 1.09 | 1.00 | 1.33 |
| Mercury | 1.68 | 1.83 | 1.78 | 2.37 |
| Fluorescent | .977 | 1.07 | 1.18 | 1.57 |
| Concrete: | | | | |
| Incandescent | 1.15 | 1.25 | 1.10 | 1.46 |
| Mercury | 1.93 | 2.11 | 1.96 | 2.61 |
| Fluorescent | 1.12 | 1.22 | 1.30 | 1.73 |
| Mean | NA | 1.43 | NA | 1.85 |

[1] Ratio of illumination required for condition to that required for an incandescent source on asphalt pavement, determine from test series I and II.

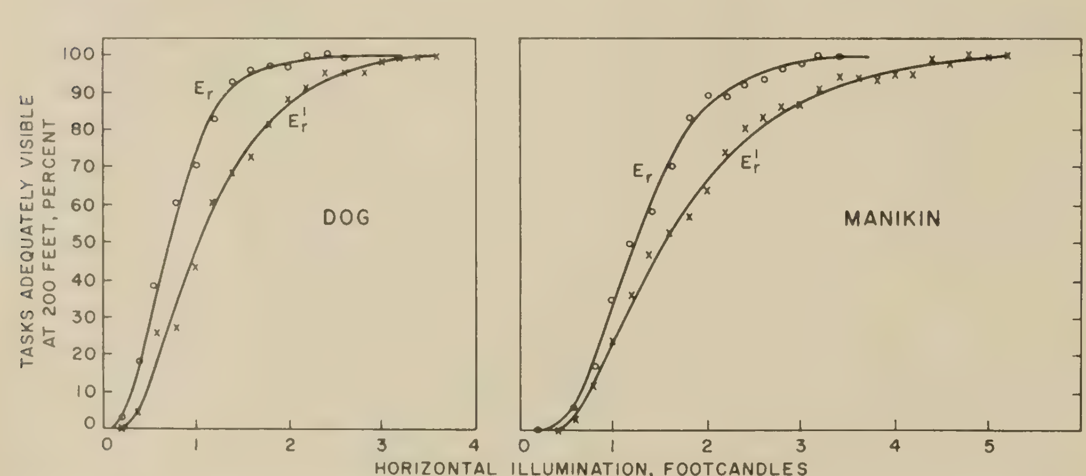


*Figure 13.—Percentage of tasks adequately visible at 200 feet for different levels of horizontal illumination.*

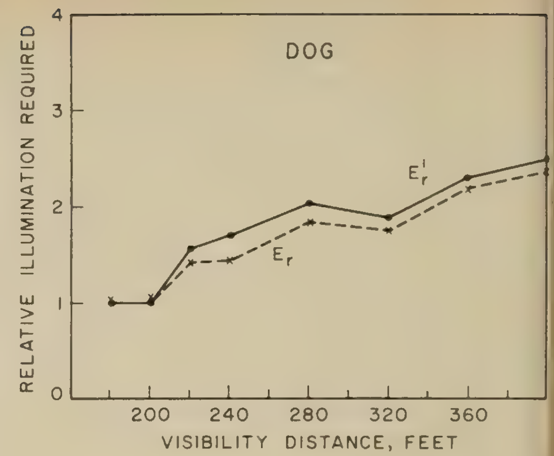


*Figure 14.—Relative illumination levels re quired to see a dog at distances other thar 200 feet.*

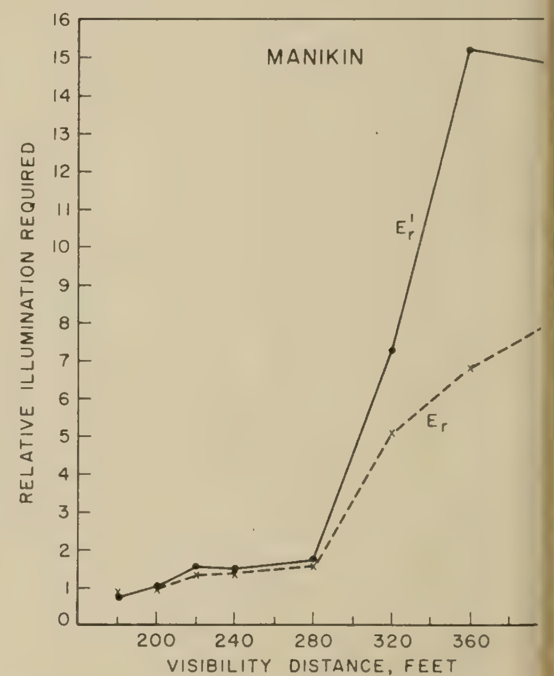


*Figure 15.—Relative illumination levels re quired to see a manikin at distances othe than 200 feet.*

200 feet, but that considerably more illumination is required at distances of 300 to 400 feet than at 200 feet. To see the manikin, the increase in illumination was considerably more than the increase needed to see the dog.

In evaluating the illumination requirement data from the Hendersonville test site, generally lower illumination values were necessary to meet the same performance criterion than illumination requirements for actual highway sites in Ohio (*5*). Thus, the final required illumination values for adequate visibility reported herein are probably conservative.

## REFERENCES


(*1*) *Development and Use of a Quantitative Method for Specification of Interior Illumination Levels on the Basis of Performance Data*, by H. Richard Blackwell, Illuminating Engineering, vol. LIV, No. 6, June 1959, pp. 317–353.

(*2*) *Use of Performance Data to Specify Quantity and Quality of Interior Illumination*, by H. Richard Blackwell, Illuminating Engineering, vol. L, No. 6, June 1955, pp. 286–299.

(*3*) *Illumination Requirements for Roadway Visual Tasks*, by H. Richard Blackwell, B. S. Pritchard, and Richard N. Schwab, Highway Research Board Bulletin 255, *Night Visibility: 1960*, pp. 117–127.

(*4*) *Further Validation Studies of Visual Task Evaluation and Other Elements of an Earlier Illumination Specification System*, by H. Richard Blackwell, Illuminating Engineering, vol. LIX, No. 9, September 1964, pp. 627–641.

(*5*) *Visibility and Illumination Variables in Roadway Visual Tasks*, by H. Richard Blackwell, R. N. Schwab, and B. S. Pritchard, Illuminating Engineering, vol. LIX, No. 5, May 1964, pp. 277–308.

(*6*) *Development of Procedures and Instruments for Visual Tasks Evaluation*, by H. Richard Blackwell, B. S. Pritchard, an Richard N. Schwab, to be published in Illum nating Engineering in the spring of 1966.

(*7*) *Measuring Disability Glare with Portable Meter*, by Glenn A. Fry, Proceedin of the 2d Research Symposium, Illuminati Engineering Research Institute, Dearbor Mich., Mar. 3–4, 1958 (processed).

(*8*) *Design and Calibration of a Disabili Glare Lens*, by Glenn A. Fry, Benjamin Pritchard, and H. Richard Blackwell, Illum nating Engineering, vol. LVIII, No. March 1963, pp. 120–123.

(*9*) *The Visual Effect of Non-Unifo Surrounds*, by Parry Moon and Domi Eberle Spencer, Journal of the Optical Socie of America, vol. 35, No. 3, March 1945, 233–248.

(*10*) *Contrast Thresholds of the Human E* by H. Richard Blackwell, Journal of th Optical Society of America, vol. 36, 1946, p 624–643.




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esign Charts for Open-Channel Flow (1961). 70 cents.

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inancing of Highways by Counties and Local Rural Governments: 1942–51 (1955). 75 cents.

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Highway Statistics, Summary to 1955. $1.00.

Highway Transportation Criteria in Zoning Law *and* Police Power and Planning Controls for Arterial Streets (1960). 35 cents.

Highways and Social and Economic Changes (1964). $1.25.

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Increasing the Traffic-Carrying Capability of Urban Arterial Streets: The Wisconsin Avenue Study (1962). 40 cents. Appendix, 70 cents.

Interstate System Route Log and Finder List (1963). 10 cents.

Labor Compliance Manual for Direct Federal and Federal-Aid Construction, 2d ed. (1965). $1.75.

Landslide Investigations (1961). 30 cents.

Manual for Highway Severance Damage Studies (1961). $1.00.

Manual on Uniform Traffic Control Devices for Streets and Highways (1961). $2.00.
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Reinforced Concrete Pipe Culverts—Criteria for Structural Design and Installation (1963). 30 cents.

Road-User and Property Taxes on Selected Motor Vehicles, 1964. 45 cents.

Selected Bibliography on Highway Finance (1951). 60 cents.

Specifications for Aerial Surveys and Mapping by Photogrammetric Methods for Highways (1958): a reference guide outline. 75 cents.

Standard Specifications for Construction of Roads and Bridges on Federal Highway Projects, FP–61 (1962–1964). $2.25.

Standard Plans for Highway Bridges (1962):
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The Identification of Rock Types (revised edition, 1960). 20 cents.

The Role of Economic Studies in Urban Transportation Planning (1965). 45 cents.

Traffic Assignment and Distribution for Small Urban Areas (1965). $1.00.

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