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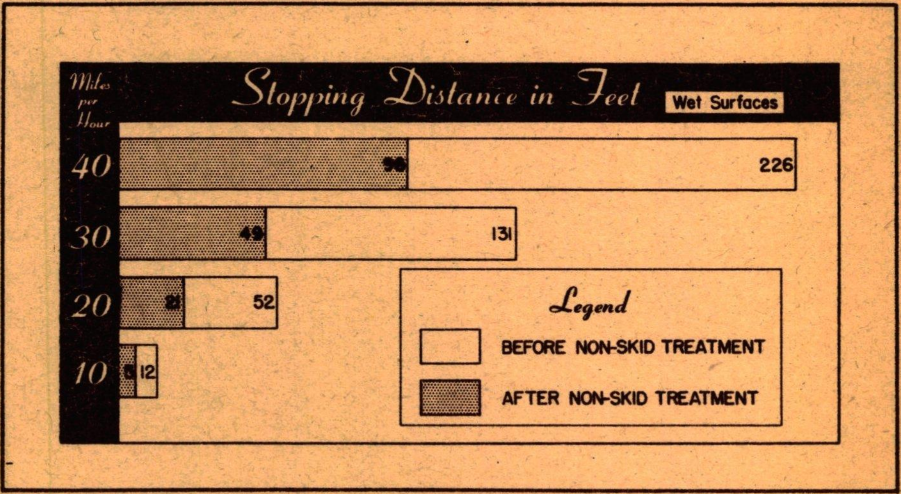

Skid Resistance Measurements

OF VIRGINIA PAVEMENTS

1948

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RESEARCH REPORT NO. 5-B

SKID RESISTANCE MEASUREMENTS OF VIRGINIA PAVEMENTS

BY

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AND

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Virginia Department of Highways

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SKID RESISTANCE MEASURMENTS OF VIRGINIA PAVEMENTS

TILTON E. SHELBURNE *Director of Research*
and
R. L. SHEPPE, *Associate Research Engineer*
Virginia Department of Highways

SYNOPSIS

More than 1000 measurements of forward skidding distances on 32 pavement surfaces both in a dry and in a wet condition are reported. The purpose of the study was to obtain data on surface characteristics that could be used as a guide for establishing future design, construction, and maintenance policies. Both good and worn tread synthetic and natural rubber tires were employed.

Skid resistance measurements were made by the stopping distance method. A light-weight automobile equipped with 6.00 by 16 in. 4-ply tires was used throughout the study. The test car was a free-moving vehicle and not influenced by the same forces as a towed vehicle. On level sections of pavement the car was brought to an initial uniform speed of 10, 20, 30, and 40 mi. per hr. and the brakes applied instantly, locking all four wheels. At the instant of locking the wheels a detonator, attached to the running board and actuated electrically through the brake pedal, fired a chalk bullet on the pavement. The forward skidding, or stopping distance, was measured from the chalk mark to the gun after the car had stopped. Two or more measurements were made at each of the four speeds on a dry surface. The pavement was then sprinkled with water and the tests repeated. Results reported are average values for each condition.

Tests were conducted principally on high-type pavements on primary roads including bituminous concrete, sand asphalt, special plant-mix, rock asphalt, sand and slag seal treatments, broom drag treatment, portland cement concrete, and glazed bituminous surfaces. Test sections were selected to permit an evaluation of different kinds of materials and methods of finishing. Tests were conducted on two-, three-, and four-lane pavements with traffic counts (24-hr. basis) ranging from 556 to 9401 vehicles. All but two tests, however, were conducted on the outside lanes. Fourteen of the 32 surfaces were resurfaced concrete pavements.

Forward skidding distances increased with speed and were much longer on wet than on dry surfaces. At 40 mi. per hr., measurements ranged from 63.6 to 88.9 ft. on dry and 72.0 to 254.5 ft. on wet surfaces. The data indicated that at this maximum speed (40 mi. per hr.) forward stopping distances are critical only on the wet pavements. It was considered unsafe to conduct tests on a wet surface at greater speeds.

Twenty-seven of the 32 surfaces were found to have satisfactory non-skid characteristics in a wet condition. Those surfaces having a harsh, gritty, sandpaper-like texture were found to have short stopping distances and correspondingly high coefficients of friction. On the other hand, long stopping distances were measured on smooth glazed surfaces. From a safety standpoint broom-finished concrete pavement was far superior to the smooth or belt-finished surface.

Non-skid treatments were effective in reducing forward skidding distances on wet surfaces. In one instance at 40 mi. per hr. the stopping distance was reduced from 224 to 94.7 ft. At 40 mi. per hr. on wet surfaces stopping distances were about 40 percent longer for worn than for good tread tires. Under comparable conditions stopping distances were about 12 percent longer with smooth synthetic (S-3) than with smooth or worn tread pre-war natural rubber tires.

The stopping distance method was found to be a relatively simple, inexpensive, quick means of measuring skid resistance of pavement surfaces. Close check results were obtained.

Since the President's Safety Conference in May 1946, more and more emphasis is being placed upon highway safety. Many factors are involved in the building of safety into highways. It is not the purpose of this paper to discuss these various items, but merely to report studies made on one factor only - the resistance of pavement surfaces to skidding.

Available data indicate that in rural Virginia 7587 accidents were reported in 1940, and 15,503 in 1946. Accurate data are lacking on the number of these accidents that can be directly attributed to the surface characteristics of the pavements; however, in many cases, the surface was reported to be slippery or hazardous. In some instances certain sections of pavements have been called to the attention of the Department as being hazardous when wet; however, by visual inspection many of these surfaces appeared to have good surface characteristics. The decided increase in the number of accidents and the desire for factual data on skidding characteristics of pavement surfaces prompted the Department to undertake this investigation.

Many studies of skid resistance are reported in technical literature. Perhaps the most complete data on factors affecting skidding are reported in Bulletin 120 of Iowa State College. *(1)*[1] In this report by Prof. Moyer all of the important phases of skidding related to highway safety were analyzed. Coefficients of friction for new tread and smooth tread tires on wet and dry surfaces were measured for both straight ahead skidding and sideways skidding at speeds of from 3 to 40 mi. per hr. by means of a two-wheel trailer test unit. Tests were made upon all types of bituminous, portland cement concrete, brick, gravel, cinders, asphalt plank, steel plates, and wood plank surfaces. Tests were also run upon mud-, snow-, and ice-covered surfaces with and without tire chains. In addition, Prof. Moyer reported results of the effect of tire pressure, wheel loads, tire tread, and temperature. As a result of these tests it was demonstrated that to be reasonably free from the dangers of skidding when braking on wet surfaces, straight skid coefficients of 0.40 or higher at 40 mi. per hr. should be available. This 0.40 coefficient of friction corresponds to a stopping distance of 133 ft.

In 1939, the Oregon State Highway Department reported results of tests conducted to determine the skidding resistance of pavements in that state *(2)*. These tests were conducted in a manner similar to those reported by Prof. Moyer. In general, the results of the Oregon tests confirmed those of Prof. Moyer and supplied information for design purposes in that State.

The purpose of the tests reported herein was to secure factual data

[1] Italicized figures in parentheses refer to list of references at the end of the paper.

on Virginia pavements. It was not intended to study the many factors involved but to hold constant as many of these variables as possible and to obtain data on the surface characteristics that could be used as a guide for establishing future design, construction, and maintenance policies. During the past year more than 1000 tests have been performed on 32 pavement surfaces. While the main perspective was to evaluate pavement surface characteristics, some tests were duplicated with four different sets of tires.

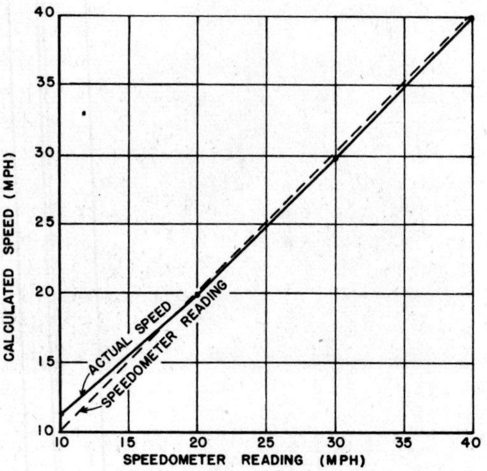


Figure 1. Speedometer Calibration

Although two types of skidding are common - forward skidding and side skidding - these measurements deal with forward skidding only. In conducting the tests it appeared that in general, forward skidding is a function of the tire and pavement texture while side skidding may be influenced by the following: car balance, brake adjustment, or pavement surface irregularities such as excess crown, transverse dips, or differences in elevation of two adjoining longitudinal surfaces. In these tests the car was a free-moving vehicle and not influenced by the same forces as a towed vehicle.

Another factor deserving consideration in evaluating skid resistance is the variation between the skidding distance on a dry surface and a wet one. For a surface to be safe for all-weather conditions its skid resistance when wet should be reasonably close to that when dry. Many skidding accidents may occur when the driver suddenly encounters a wet surface after he has been driving on a dry one. Often he is not aware that the resistance of the surface has changed appreciably.

DESCRIPTION OF TESTS

The skid resistance measurements were made by the stopping distance method. This procedure has previously been described by Prof. Moyer *(3)*. In this study Prof. Moyer found that results obtained in stopping distance tests were remarkably uniform and that the possibilities of error by this method were very small. He further found that the rating of surface slipperiness was almost identical for both the towing-braking tests and the stopping distance tests.

Figure 2. Test Car

A 1946 Stylemaster Chevrolet car weighing 3090 lb. and equipped with 6.00 by 16 in. 4-ply tires was used throughout the tests. Prior to performing the tests the speedometer was calibrated by means of a stop-watch over a measured mile. Several trial runs were made at each speedometer reading of 10, 20, 30, and 40 mi. per hr. and the time checked carefully. Results of the calibration are shown grafically in Figure 1.

All skid measurements were performed on tangents with relatively even surfaces and on level sections of pavement. Before each series of tests the air pressure in each tire was adjusted to 30 psi. as measured by a bourdon type gauge. In performing the tests the car was brought to an initial uniform speed of 10, 20, 30, and 40 mi. per hr. and the brakes applied hard instantly, or as nearly so as possible, to lock all four wheels. At the

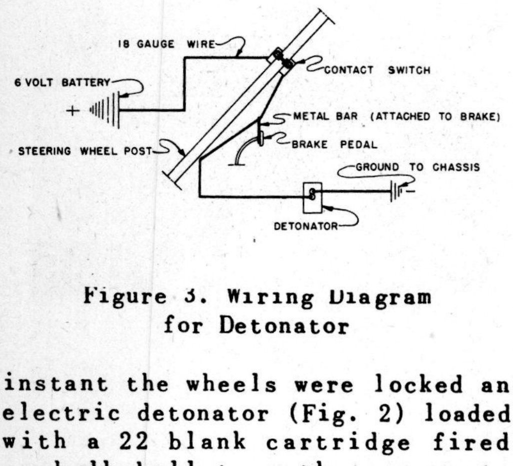


Figure 3. Wiring Diagram for Detonator

instant the wheels were locked an electric detonator (Fig. 2) loaded with a 22 blank cartridge fired a chalk bullet on the pavement. The gun was attached to the running board of the car and actuated electrically through the brake pedal. The wiring diagram for the electric detonator is shown in Figure 3. The stopping distance, that is, the distance from the chalk mark to the gun, was measured by means of a steel tape (Fig. 4). Two or more measurements were made at each speed and results reported are average values.

Four sets of tires were used in the tests. For convenience, the four sets of tires are designated by the letters "A", "B", "C", and "D". (See Fig. 5).

Tires A - These tires were furnished with the car when it was purchased by the Department. From correspondence with the manufacturer it was

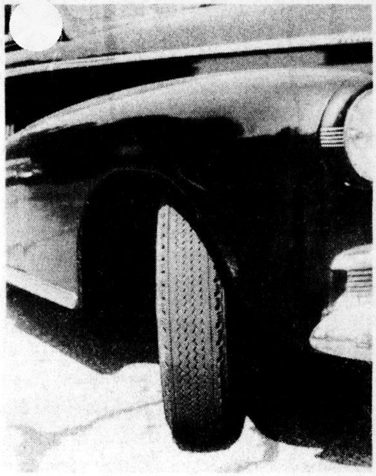
Figure 4. Measuring Skidding Distance

learned that they are composed of 77 percent synthetic and 23 percent natural rubber. The tires were new when the tests were started.

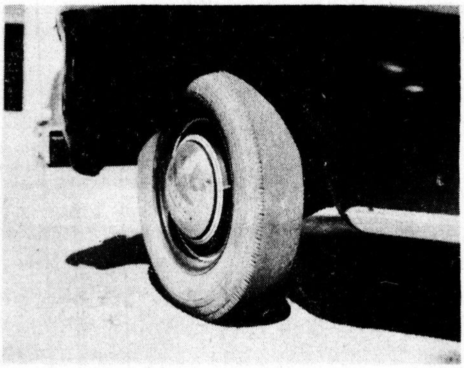
Tires B - Good tread natural rubber tires which were supplied by one of the prominent rubber companies. They were built for purely experimental purposes and had been used by the National Safety Council for its tests on ice at Houghton Lake, Michigan during January, 1946. The tires had been used for about 1500 miles on roads prior to the tests. They represent a typical condition of tires on a new car during the first three or four months of service.

Tires C - Smooth tread synthetic tires which were furnished by the Equipment Depot of the Department. They were worn tires and marked S-3. No record is available as to their mileage; however, the tread was quite smooth. They represent a poor tread condition and would undoubt-

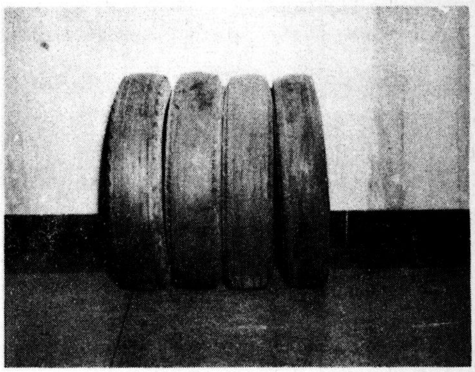
"A"



"B"



"C"



"D"

Figure 5. Tires Used for Skid Resistance Tests

edly be discarded as unsafe by the average driver.

Tires D - Worn tread natural rubber tires which were secured from members of the Department. In many cases these tires, although seven years old, were still used as spares or were stored in garages. They were pre-war tires with worn tread. In most cases these tires would be considered to have reached their maximum life.

In performing the tests, measurements were made at the four speeds with the surface in a dry condition. The pavement was then sprinkled with water and the tests repeated. In most cases in hot weather, especially on coarse textured surfaces or on pavements with high crown, it was necessary to sprinkle the pavement more than once to maintain a uniformly wet condition. Most of the tests were performed with the good tread type A tires. Exceptions were surfaces 30, 30-A, and 31. The remainder of the tests were performed in an attempt to secure some information on the tire variable.

As a matter of record both air and pavement temperatures were taken for each test. Since the tests were performed intermittently during a 9-mo. period, air temperatures ranged from 36 to 92 F. and averaged 73 F. Corresponding pavement temperatures varied from 33 to 102 F. and averaged 70 F.

An alert, skillful driver in good physical condition performed the tests. Insofar as possible the same driver was used throughout the series of tests. Since these tests were rather severe on the car it was necessary to maintain it in excellent operating condition. This required frequent balancing of wheels and adjustment of brakes. At the greater speeds and particularly on wet surfaces there was some side skidding as the car came to rest. By careful selection of an even test section and by skillful driving this was held to a minimum. In cases where side skidding was considered excessive forward skidding measurements were discarded and the test repeated.

Since the tests were conducted on actual highways it was necessary to take precautions for the safe handling of traffic. Flagmen were stationed at either end of the test sections approximately 1500 ft. apart. Troopers from the State Police force directed traffic during the conduct of the tests.

DESCRIPTION OF SURFACES TESTED

When the working plan for this study was formulated, it was decided to evaluate the different types of surface according to their performance with one tire condition (tires A). Tires B, C, and D were to be used only for sufficient tests to enable a comparison of the tire variable. This procedure was followed except for tests at locations Number 30 and 31 which were included after tires A were discarded. Due to the very thin layer of rubber on tires C and D, it was deemed advisable to run tests with these tires on the wet pavements only.

The surfaces tested were selected with the view of determining skid resistance qualities of the high-type pavements and the effect of different kinds of material or methods of finishing. Some tests were run on low-type surfaces because of some condition peculiar to that surface or because its location was convenient to other test sites. It is realized that there is considerable variation in surfaces of the same type but it is believed that the pavements selected are representative. There are also many other types of pavement in Virginia; however, these are principally on the secondary roads where speeds are not so great as on primary highways. Many of the pavements under consideration are on U.S. Route 1. Test locations are shown in Figure 6.

The pavements tested were two-, three-, and four-lanes in width. Surfaces 5 and 11 were on the inside lanes and all others tested with tires A were on the outside lanes. Due to high crown in the pavements, surfaces 30, 30-A, and 31 were tested in the center of the road. All tests on each surface were made in the same direction, which was selected so as to take advantage of alignment and grade.

On the three- and four-lane roads there was often some difference in texture between the inside and outside lanes caused by the greater amount of traffic in the outside lanes. Also, oil drippings from traffic were more prevalent on the outside lanes. Since this is

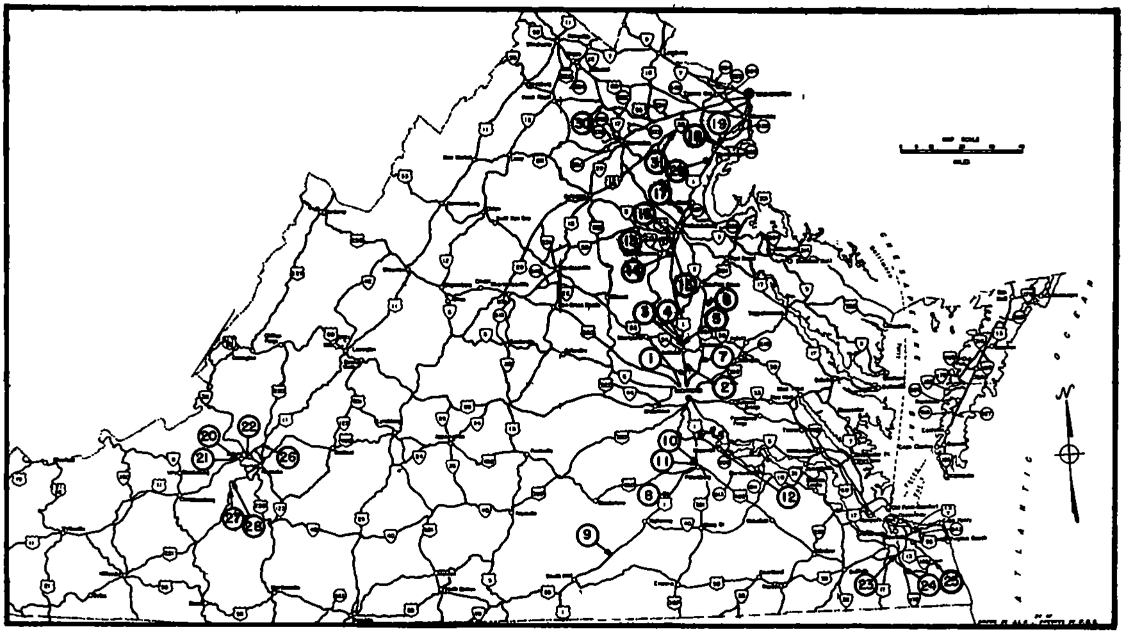


Figure 6. Location of Pavement Surfaces Tested for Skid Resistance

KEY

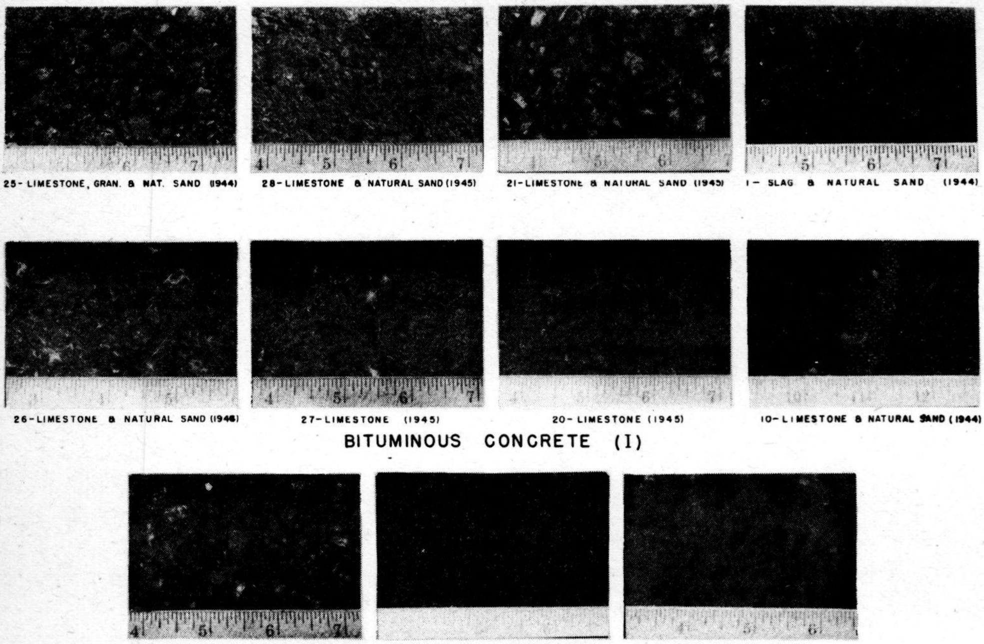
Test No.	Description	Test No.	Description
1	*H-3 (Slag & Natural Sand-1944)	16	Special Plant Mix (1940)
2	F-1 (25% Granite Scr.-75% Nat.Sand-1945)	17	F-1 (100% Natural Sand-1945)
3	F-1 (100% Granite Screenings-1945)	18	F-1 (100% Granite Screenings-1945)
4	P.Cement Concrete (Belt Finish-1938)	19	F-1 (100% Trap Rock Screenings-1945)
5	Natural Sand Seal-1946	20	H-3 (100% Limestone Aggregate-1945)
6	F-1 (100% Natural Sand-1943)	21	H-3 (Limestone & Natural Sand-1945)
7	Road Mix (No.11 Gravel-1946)	22	F-1 (100% Limestone Screenings-1944)
8	Slag Seal on Sheet Asphalt-1938	23	P.Cement Concrete (Belt Finish-1943)
9	Natural Rock Asphalt (1941)	24	F-1 (100% Granite Screenings-1944)
10	H-3 (Limestone & Natural Sand-1944)	25	H-3 (Granite & Natural Sand-1944)
11	Slag Seal on H-3 Plant Mix (1946)	26	*I-3 (Limestone & Natural Sand-1946)
12	Special Plant Mix (1946)	27	H-3 (100% Limestone Aggregate-1945)
13	P.Cement Concrete (Broom Finish-1946)	28	H-3 (Limestone & Natural Sand-1945)
14	F-1 (100% Granite Screenings-1945)	29	F-1 (100% Trap Rock Screenings-1946)
15	F-1 (100% Natural Sand-1946)	30	Bituminous Seal Treatment
		31	Mixed in Place Surface Treatment

*Surfaces such as H-3, F-1 & I-3 are plant-mix bituminous surfaces

the condition to which most traffic is subjected it was felt that tests where possible should be run on the outside lanes.

On the basis of average 24-hr. counts, traffic on the roads tested varied from 556 vehicles on surfaces No. 27 and 28 (2 lanes) to 9401 on surfaces No. 10 and 1 (4 lanes). For the year ending June 30, 1946, an average of the traffic counts at the 31 locations was 3693 vehicles.

Since many of these surfaces are of similar types they will be described below in eight groups. This will not only prevent repetition but will also enable the reader to get a clearer understanding of the various surfaces. It should be pointed out that 14 of the 32 surfaces are resurfaced concrete pavements (Nos. 1, 2, 3, 5, 8, 9, 10, 11, 14, 15, 17, 18, 19, and 29). In conjunction with the tests,



25-LIMESTONE, GRAN. & NAT. SAND (1944) 28-LIMESTONE & NATURAL SAND (1945) 21-LIMESTONE & NATURAL SAND (1945) 1-SLAG & NATURAL SAND (1944)

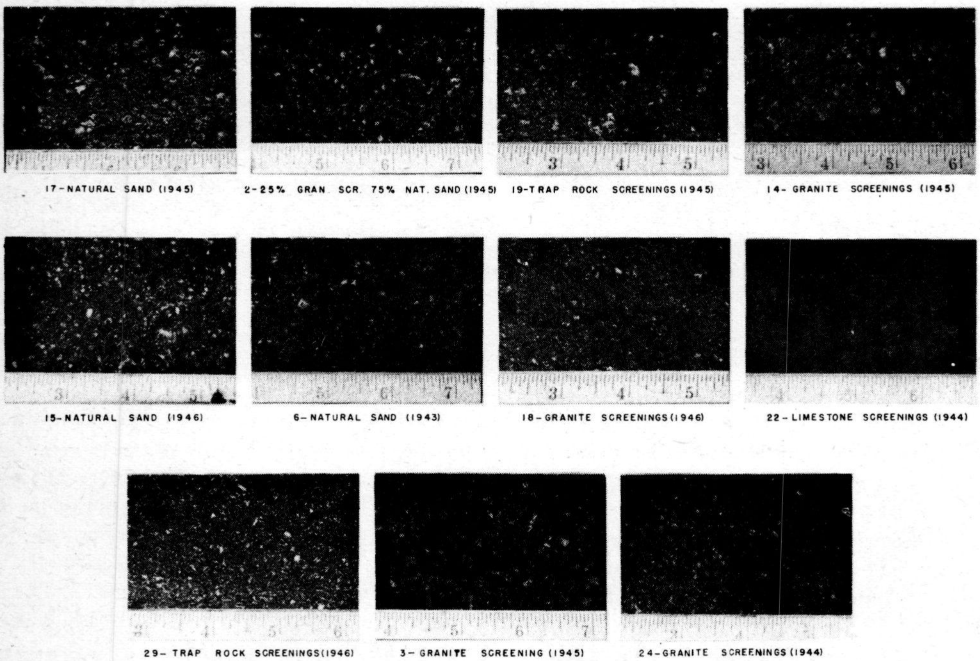
26-LIMESTONE & NATURAL SAND (1946) 27-LIMESTONE (1945) 20-LIMESTONE (1945) 10-LIMESTONE & NATURAL SAND (1944)

BITUMINOUS CONCRETE (I)

16-LOCAL SAND & GRAVEL (1940) 12-LOCAL SAND & GRAVEL (1946) 8-ROCK ASPHALT (1941)

SPECIAL PLANT MIX (III) (IV)

Figure 7. Texture of Surfaces



17-NATURAL SAND (1945) 2-25% GRAN. SCR. 75% NAT. SAND (1945) 19-TRAP ROCK SCREENINGS (1945) 14-GRANITE SCREENINGS (1945)

15-NATURAL SAND (1946) 6-NATURAL SAND (1943) 18-GRANITE SCREENINGS (1946) 22-LIMESTONE SCREENINGS (1944)

29-TRAP ROCK SCREENINGS (1946) 3-GRANITE SCREENING (1945) 24-GRANITE SCREENINGS (1944)

SAND ASPHALT (II)

Figure 8. Texture of Surfaces

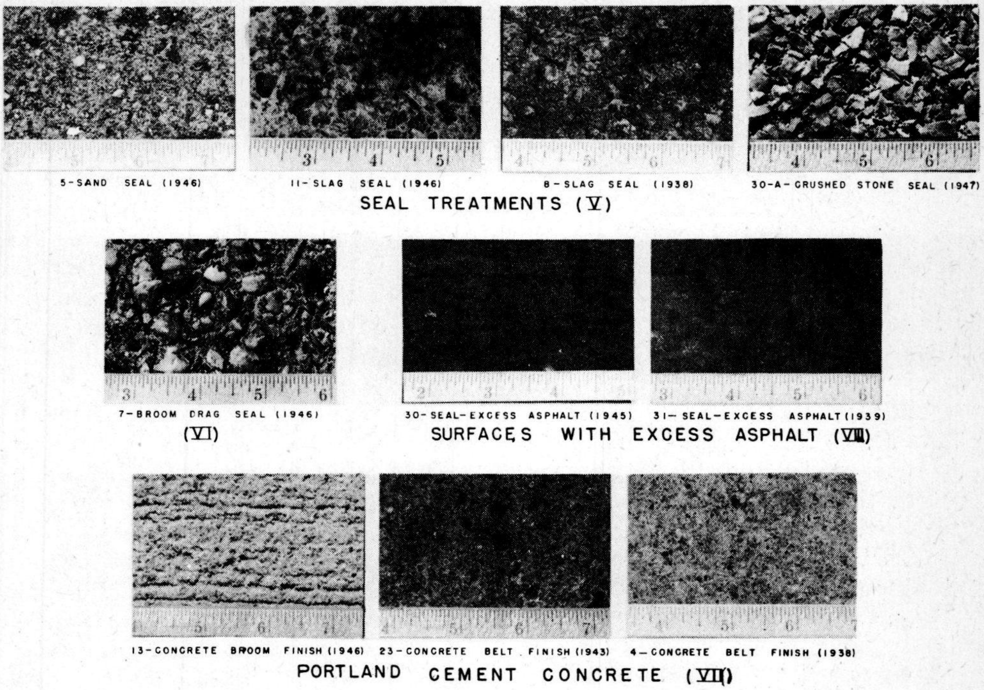


Figure 9. Texture of Surfaces

photographs were taken of pavement textures to record actual conditions at the time of tests. The texture pictures are arranged by groups and identified by test number in Figures 7, 8, and 9.

Group I - Bituminous Concrete (Type H-3 and I-3 Plant Mix) Bituminous concrete pavement (type H-3 and I-3) consists of a dense-graded bituminous concrete surface. These wearing courses are usually about one in. thick and placed on a binder or leveling course. This is a hot plant-mixed material composed of 85-100 penetration asphaltic cement (5.5 to 8.5 percent) combined with grade A crushed stone, or slag coarse aggregate and sand, slag, or stone screenings, or a combination thereof. The aggregate is uniformly graded from coarse to fine with 100 percent passing the ½-in. sieve and 2 percent to 10 percent passing the No. 200 mesh sieve. Specifications for this type surface are also varied within the above limits in order to obtain the most desirable mix with the materials available. Prior to 1946 this type was designated as H-3; since that time it is known as I-3. All surfaces listed as H-3 in this report were constructed before this change in designation.

Eight surfaces of this type from one to three years in age were tested. Surface No. 1 was composed of slag and sand aggregate; Nos. 10, 21, 26, and 28 of limestone and sand; Nos. 20 and 27 of 100 percent limestone; and No. 25 of limestone, granite, and sand aggregate. Limestone is the most commonly used coarse aggregate for this type of surface.

Group II - Sand Asphalt (Type F-1 Plant Mix) Sand asphalt pavement (type F-1) consists of one or more courses of sand asphalt constructed on a prepared base. This is a hot plant-mixed material composed of 85-100 penetration asphaltic cement in the amount of 5.5 to 9.5 percent combined with natural sand, stone screenings, or a combination of both sand and stone screenings. All aggregates must pass a 3/8-in. sieve with 2 to 12 percent passing the No. 200 mesh sieve. The aggregate must be of such quality that the resulting mixture will have a minimum stability of 1500 lb. at 140 F. as determined by the Hubbard-Field test. On some projects on Route 1 a 2000-lb. stability was specified. While specifications may vary slightly from year to year and from job to job depending upon the available aggregate and the amount of traffic, they have remained essentially the same.

Eleven sand asphalt surfaces constructed from 1943 to 1946 were tested. Of these surfaces No. 2 was composed of 25 percent granite screenings and 75 percent natural sand aggregate. The aggregate in surfaces No. 6, 15, and 17 was composed of 100 percent natural sand; Nos. 3, 14, 18, and 24 of 100 percent granite screenings; Nos. 19 and 29 of 100 percent trap rock screenings, and No. 22 of 100 percent limestone screenings. Natural sand is the most commonly used aggregate for this type of surface but numerous sections of pavement have been constructed using the other types of aggregate either in experimental work or because of the availability of the material.

Group III - Special Plant Mix Bituminous pavements of this type consist of one or more courses of asphalt-coated aggregate constructed on a prepared base course or road surface. This is a hot plant-mixed material composed of 5 to 8.5 percent 85-100 penetration asphaltic cement combined with natural sand gravel from local pits with no initial processing other than screening out the oversize material. Each job is designed to give the greatest stability with the aggregate available. Aggregate specifications vary with the available material but usually have a maximum size of 3/4 or 1 in. with a tolerance of 0-5 percent passing the No.200 mesh sieve. These mixes are quite similar in texture to the sand asphalt mix except for the presence of some larger size particles. They are represented in this report by surfaces Nos. 12 and 16.

Group IV - Rock Asphalt (Type G-1) Rock asphalt pavement (type G-1) consists of a wearing surface composed of natural rock asphalt constructed on a prepared base. It is a sandstone rock asphalt obtained from an approved source and prepared in a plant capable of maintaining uniform quality. Preparation of this material consists of grinding only, with no sand or bituminous material added. Natural rock asphalt must have an asphalt content (by extraction) of 6.0 to 7.5 percent. The mixture must pass a 3/4-in. sieve with not less than 99 percent passing a ½-in. sieve and not less than 80 percent passing the No. 4 sieve. Where this material is applied at the rate of 25 lb. per sq. yd. or less, 100 percent must pass the 3/8-in. sieve and 99 percent must pass the No. 4 sieve.

Surface No. 9 consists of a ¾-in. wearing course of natural rock asphalt laid in 1941 on a sheet asphalt base.

Group V - Seal Treatment Seal treatments consist of applying from 0.1 to 0.2 gal. of liquid bituminous material to the road surface and covering with the maximum amount of coarse sand, gravel, slag, or stone chips that the bituminous material will hold. This treatment is rolled with a medium weight roller to set the aggregate into the bituminous material. Slag and crushed stone used for this work are required to pass a 3/8-in. sieve with not more than 10 percent passing a No. 8 mesh sieve. Surfaces Nos. 5, 8, 11 and 30-A are of this type.

Group VI - Bituminous Surface Treatment, Broom Drag Seal This surface is a cold mixed-in-place bituminous surface treatment applied on a prepared base. It is composed of liquid bituminous material combined with washed gravel, crushed stone, or slag. The bituminous material is applied at the rate of 0.1 to 0.2 gal. per sq. yd. and covered with from 20 to 40 lb. of aggregate per sq. yd. An additional application of from 0.2 to 0.4 gal. per sq. yd. of bituminous material is then applied after which it is mixed with a long broom drag and then rolled. During rolling operations from 5 to 8 lb. per sq. yd. of choke material is broomed into the surface. A seal coat of from 0.1 to 0.2 gal. per sq. yd. of bituminous material is then applied and the surface uniformly covered with aggregate which is broomed into the voids and rolled. Aggregate specified for mixing is material passing the ½-in. sieve and retained on the No. 8 sieve or material passing the 3/8-in. sieve and retained on the No. 8 sieve. Each of the above aggregates as well as coarse sand and crushed stone or slag passing the No. 4 sieve and retained on the No. 100 sieve may be used for choke.

Surface No. 7 is a broom drag seal in which washed gravel was used as covering aggregate.

Group VII - Portland Cement Concrete Portland cement concrete pavement is composed of class *p* or class *x* concrete (Virginia specification) and constructed according to approved methods. The texture of the surface is determined by the method of finishing. Belt finishing gives a rather smooth texture with shallow depressions and low ridges. The broom finish which is applied with a stiff broom gives a rougher texture with deep grooves and rather pronounced ridges. Since the grooves in this finish are more or less continuous from the center of the pavement to the edge it allows water to drain off very rapidly so that a continuous film of water is less likely to be found on pavements of this type.

Surfaces No. 4 (constructed in 1938) and 23 (1943) were belt-finished while surface No. 13 (1945) was broom-finished.

Group VIII - Surfaces with Excess Asphalt Pavements in this group have an excess of asphalt on the surface. With this type of surface especially where the asphalt is greatly in excess, the aggregate has little or no effect on the skid resistance of the pavement. Several factors may cause pavement surfaces to get in this condition. A large amount of heavy traffic, particularly in hot weather, may cause more compaction of the mix than was anticipated and thus work the asphalt to the surface. On bituminous surface treatments excess asphalt may be applied at the time the treatment is placed or weather conditions may cause the asphalt to strip from the aggregate so that the aggregate is whipped from the surface leaving an excess of asphalt. Surfaces Nos. 30 and 31 are of this type.

DISCUSSION OF TEST DATA

The complete data for more than 1000 stopping distance tests on 32 pavement surfaces are presented in Table 1. These stopping distances are actual average forward skidding distances and do not include driver reaction time. Other investigators report test results as coefficients of friction; however, in

TABLE 1 SKID RESISTANCE MEASUREMENTS

Location Key	TYPE TIRES / SURFACE CONDITION / SPEED M P H / TYPE SURFACE	A - GOOD TREAD, DRY, 11.2 S.D.	C	20 S.D	C	30 S D	C	40 S D	C	A - GOOD TREAD, WET, 11.2 S.D.	C	20 S D	C	30 S D	C	40 S.D	C
I	BIT CONC. (TYPE H-3 & I-3)																
1	Slag & Nat. Sand (1944)	5 5	76	19 9	67	41 3	73	73 0	71	7 6	55	21 2	63	50 8	59	95 9	56
10	Limestone & Nat Sand (1944)	6 5	65	17 0	78	37 1	81	70 0	76	9 5	44	26 1	51	61 1	49	120 5	44
21	Limestone & Nat Sand (1945)	5 5	76	18 0	74	38 8	77	68 5	78	7 3	58	26 5	50	57 3	52	91 4	58
26	Limestone & Nat Sand (1946)	4 7	89	17 2	78	34.2	87	75 5	71	6 4	66	20 6	63	48 5	63	96 2	50
28	Limestone & Nat Sand (1945)	4 7	70	16 8	79	40 4	74	70 4	75	6 0	89	18 6	72	47 0	64	84 2	63
25	L.S., Granite & Nat Sand (1944)	6 3	67	17 7	75	37 1	81	70 9	75	6 7	63	21 4	63	46 1	65	83 3	60
20	100% Limestone (1945)	6 4	66	16 9	79	36 1	83	65 7	81	8 9	47	28.5	49	67 7	44	120 1	44
27	100% Limestone (1945)	4 7	89	16 7	80	37 7	79	69 3	77	5 9	71	21.7	62	58 9	81	97.7	54
	Average	5 5	74	17 5	76	37 8	79	70 4	75	7 2	61	23 0	59	54 6	55	98 5	53
II	SAND ASPHALT (TYPE F-1)																
2	25% Gr. Scr, 75% Nat. Sand (1945)	4 7	89	16 3	83	34 3	87	66 2	80	4 5	65	18 1	74	46 5	64	78 0	68
3	100% Granite Scr. (1945)	4 7	89	17 1	78	36 4	84	69 1	77	5 9	71	22 2	60	50 4	59	110 8	48
14	100% Granite Scr. (1945)	5.4	77	18 2	73	38 0	79	73 1	73	7 4	56	20 5	63	44 8	67	83 8	64
18	100% Granite Scr (1946)	6 4	66	17 0	78	41 0	73	73 8	72	6 1	52	21 6	62	46 2	65	87 6	61
24	100% Granite Scr (1944)	5 5	76	17 7	75	39 2	77	70 2	76	9 0	47	27 0	49	59 2	51	115 9	46
19	100% Trap Rock Scr (1945)	6 7	62	19 5	68	40 3	74	76 1	70	7 7	55	21 0	63	42 8	70	82 0	65
29	100% Trap Rock Scr (1946)	5 5	76	15 8	84	35 3	85	69 2	77	7 2	58	21 7	62	53 6	56	99 5	54
22	100% Limestone Scr (1944)	6 4	66	17 7	75	39 4	76	72 7	73	6 4	66	19 9	67	52 2	57	95 2	56
6	100% Nat. Sand (1943)	5 4	78	18 5	73	39 4	76	73 4	73	6 4	66	19 9	67	44 5	67	87 6	61
15	100% Nat. Sand (1946)	5 1	82	17 8	75	34 6	87	64 8	82	6 1	69	21 4	62	41 3	73	86 0	62
17	100% Nat Sand (1945)	6 0	70	19 9	67	39 5	76	64 3	80	7 3	58	21 0	63	43 0	70	72 9	73
	Average	5 6	75	17 7	75	37 9	79	70 4	75	7 0	60	21 3	63	47 6	62	90 8	59
III	SPECIAL PLANT MIX																
12	Local Sand & Gravel (1946)	6 6	64	18 9	71	39 3	76	67 0	79	6 9	61	19 6	68	39 9	75	79 8	67
16	Local Sand & Gravel (1940)	6 4	66	17 2	77	39 5	76	64 9	80	6 2	68	18 9	71	46 1	65	72 0	74
	Average	6 5	65	18 0	74	39 4	76	66 9	79	6 5	64	19 2	69	43 0	70	75 9	70
IV	ROCK ASPHALT (TYPE G-1)																
9	Rock Asphalt (1941)	5 5	76	16 2	82	39 0	77	67 7	79	5 6	75	18 6	72	41 6	72	77 7	69
V	SEAL TREATMENTS																
5	Sand Seal (1946)	5 0	84	18 1	74	37 6	80	63 6	84	6 0	70	18 5	73	43 0	70	81 0	66
8	Slag Seal (1938)	5 6	75	17 0	78	42 8	70	79 3	68	5 6	75	20 0	67	45 5	66	83 5	64
11	Slag Seal (1946)	5 6	75	19 0	70	41 7	72	76 0	70	7 3	58	22 0	61	46 9	64	82 3	65
30-A	Crushed Stone Seal (1947)																
	Average	5 4	78	18 0	76	40 7	74	73 0	74	6 3	67	20 2	64	45 1	66	82 3	65
VI	BIT SURF TREAT - BROOM DRAG SEAL																
7	Broom Drag - Gravel (1946)	6.4	66	19 3	69	40 7	73	73 3	73	9 8	43	28 6	47	63 9	45	111 3	48
VII	PORTLAND CEMENT CONCRETE																
4	Concrete-Belt Finish (1938)	4 9	86	15 7	85	36 4	82	67 6	79	7 3	58	24 9	54	62 7	48	112 2	47
23	Concrete-Belt Finish (1943)	6 8	65	19 6	68	39 0	77	70 6	76	9 1	44	28 8	46	67 1	45	111 0	48
13	Concrete-Broom Finish (1946)	5 4	77	17 0	78	39 1	77	73 3	73	6 2	68	16 9	79	42 7	70	84 3	63
	Average	5 6	76	17 4	73	38 3	78	70 5	76	7 8	57	23 5	59	57 8	54	102 5	53
VIII	SURF WITH EXCESS ASPHALT																
30	Pentr. Treat. (1944), Seal (1945)							(Sealed with RC-2 and No. 11 Stone - Very Fat)									
31	Mixed-in-Place S.T. Sealed (1939)							(Sealed with No. 12 Stone - Very Fat)									
	Average																
	GRAND AVERAGE	5 6	74	17 7	75	38 4	78	70 4	75	7 1	61	21 9	62	50 3	60	92 5	58

Location Key	TYPE SURFACE	B - GOOD TREAD, DRY, 11.2 S D	C	20 S D	C	30 S D	C	40 S D	C
1	Slag & Nat. Sand (1944)	4 2	99	16 5	81	40 8	74	77 9	[illegible]
10	Limestone & Nat Sand (1944)	4 0	1 05	15 4	86	39 2	76	70 7	[illegible]
	Average (I)	4 1	1 03	15 9	83	40 0	75	74 3	72
2	25% Gr. Scr, 75% Nat. Sand (1945)	4 1	1 02	15 8	85	37 2	80	67 4	79
3	100% Granite Scr. (1945)	4 5	93	16 5	81	37 4	80	70 5	76
14	100% Granite Scr. (1945)	4 6	91	16 2	82	39 0	78	72 2	74
15	100% Nat. Sand (1946)	4 3	98	16 6	80	39 2	77	74 3	72
	Average (II)	4 3	96	16 2	82	38 3	78	71 1	75
16	Local Sand & Gravel (1940)	4 7	89	17 3	77	41 2	73	69 5	76
	Average (III)	4 7	89	17 3	77	41 2	73	69 5	76
9	Rock Asphalt (1941)	4 4	95	16 3	82	39 7	75	72 2	72
5	Sand Seal (1946)	5 0	84	17 6	76	38 9	77	73 8	72
8	Slag Seal (1938)	4 8	88	18 2	73	43 7	68	88 9	60
11	Slag Seal (1946)	4 9	86	17 3	77	38 8	78	75 5	70
	Average (V)	4 9	86	17 7	75	40 4	74	79 4	67
7	Broom Drag - Gravel (1946)	4 2	99	16 5	81	39 0	77	68 8	[illegible]
4	Concrete-Belt Finish (1938)	4 7	89	16 3	82	37 2	80	72 0	[illegible]
13	Concrete-Broom Finish (1946)	4 4	91	17 1	78	40 7	74	76 4	[illegible]
	Average (VII)	4 6	90	16 7	80	36 7	77	74 3	[illegible]
	GRAND AVERAGE	4 5	93	16 6	80	39 4	76	73 5	[illegible]

S.D - AVERAGE SKIDDING DISTANCES IN FEET
C - AVERAGE COEFFICIENT OF FRICTION

this report the stopping or skidding distance is used since it is believed that such values may be more readily visualized. All test results presented graphically are plotted as forward skidding distances. In order that these results might be readily compared with those of others, Table 1 includes the average coefficient of friction as computed from the following standard stopping distance formula:

$$f = \frac{v^2}{30\,S}$$

where f = average coefficient of friction

v = initial speed in mi. per hr. at the time of applying brakes

S = average stopping distance in ft.

Results are grouped according to the particular pavement types previously described for both dry and wet surfaces. Average values are given by groups as well as a grand average for each type of tire. Where the surface was tested with all four tires in a wet condition average values are given in the right hand column.

With the exception of 30, 30-A, and 31, all surfaces were tested with type A tires. The primary objective was to evaluate the skid resistance of surfaces; however, additional tests were performed to determine the effect of the tire variable. Also, tests were made to ascertain the effectiveness of nonskid treatments on a glazed surface.

Comparison of Surface Types Results of all measurements for the 29 surfaces tested with tire A are shown graphically in Figure 10. These are grouped according to surface types and arranged in order from short to long stopping distance

TABLE 1 - Continued

	B - GOOD TREAD WET						C - WORN TREAD WET								D - WORN TREAD WET								AVERAGES WET							
11.2	20		30		40		11.2		20		30		40		11.2		20		30		40		11.2		20		30		40	
C	S D	C	S D	C	S D	C	S D	C	S D	C	S D	C	S.D	C	S D	C	S D	C	S D	C	S D	C	S D	C	S D	C	S.D	C	S D	C
84	16.8	79	43.4	69	96.4	55	6.9	61	22.3	60	52.4	57	111.3	48																
65	23.4	58	66.7	47	164.4	33																								
74	20.1	68	54.0	58	130.4	44	6.9	61	22.3	60	52.4	57	111.3	48																
75	19.7	68	47.5	63	98.6	54	6.4	66	23.1	58	64.6	46	140.0	38																
71	20.0	67	47.4	63	110.5	46	6.7	63	29.4	45	84.1	36	204.6	26																
67	20.9	64	49.1	61	90.8	59	6.0	70	22.4	60	61.0	49	161.2	33	5.9	71	24.6	54	43.1	48	131.6	41	6.4	66	22.1	60	54.5	56	116.6	49
78	19.4	.69	45.0	66	84.1	63	6.3	67	19.9	66	48.6	62	86.0	62	6.2	68	19.8	67	44.1	68	85.2	63	6.0	70	20.1	66	44.7	67	85.3	.62
72	20.0	67	47.2	63	96.0	55	6.3	64	23.7	57	64.5	48	147.9	39																
73	19.0	70	45.6	66	86.2	62	5.8	73	22.3	60	58.6	52	134.3	40	5.7	77	20.0	67	54.7	55	124.8	43	5.8	72	20.0	67	51.2	59	104.3	54
73	19.0	70	45.6	66	86.2	62	5.8	73	22.3	60	58.6	52	134.3	40																
84	18.1	72	41.3	73	81.7	66	4.9	86	19.7	68	49.6	61	125.0	43	4.5	93	15.4	87	48.3	62	107.0	.50	5.0	84	17.9	74	45.2	67	97.8	57
81	19.4	69	44.6	65	98.7	55																								
.86	18.0	74	45.3	66	92.1	58	5.5	76	21.5	64	52.6	57	123.1	43	4.7	89	17.2	78	47.0	64	107.7	50	5.1	81	19.1	70	47.6	63	101.6	53
76	19.8	68	47.2	64	100.3	53																								
							5.9	71	20.3	64	48.6	62	94.7	58																
81	19.0	70	46.3	65	97.1	55	5.7	73	20.9	65	50.6	59	108.9	50																
53	26.1	51	60.2	50	108.5	49	7.2	56	29.4	45	64.5	47	119.7	44																
68	25.0	54	63.5	47	128.7	42	8.1	52	33.8	39	84.3	36	203.3	26																
78	17.9	74	53.3	58	106.9	50	5.0	84	21.6	62	60.8	49	128.3	41	6.2	68	19.8	67	54.8	55	123.6	43	5.7	74	19.0	70	.53.9	58	110.5	49
73	21.4	64	58.4	52	114.8	46	6.5	68	27.7	50	72.5	47	165.8	33																
							12.3	.27	51.0	26	130.7	22	224.0	23																
							10.2	10	72.0	19	160.0	19	254.5	21																
							13.2	22	61.8	22	145.3	20	229.2	22																
74	20.2	67	50.0	61	103.4	53	7.5	62	29.1	52	72.8	46	150.7	39	5.8	78	19.8	70	53.0	59	113.1	48	5.7	74	19.7	68	49.3	61	102.7	54

values. An inspection of the chart reveals that at the lower speeds less difference is found in the surfaces than at the higher speeds. Differences between dry and wet surfaces are also more pronounced at higher speeds. Consequently, it appears that the results obtained on a wet surface at 40 mi. per hr. can be used successfully to compare the skid resistance of the various pavements.

Considering data in Figure 10 and a speed of 40 mi. per hr., stopping distances from 63.6 to 79.3 ft., and averaging 70.4 ft., were measured on dry surfaces. Corresponding computed coefficients of friction ranged from 0.84 to 0.68 and averaged 0.75. Thus it can readily be seen that all 29 surfaces have excellent skid resistance when tested dry.

Values for the same surfaces when wet (at 40 mi. per hr.) ranged from 72.0 to 120.5 and averaged 92.5 ft. The corresponding computed coefficients of friction varied from 0.74 to 0.44 and averaged 0.58. If the surfaces are rated according to previous investigators on the basis of stopping distances of 133 ft. (coefficient of friction 0.4) then all 29 surfaces would fall within this limit. According to AASHO design standards (4) in which a factor of safety of 1.25 is used, the safe braking distance on wet level surfaces at 40 mi. per hr. should not exceed 113 ft. (coefficient of friction 0.47). If the 29 surfaces are evaluated on this basis, then all but three are found to be within this limit. The three locations not meeting AASHO requirements are surfaces 10, 20, and 24. Two of these surfaces are bituminous concrete type H-3 (10 and 20). Surface 10 contained an excess of asphalt which under heavy traffic

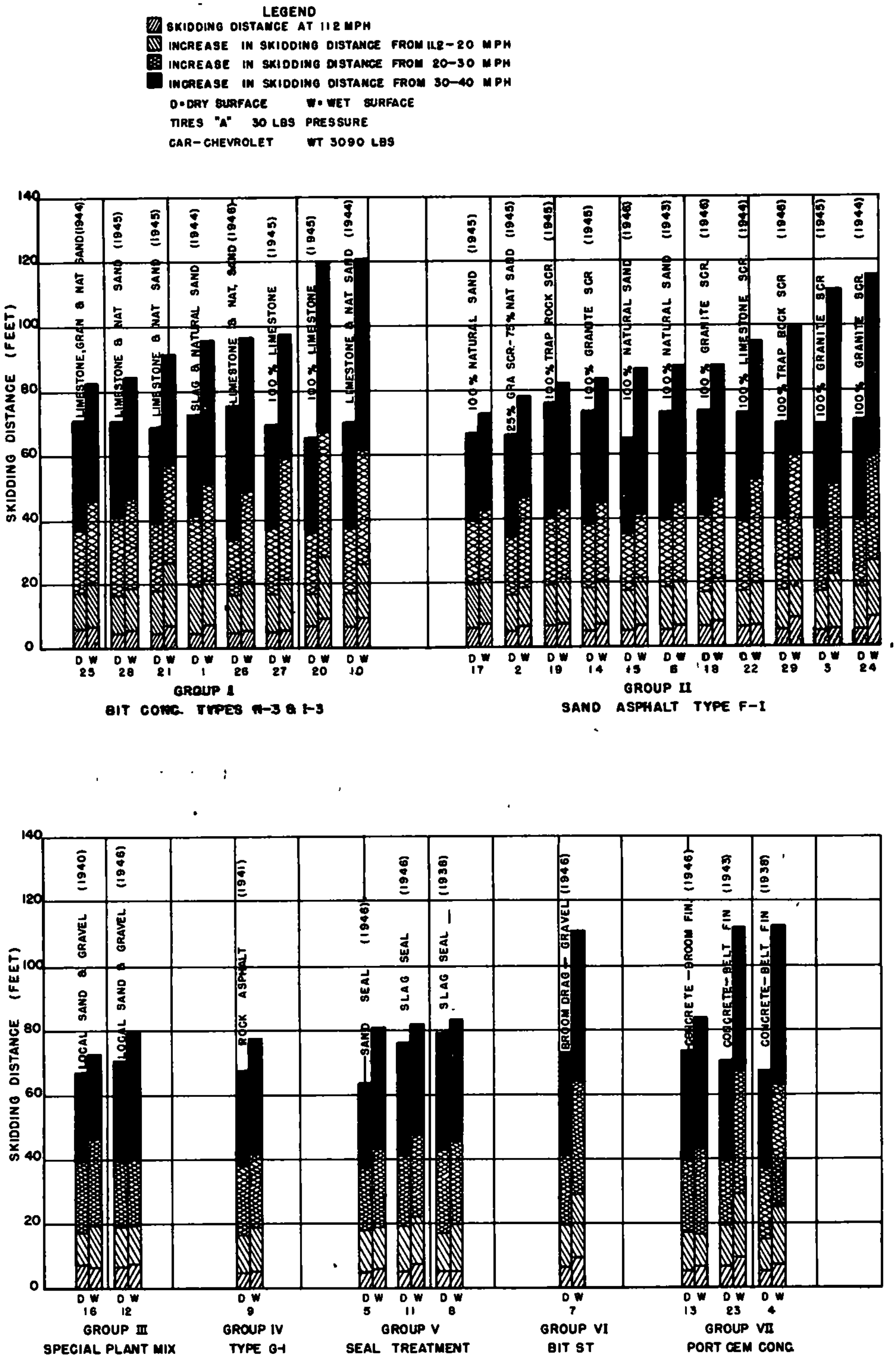


Figure 10. Skid Resistance Measurements on Various Pavement Surfaces

was worked to the surface resulting in a glazed texture. In fact, since this surface was tested it has been replaced with a safer surface. Surface 20 is a short experimental section approximately 500 ft. in length. Although laboratory test reports do not show that the mixture contains an excess of asphalt, the surface appears to be glazed. Polishing of the limestone

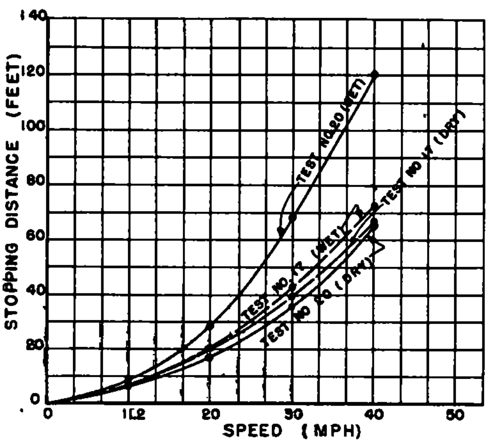


Figure 11. Speed vs. Stopping Distance - Wet & Dry Surfaces (Tires A)

aggregate in this mix may be a contributing factor. The other surface (24) is a sand asphalt pavement (type F-1) containing crushed stone aggregate with a high abrasion loss. This mixture also contained an excess of asphalt which combined with the soft polished aggregate resulted in a surface with poor skid resistance.

While the results on the remaining 26 surfaces indicate them to be entirely within prescribed limits, certain comparisons of materials and methods of finishing are extremely interesting. For the surfaces tested wet (tires A) the eight groups listed according to stopping distances from the lowest to the highest are as follows: special plant mix, rock asphalt, seal treatments, sand asphalt, bituminous concrete, portland cement concrete, broom drag treatment, and glazed bituminous surfaces. Since equal samplings were not practical for the various surfaces too much emphasis should not be placed upon this rating. It does indicate certain pertinent trends.

To illustrate the speed versus stopping distance relationship, Figure 11 has been prepared. Curves are shown for two surfaces - a bituminous concrete (20) and a sand asphalt (17). It will be noted that when dry, lower stopping distances were obtained on surface 20 than on surface 17; however, after these surfaces were sprinkled with water stopping distance values for surface 20 are much higher than those for surface 17. In fact, wetting the surface of the sand asphalt had only a slight effect on the stopping distance. Such curves can be used to predict stopping distances at speeds other than those tested. For example, at 50 mi. per hr., the stopping distance on surface 17 (wet) would be approximately 110 ft.

With fine-textured surfaces such as sand asphalt, rock asphalt, special plant-mix and the seal treatments, the aggregate particle shape and resulting surface texture is very important from a skidding standpoint. The use of aggregate containing the more cubical particles resulted in a gritty or sandpaper-like texture which was found to give shorter stopping distances than if the aggregate particles were flat. Sharp natural sands, and slags are typical examples of cubical particles. This is illustrated by Figure 12 where curves for wet F-1 surfaces are shown. Surface 6 is a natural sand asphalt and No. 3 is a relatively soft granite screenings mixture. In analyzing the data it was found that for stone screenings (F-1) mixtures stopping distances varied directly with the abrasion loss of the aggregate. In other words, those surfaces containing aggregates with low abrasion losses were found to produce shorter stopping distances than those containing aggregates with higher abrasion losses.

In general the coarser textured surfaces (bituminous concrete and broom drag treatment) offered less resistance to skidding when wet than the fine-textured surfaces. With the coarser textured surfaces some of the larger particles were observed to polish under traffic. This was particularly noticeable on surface 7.

The data obtained on concrete pavements while not extensive permits a comparison of belt-finished and broom-finished surfaces. In a dry condition the smoother surface offers slightly greater resistance to skidding; however, as has previously been stated, none of the surfaces tested dry were found to be critical. In a wet condition the broom finish was found to be far superior. For example, stopping distance values for both the dry and wet conditions (at 40 mi. per hr.) were as follows:

	Stopping Distances			
	Dry ft.	Wet ft.	Increase ft.	Increase percent
Belt Finish (avg. of 2)	69.1	111.6	42.5	61.5
Broom Finish	73.3	84.3	11.0	15.0

This further illustrates the importance of the differences in stopping distance of a surface from a dry to a wet condition.

A comparison of surfaces 10 and 11 are of interest since they are adjacent lanes of a 4-lane pavement. Surface 10 is the outside lane and surface 11 the inside lane. Surface 11 was originally the same as 10; however, it was sealed early in February 1946 with tar (RT-5) and slag aggregate (½-in. No. 8). A comparison of the stopping distances at 40 mi. per hr. with tires A are as follows:

	Stopping Distance			
	Dry ft.	Wet ft.	Increase ft.	Increase percent
Surface 10	70.0	120.5	50.5	72.3
Surface 11	76.0	82.3	6.3	8.3

Thus, it can be seen that although the stopping distance on surface 10 is less than that on surface 11 in a dry condition, the important item from a safety standpoint is the increase in stopping distance from a

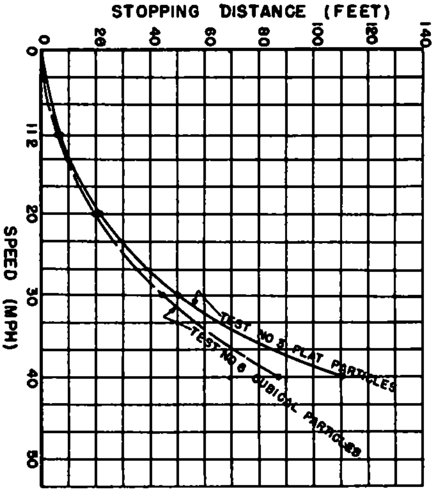


Figure 12. Comparison of F-1 Surfaces (Wet) on Basis of Particle Shape (Tires A)

dry to a wet condition. It will be observed that this percentage increase is nearly nine times greater for surface 10 than for surface 11.

Surfaces on which a relatively low increase in stopping distance from the dry to the wet condition were measured include locations 17, 19, 16, 12, 11, 9, 8, and 13. From a safety standpoint these surfaces should be considered much better than those in which the stopping distance is greatly different when the surface changes from a dry to a wet condition.

In performing the tests on the finer textures it was observed that when a pavement was first sprinkled lightly the stopping distances were greater than after the surface was

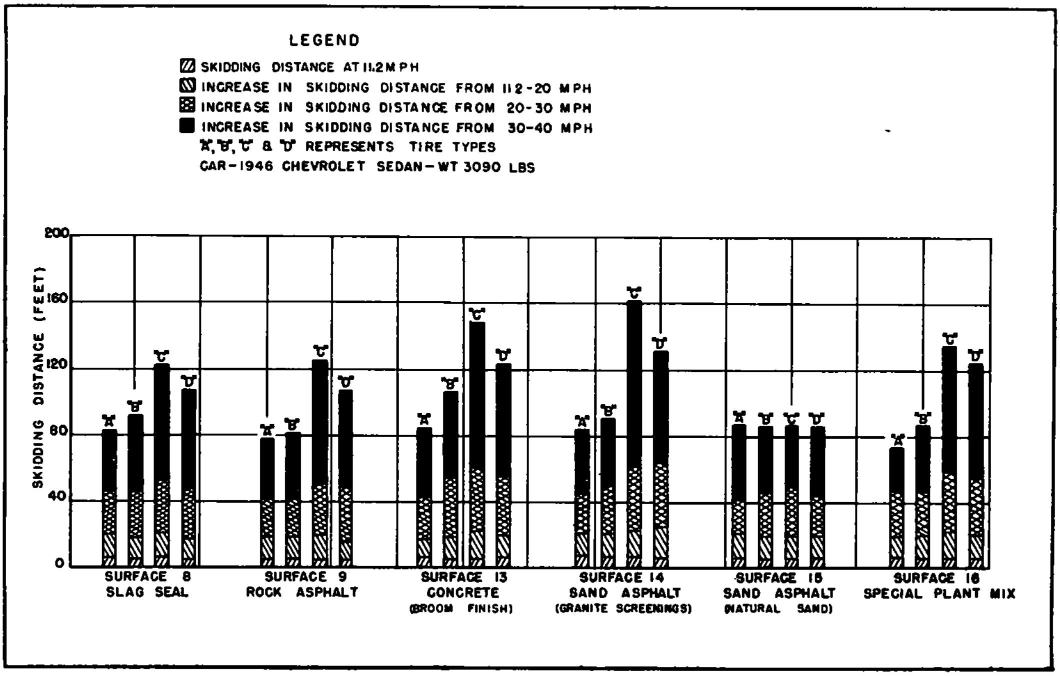


Figure 13. Skid Resistance Measurements on Six Surfaces (Wet) with Four Tire Conditions

thoroughly wet. This was noticed particularly on the finer-textured surfaces subjected to large volumes of traffic and on the roads on which oil drippings were more prevalent. It seems logical that a thin oily film may serve as a better lubricant than a thicker film of plain water. After complete soakings much of the oil may have been washed away. While enough data were not available to prove this point conclusively, the trend was noticeable.

Tire Variable In studying forward skidding it is recognized that the two principal factors involved are the pavement surface and the tire. While the main objective was an evaluation of surfaces, the tire variable could not be neglected. Only six surfaces were tested in a wet condition with all four sets of tires. Figure 13 is a bar graph of the measurements and permits a comparison of the skid resistance of the four tires on the six surfaces. With the exception of surface 15, decided differences were found with the four tires. For some unaccountable reason this surface (15) was found to have excellent skid characteristics with all four tires. With the exception of surface 15 the tires rated in order of skid resistance are as follows: A, B, D, and C. It will be observed that at 30 mi. per hr. on surface 15 the rating of the tires is in the same order as for the other surfaces at 40 mi. per hr. The tire variable is more pronounced on some surfaces than on others. For example, greater differences were found in stopping distances on surface 14 than on surface 8. The stopping distance at 40 mi. per hr. on surface 8 was only 47.5 percent longer with tires C than with A, whereas on surface 14 tires C produced 92.5 percent greater stopping distance than tires A.

The same data are presented in

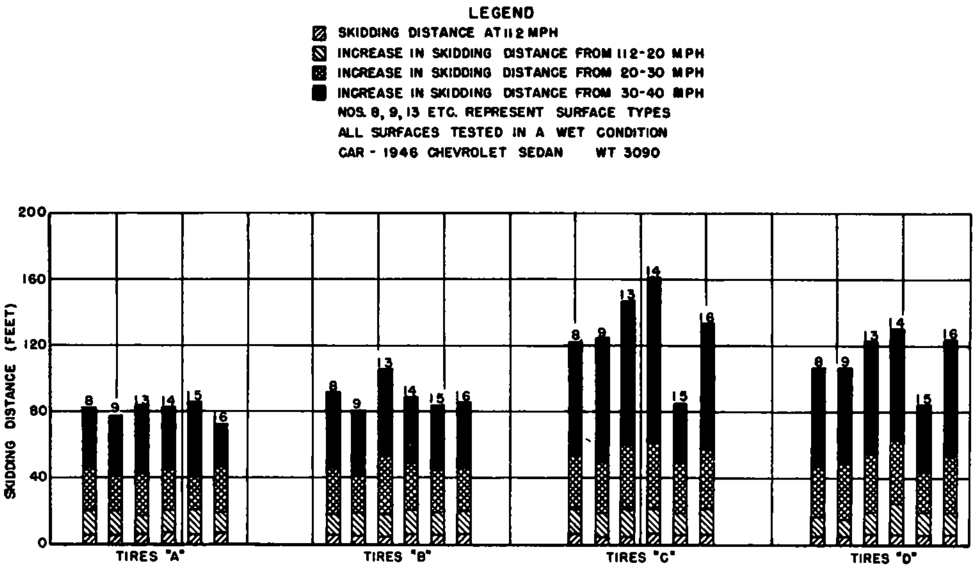
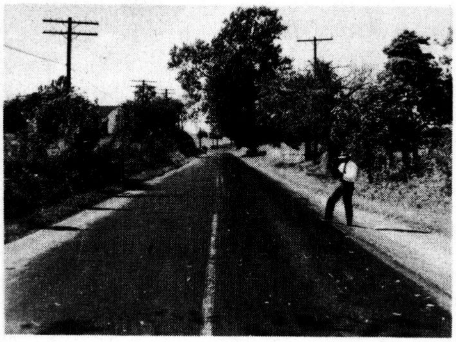


Figure 14. Skid Resistance Measurements with Four Tire Conditions

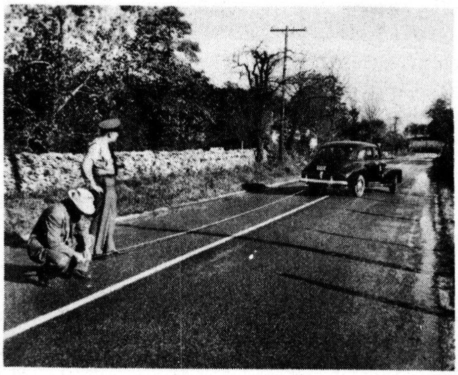
Figure 14; however, in this case values for all six surfaces are grouped together for each set of tires. Stopping distances for the good tread tires (A and B) are much lower and more uniform than for smooth tread tires. Smooth tread natural rubber tires were found to be safer than smooth tread synthetic tires (S-3). The average stopping distance for D tires was 113.1 ft. as compared to 126.3 ft. for C tires. Slightly higher values were obtained with B tires than with A tires. The average stopping distance at 40 mi. per hr. for the six surfaces was 81.2 ft. for A tires as compared with 90.3 ft. for B tires. Average stopping distances for smooth tread tires (C and D) were 39.5 percent longer than for good tread tires (A and B). As previously mentioned air and pavement temperatures were recorded at the time of each series of test. It so happened that tires A were tested first during periods of much lower temperatures. Average pavement temperature for the six surfaces tested with tires A was 57 F. as compared with averages of 80 to 90 F. at the time of testing the other three tires. This may partially account for the difference in stopping distances with tires A and B. Prof. Moyer reports a definite increase in coefficient of friction with a decrease in temperature; however, he found that this was more marked in the side-skid than in the straight-skid coefficients.

Non-Skid Treatment Two surfaces (30 and 31) in which there was a noticeable excess of asphalt flushed to the surface were tested wet with tires C. Several accidents had been reported on these sections and they were included in the Department's non-skid program. To determine the effectiveness of non-skid treatments, measurements were made on both of them prior to treatment and on one of them a short time afterward.

These glazed surfaces gave the least resistance of any pavements tested. At 40 mi. per hr. on the wet surfaces the stopping distance was 224 ft. for location 30 and 254.5 ft. for surface 31. The non-skid treatment consisted of priming the surface with about 0.1 gal. per sq. yd. of cut-back asphalt (RC-2A), covering immediately with about 20 lb. per sq. yd. of No. 12 (3/8-in. No. 10) stone chips and compacting. Traffic was not permitted on the surface for a 12-hr. period. Measurements made approximately five weeks later on the non-skid treatment (30-A) revealed that



BEFORE NON-SKID TREATMENT



AFTER NON-SKID TREATMENT

Figure 15. Route 55
Fauquier County

the stopping distance at 40 mi. per hr. had been reduced from 224 to 94.7 ft. (Figs. 15 and 16). An inspection of Figure 16 clearly reveals that the stopping distances have been improved at all speeds but more particularly at the higher

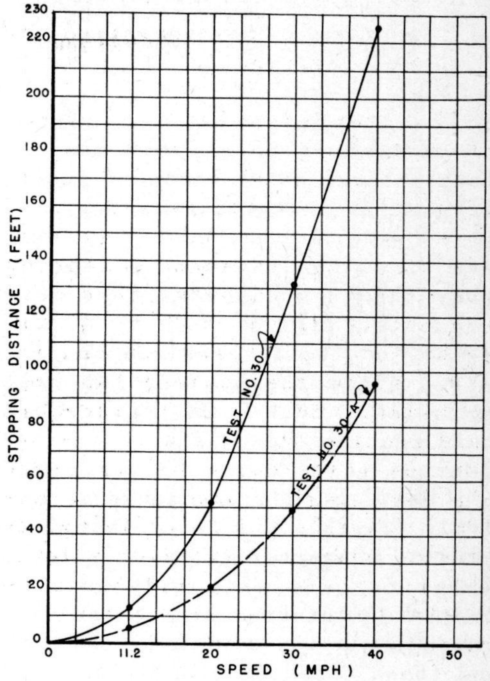


Figure 16. Before and After
Non-Skid Treatment
(Surfaces Wet) Tires C

ones. From the curve it can be predicted that the stopping distance at 50 mi. per hr. on surface 30-A would be approximately 160 ft. Before treatment this same stopping distance would have resulted at a speed of about 33 mi. per hr.

Several seal treatments were tested and found to have good non-skid characteristics. It should not be inferred that all seal treatments have such characteristics. In fact, the sealing of a surface is an exacting operation and if not controlled carefully or done under ideal weather conditions a slick surface may result. If an excess of asphalt is present the surface has a tendency to bleed during hot weather and may become slick. Plant-mixed materials permit a better control over the proportions than is normally possible by road-mix methods. The Department is experimenting with lean sand-asphalt (4 to 5 percent) plant-mixed non-skid treatments; however, time did not permit before and after tests on these treatments.

The stopping distance method proved to be a relatively simple, practical and quick means of evaluating surface characteristics. If curves were not needed to approximate stopping distances at speeds other than test speeds, then measurements could be made at 40 mi. per hr. on the dry surface and repeated after the surface had been sprinkled. Such a test procedure would require only a short time and numerous tests could be performed in a day. Such a procedure is entirely feasible for determining the skidding characteristics of a large number of pavements and could be used in conjunction with a non-skid program.

SUMMARY OF RESULTS

Based upon more than 1000 measurements of forward skidding distances on 32 pavement surfaces with the four tire variables and under the conditions of tests previously described the more important results have been summarized as follows:

General

1. On relatively smooth level pavements the stopping distance method is an excellent means of determining the average skid resistance. The equipment involved is relatively simple and inexpensive. Tests can be performed and checked in a relatively short time. With few exceptions close check results were obtained. In many instances differences between repeat tests were only a few tenths of a foot apart.

2. At 40 mi. per hr. and with all four types of tires the stopping distances varied from 63.6 to 88.9 ft. on a dry and from 72.0 to 254.5 ft. on a wet surface. Corresponding coefficients of friction ranged from 0.84 to 0.60 on a dry and from 0.74 to 0.21 on a wet surface. These data indicate clearly that stopping distances at this maximum speed are critical only on the wet surfaces. It was considered unsafe to conduct tests on wet surfaces at a speed greater than 40 mi. per hr.

3. Skidding distances increased with speed. For the 29 surfaces tested with tires A the average stopping distances are as follows:

Speed, mi. per hr.	Dry Surface ft.	Wet Surface ft.
11.2	5.6	7.1
20	17.7	21.9
30	38.4	50.3
40	70.4	92.5

These results emphasize the importance of speed from a safety standpoint, particularly on wet pavements.

4. The data can be used to establish departmental policies concerning the design, construction, and maintenance of surfaces with good non-skid characteristics. The data obtained checks reasonably closely with that of previous investigators.

5. The results obtained at the four speeds can be used to predict stopping distances at other speeds.

Surface

6. All of the pavements tested in a dry condition were found to have satisfactory resistance to skidding for speeds of 40 mi. per hr. or less.

7. Twenty-seven of the 32 surfaces tested wet were considered to have satisfactory non-skid characteristics. Three of the five surfaces (10, 30, and 31) found to be unsafe from a skidding standpoint have been rebuilt or resurfaced.

8. For the surfaces tested wet the eight groups listed in accordance

with the stopping distances from the lowest to the highest are as follows: special plant mix, rock asphalt, seal treatments, sand asphalt, bituminous concrete, portland cement concrete, broom drag treatments, and surfaces with excess asphalt. It should be pointed out that this rating applies only to the surfaces tested, and since only a few surfaces are included in some groups, too much emphasis should not be placed on this rating.

9. Skidding resistance varies with the texture and composition of the pavement surface. Those surfaces having a harsh, gritty, sandpaper-like texture were found to have short stopping distances. Typical examples of this condition are surfaces 17, 2, 19, 16, 12, 9, 5, 11, 8, and 13. On the other hand, glazed surfaces such as 30 and 31 were found to have long stopping distances and correspondingly low coefficients of friction.

10. Broom-finished portland cement concrete pavement was found to be much safer from a skidding standpoint than smooth or belt-finished concrete. Stopping distances on dry broom-finished concrete are slightly higher than on belt-finished surfaces; however, on wet surfaces stopping distances for the belt-finished were considerably higher than for the broom-finished pavement. It was observed that the broom-finished surface allows water to drain off very rapidly so that a continuous film of water is less likely to be encountered on such a surface.

11. With sand asphalt (F-1) containing stone screenings (plant-mixed), the stopping distance at 40 mi. per hr. on a wet surface varied directly as the abrasion loss of the stone screenings, other factors being equal. In other words, long stopping distances were measured on the F-1 surfaces containing 100 percent stone screenings with high abrasion loss and short stopping distances were encountered with surfaces containing low abrasion loss stone screenings.

Tires

12. Good tread tires were found to offer greater resistance to skidding than smooth or worn tires. This was particularly true on the fine-textured surfaces and not so pronounced on the more open-textured ones such as the broom-drag treatment. Considering only the tests at 40 mi. per hr. on the six surfaces tested wet with all four tires, the average stopping distance was 39.5 percent longer for worn than for good tread tires.

13. The composition of the tire is a factor affecting skidding. While this phase of the problem was not investigated thoroughly, under comparable conditions the average stopping distances with smooth tread synthetic (S-3) tires were 11.7 percent longer than with smooth tread natural rubber tires.

Non-Skid Treatments

14. Non-skid treatments are effective in producing surfaces with good resistance to skidding. In the case tested, the stopping distance on the wet surface was reduced from 224 to 94.7 ft. at 40 mi. per hr.

ACKNOWLEDGEMENTS

The authors wish to acknowledge, with sincere appreciation, the help given by all those who have assisted from time to time in this investigation. The study was initiated by Mr. Burton Marye, Traffic and Planning Engineer, authorized by Mr. C. S. Mullen, Chief Engineer, and conducted under the general supervision of Mr. Shreve Clark, Engineer of Tests.

Professor R. A. Moyer of Iowa State College gave very valuable advice on the type of equipment necessary and the procedure for making the tests. The detonator was loaned by Mr. Earl Allgaier, Research Engineer of the American Automobile Association. One set of tires was supplied by Mr. R. D. Evans, Manager-Tire Design Research of The Goodyear Tire and Rubber Company. State Police officers were very cooperative in directing traffic during tests as were Resident Engineers in furnishing water wagons and flagmen. The Equipment Division furnished and maintained the test car in excellent operating condition. Special commendation should be given to Messrs. Frank Cummings and M. B. McReynolds for their careful and skillful handling of the car during tests. Members of the photographic laboratory of the Landscape Division took all photographs and made the necessary prints.

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DISCUSSION

R. A. Moyer, *Iowa State College*

I would like to take this opportunity to congratulate Messrs. Shelburne and Sheppe for their interesting report of skid resistance measurements of Virginia pavements using the relatively simple method of locked wheel braking in stopping distance tests from various initial speeds with a popular make passenger car. We have used this method of test in similar investigations at Iowa State College at various times during the past 10 years. This method has also been used in braking tests on ice, snow, and on bare pavements by the National Safety Council's Committee on Winter Driving Hazards, of which I am chairman. A report of the results obtained during the past two years in the tests conducted by the Committee on Winter Driving Hazards was presented at one of the sessions of the Traffic Department.

In this discussion I would like to present the results of some recent braking tests on Iowa pavements in connection with a program of resurfacing of concrete pavements using bituminous surface treatments. Only a limited number of braking tests were run but the resurfacing of concrete pavements is being carried out on a large scale in many states and this has led to some interesting problems, one of which is the determination of the effectiveness of bituminous surface treatments in providing a smooth non-skid surface.

The tests in Iowa revealed certain important features in this type of bituminous resurfacing, and for this reason it seems appropriate to present the results in this discussion to supplement the results and observations by Shelburne and Sheppe. At this time, I would, also, like to emphasize the need and value of tests of this type which can and should be run by all state highway departments where "slippery when wet" pavements are responsible for many needless skidding accidents. The only special equipment needed in running stopping distance tests is an electric detonator[1] or a fifth wheel with an accurate speed indicator and stop meter[2] to measure the stopping distance automatically after the brakes are applied.

During the past six years a considerable mileage of old concrete pavement in Iowa has been resurfaced with bituminous surface treatments as a maintenance measure where extensive cracking and a certain amount of surface scaling and pitting have indicated the need for such treatment. Many of these treatments have been made on the heavily traveled Lincoln Highway (U.S. 30) which crosses the state east and west and which was the first highway in Iowa to be paved with concrete across the state. These bituminous treatments have been effective in eliminating further cracking, scaling, and pitting of old concrete pavements at a moderate cost, but the Iowa State Highway Patrol reported an excessively large number of skidding accidents on sections of pavement so treated. The skid tests were run to provide a measure of the relative slipperiness of the treated sections and to determine the cause of the slippery condition and remedies to prevent it.

The test procedure in these tests

1 The electric detonator of the type used by many investigators and as described in the paper by Shelburne and Sheppe, may be purchased from the American Automobile Association, Traffic Engineering and Safety Department, Pennsylvania Ave. at 17th St., Washington, D. C. The cost is approximately $20.00.

2 A fifth wheel device (the Wagner Stopmeter) with speed indicator and stopmeter may be purchased from the Wagner Electric Corporation, 6400 Plymouth Ave., St. Louis 14, Missouri.

was the same as that outlined by Shelburne and Sheppe except that in this exploratory study the test runs were all made at a speed of approximately 30 m.p.h. instead of the preferred speeds of 10, 20, 30, and 40 m.p.h. on each surface. All of the tests were run with a recent model popular make car equipped with new rib-tread synthetic rubber tires.

As an added feature to aid in the interpretation of the test results samples were taken from each bituminous surface tested and the percent of asphalt in the samples was determined in the laboratory. The asphalt determinations were made because previous studies showed that bituminous surfaces with high asphalt contents are quite certain to be slippery when wet. In hot weather these surfaces are subject to bleeding and have a tendency to form a glaze of pure asphalt over the surface which makes them slippery when wet.

The surface treatments have varied from light to heavy using various types of cover material. In 1941 the treatments consisted of the application of 0.6 gal. per sq. yd. of an R.C.-4 cut-back asphalt and 60 lb. per sq. yd. of stone chips (maximum size ½ in.) as cover material. In 1945 the above amounts were cut in half and pea gravel (3/8 in. maximum size) was used as cover material. Also in 1945 treatments with as low as 0.20 gal. per sq. yd. of an R.C.-5 cut-back asphalt and 20 lb. per sq. yd. of sand, were used.

In Table 1 the results of the braking tests are given for the various types of road surfaces together with a detailed description of the surfaces. Stopping distances were obtained for the dry pavement condition on only three surfaces. The results in the dry pavement tests gave the usual consistently low values with stopping distances considerably shorter than for the same pavements when wet. Locked wheel stops on dry pavements take a large amount of rubber off the tires, they cause flat spots on the tires, and the values are usually so much shorter than for the wet pavement stops that only a few tests were run on each of the surfaces for which dry pavement stops are reported. The coefficients of friction in Table I are computed values based on the stopping distances obtained in the tests on each surface.

For new-tread tires on wet surfaces, the coefficients of friction at 30 m.p.h. should be 0.5 or more to provide a reasonable margin of safety, especially when braking. The stopping distance when braking from 30 m.p.h. for a coefficient of friction of 0.5 is 60 ft. The stopping distance on the dry pavements averaged 36 ft. which is well below the 60-ft. value recommended as the minimum for wet pavements. In the wet pavement tests, the only surface which measured up to the above standard was the concrete pavement in Marshall County with an average coefficient of friction of 0.51 and a stopping distance from 30 m.p.h. of 59 ft. The coefficients of friction for the wet bituminous surfaces ranged from 0.27 to 0.46 and the corresponding stopping distances ranged from 111 to 65 ft. While these surfaces do not measure up to the above recommended standard for safety in the braking tests, the only surface which these tests indicated as dangerously slippery is the surface in Benton County for which the coefficient of friction was found to average 0.27. This value is only slightly higher than the coefficient of friction on packed snow for which the skidding accident rate has been found to be approximately 20 times greater than on dry pavements.

The most surprising results in this study were the high percentages of asphalt found in the samples taken from the various bituminous

TABLE I. RESULTS OF BRAKING TESTS ON P.C. CONCRETE AND BITUMINOUS TREATED P.C. CONCRETE ROAD SURFACES IN IOWA.

Type	Year Built	Cover Aggregate Amount lb. per sq. yd.	Cover Aggregate Type	Asphalt Amount gal. per sq. yd.	Asphalt Type	Bitumen Content When Tested percent	County	Route No.	Test Condition	Initial Speed mph	Braking Distance ft.	Coefficient of Friction
Concrete	1922						Marshall	30	Dry	30	36	0.83
Concrete	1922						Marshall	30	Wet	30	59	0.51
Bit. Tr.	1945	30	3/8 gravel	0.3	RC 5	8.6	Polk	64	Dry	30	37	0.81
Bit. Tr.	1945	30	3/8 gravel	0.3	RC 5	8.6	Polk	64	Wet	30	86	0.35
Bit. Tr.	1945	30	3/8 gravel	0.3	RC 5	8.5	Marshall	30	Dry	30	36	0.83
Bit. Tr.	1945	30	3/8 gravel	0.3	RC 5	8.5	Marshall	30	Wet	30	73	0.41
Bit. Tr.	1946	25	sand	0.25	RC 5	13.1	Marshall	30	Wet	30	81	0.37
Bit. Tr.	1945	20	sand	0.2	RC 5	21.6	Benton	30	Wet	30	70	0.43
Bit. Tr.	1941	60	½ stone	0.6	RC 4	10.8	Benton	30	Wet	30	83	0.36
Bit. Tr.	1941	60	½ stone	0.6	RC 4	14.9	Benton	30	Wet	30	111	0.27
Bit. Tr.	1945	25	sand	0.25	RC 5	13.5	Tama	30	Wet	30	65	0.46
Bit. Tr.	1946	25	sand	0.25	RC 5	12.7	Marshall	30	Wet	30	81	0.37

surfaces tested, with values ranging from 8.5 to 21.6 percent. If the specified amounts of asphalt and cover material had all been retained on the surface, the asphalt content should have been about 7 percent. It is apparent that a large amount of cover material was whipped off the surface by traffic probably because the concrete pavement was so hard that it was difficult to imbed the aggregate in the asphalt and hold it there. The excess asphalt present on the surfaces tested contributes to bleeding which was observed in varying amounts on all of the bituminous treated surfaces. While the excess asphalt provided a waterproof treatment and thus protected the surface against further cracking and scaling, it also was responsible for the slippery when wet condition as determined in these exploratory tests.

The wide range in asphalt content obtained in the laboratory tests of samples taken from bituminous treated sections of concrete pavements, clearly indicates the difficulty of providing the correct proportions of asphalt and aggregate by this method of asphalt surface treatment on a hard surface such as concrete. For maximum stability as determined by laboratory stability tests, and to eliminate the tendency for bleeding, the asphalt content for bituminous mixtures using sand or gravel or stone chips should be less than 7 percent and the experience in Iowa has been that it is practically impossible to hold to this low asphalt content by the surface treatment method. Unless the asphalt content can be held to the low value required to prevent bleeding or flushing of asphalt to the surface, the surface is certain to be slippery when wet. This has been the result in every investigation of skid resistance of bituminous surfaces with which I am familiar and was again clearly indicated in the results of tests by Shelburne and Sheppe for surfaces 30 and 31 for which they reported a noticeable excess of asphalt flushed to the surface, stopping distances of 224 ft. and 254.5 ft. from 40 m.p.h., and a record of a number of skidding accidents on these surfaces.

In view of the difficulty of providing a satisfactory seal coat on concrete pavements by the surface treatment method, the Iowa State Highway Commission has decided to discontinue this method of treatment for resurfacing old concrete pavements and now plans to use the asphaltic concrete type of construction with a combined thickness for binder and surface course of 3 in. The asphaltic concrete will consist of a dense-graded hot plant-mixed material which will permit accurate control of the asphalt content and which will provide high stability and high skid resistance. Since the mixture is dense-graded, it should have good water-proofing properties to protect the old concrete and for this reason a seal coat should not be necessary. The elimination of the seal coat will be an important factor in preventing the large variations in the asphalt content and especially the high percentages of asphalt which are responsible for the slippery when wet surface condition. The plant-mix method should assure accurate control of the asphalt content. This control and the use of a well graded mixture of hard sharp cubical-shaped (instead of soft flat elongated shaped) aggregates, should assure many years of high stability and high resistance to skidding for the wet pavement condition.

In previous studies at Iowa State College[3], the asphaltic concrete surfaces of the type described above provided the highest resistance to skidding of all the types of surfaces tested. Shelburne and Sheppe report similar results for the plant-mixed asphaltic surfaces on which they ran braking tests. This provides some convincing arguments justifying the construction of the hot plant-mixed asphaltic concrete surfacing on old concrete pavements instead of the relatively light asphalt surface treatments recently used in Iowa and in many other states.

Before bringing this discussion to a close, I would like to emphasize again that the recent braking tests in Iowa were of an exploratory type and were not nearly as complete as the Virginia tests. For this reason it was not possible to make as complete and as satisfactory an analysis and interpretation of the results of the tests as Shelburne and Sheppe could make of the results of the Virginia tests. A far better measure of the skid resistance properties of road surfaces can be obtained if the braking tests are run for the surfaces in the wet condition at four speeds ranging from 10 to 40 m.p.h. using both new tread and smooth tread tires, than if the tests are run at only one speed and with only new tread tires. With braking tests at four speeds, curves may be plotted which indicate significant trends in the coefficients of friction as the speed is increased. In my opinion the most hazardous feature of a road surface which is slippery when wet is the marked decrease in the coefficient of friction (especially for smooth tread tires) as the speed of the vehicle is increased. Slippery surfaces may have fairly high coefficients of friction when wet at slow speeds in the range of 5 to 15 m.p.h. but at 40 m.p.h. the coefficients of slippery surfaces will frequently reach the low values of 0.30 to 0.20 or about the same as the coefficients obtained in the braking tests on packed snow. It is this characteristic which makes these surfaces so hazardous when wet because it should be realized that as the speed of the vehicle is increased the frictional requirements for steering and for braking in-

[3] R. A. Moyer, "Skidding Characteristics of Road Surfaces" *Proceedings*, Highway Research Board, 13:123-168. 1934.

crease approximately as the square of the speed whereas the friction which may be developed by the tires and the slippery road surface actually decreases as the vehicle speed is increased until a speed is reached where control of the vehicle is lost, resulting too frequently in a serious skidding accident.

The report by Shelburne and Sheppe provides an excellent guide which various state and city highway departments may follow in developing their own testing program to measure the skid resistance of slippery when wet road surfaces and to measure also the skid resistance obtained after various methods and types of skid resistance treatments have been used to correct the slippery when wet surface condition. Their report shows that the slippery condition can be corrected at a moderate cost, and in my opinion the advantages gained in the reduction of accidents caused by the slippery condition far outweigh the cost of correcting the slippery condition, if this work is carried out on the basis of a carefully conducted testing program of the type used in Virginia.

MR. SHELBURNE, *Closure:* We are indeed greatful for Professor Moyer's generous remarks. His advice concerning the equipment and procedures for conducting the tests was very helpful. The Virginia tests for the most part were conducted on high-type pavements where the surface composition and texture are practically constant for the particular section involved. While a few tests were performed on surface treatments such as Professor Moyer describes, it is believed that for most road-mix applications a considerable range in texture and composition is encountered.

In view of the Iowa policy of discontinuing the use of thin surface treatment applications in favor of a 3-in. binder and surface course of asphaltic concrete it might be well to cite similar experiences in Virginia. In 1937, the Virginia Department discontinued the use of relatively thin bituminous surface treatments on concrete pavements. Since that time, where surfacing of concrete pavements is done it has become standard practice to use plant-mixed materials consisting of about 170 lb. of binder and 130 lb. per sq. yd. of either a dense-graded asphaltic concrete or a sand asphalt wearing course. Experience has shown that these thicker plant-mix courses, where the gradation and qualities of material are rigidly controlled, have performed very satisfactorily. As shown by the paper, these surfaces also have relatively short stopping distances even when wet and are therefore much more desirable from a safety standpoint. Road roughness measurements on these plant-mix bituminous concrete surfaces also indicate that these pavements have excellent riding qualities.

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Research Report 6-B

SURFACE DRAINAGE OF HIGHWAYS

COMMITTEE REPORT AND THREE PAPERS

PRESENTED AT THE TWENTY-SEVENTH ANNUAL MEETING

1947

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PROGRESS REPORT, COMMITTEE ON SURFACE DRAINAGE OF HIGHWAYS

December 1947

C. F. IZZARD, *Chairman*

The third annual meeting of the Committee on Surface Drainage of Highways was held on December 1, 1947 with nine members present and three others reporting by letter. The major part of the meeting was devoted to discussing the Drainage Guide prepared by the Ohio Department of Highways and to reports on progress of research projects in which the Committee is interested.

Ohio Drainage Guide - The Ohio Drainage Guide is part of a Plan Preparation Manual and is intended primarily as a tool to facilitate hydraulic design of culverts and small open channels. It does not deal with waterways for bridge structures. The Guide includes 23 pages of text and examples demonstrating how to use the 24 charts, four of which deal with estimating peak rates of runoff, 12 with flow in open channels, and eight with culvert and pipe flow. The basic principle followed is that of designing culverts as hydraulic structures to carry a design discharge estimated in cubic feet per second. The charts enable determination of the depth of ponding at the entrance for any given discharge depending on the head loss through the structure measured from the tailwater elevation in the outlet channel or, in the case of barrels on steep grades, on the size and shape of the entrance.

The Committee reviewed the Ohio Drainage Guide thoroughly and recommends that other highway departments give consideration to developing similar guides for use by their design engineers. It is recognized that the weakest point in the method is the estimation of the peak rate of runoff in cubic feet per second but as more data become available from hydrologic studies of the Soil Conservation Service, U.S. Geological Survey and other agencies, this deficiency is gradually being removed. The runoff chart in the Ohio Guide is identical to that published in Part II, Roadside Development Report, Highway Research Board, April 1940, which has been found to be reasonably in line with the most recent runoff data available on small drainage areas. It is suggested that each State contact the Soil Conservation Service - Research, Washington 25, D.C., for latest available bulletins on peak rates of runoff on small drainage areas in that State.

With suitable charts at hand hydraulic analysis of a culvert can be made quickly for a range of discharge rates, and for alternate types of pipes, or boxes, either single or in multiple. The designer can then select the most economical layout meeting the limitations on cover over the culvert and for freeboard against overtopping the embankment or submerging valuable improvements on the upstream side of the roadway. The final selection of the design discharge may be made with reference to the probability of excessive damage if exceeded.

Handbook formulas which give only the required waterway area in square feet permit no analysis of the hydraulic effect of these many variables which ought to influence the final design.

The widespread application of hydraulics to design of culverts depends on reducing the involved mathematical formulas to relatively simple charts, giving results within practical limits of accuracy, which field engineers can use quickly and confidently. The Ohio Guide takes a long step in this direction but some of the committee members feel that the procedure is still too complicated for general use. Further simplification and clarification is undoubtedly possible. On the other hand it must be recognized that the field engineers ought to be given more training in the elements of flow in open channels and through culverts so that they may be able to apply the working charts more intelligently.

Hydrologic Data - The U.S. Geological Survey reported that a number of State highway departments are entering into cooperative agreements with the Survey, among these being Ohio, Missouri, and Georgia. These agreements vary in their provisions. Some provide for statistical analyses of existing streamflow records to provide data on magnitude and frequency of peak flows. Some provide for installation of additional gaging stations on small drainage areas each selected as being representative of a physiographic area of uniform runoff and climatic characteristics. In several States agreements provide for studies by the Survey regarding magnitude and frequency of floods and stages of such floods in the vicinity of proposed bridge sites for use by the State in designing the bridge waterways and highway gradelines. Special studies of thunderstorms are being made in Ohio. In New York State studies are being made of floods to obtain data for use by the Department of Public Works in contesting claims by property owners for damages caused by such floods.

The Committee invites attention to Bulletin No. 7 of the Ohio Water Resources Board entitled "Floods in Ohio--Magnitude and Frequency". This bulletin, compiled in cooperation with the U.S. Geological Survey, includes tabulation of peak flows at 44 gaging stations, from which are plotted graphs of the recurrence intervals of these floods. This type of graph enables good estimates of the magnitude of probable floods for frequencies not greatly in excess of the period of record for the stream gaged. By comparing floods of a given frequency on drainage areas having similar physiographic and climatic conditions but varying in size, it is usually possible to make fairly good estimates of probable floods on streams which are not gaged located in the same region.

Rainfall intensity-frequency duration data compiled by Yarnell and published in Miscellaneous Publication No. 204 of the U.S. Department of Agriculture included only storms through 1933. Since that time many additional first order weather stations have been installed and 14 more years of record obtained from the original stations. The Committee believes that the Weather Bureau ought to analyse the data now available and publish a new bulletin on intensity-frequency relations. Data of this kind are particularly valuable in the design of storm drains for urban highways and airports.

Some runoff data are being obtained by installation of crest stage recorders, a simple device which leaves ground cork particles adhering to a staff, placed in a vertical pipe, at the maximum elevation reached by the flood waters.

An observer records the maximum stage and resets the gage for the next flood. These gages are inexpensive, and if supplemented by current meter measurements to establish stage-discharge curves, can yield valuable information on flood discharges. Used in pairs or in series on a carefully selected reach they enable slope-area determinations of discharge.

The committee noted that a series of Regional Hydrologic Conferences is being planned by the Subcommittee on Hydrologic Data of the Federal Inter-Agency River Basin Committee for the purpose of compiling recommendations from all interested Federal agencies as to additional hydrologic measuring stations needed. The Public Roads Administration will cooperate with the State highway departments in preparing listings of stream-gaging stations which would be useful in connection with future road and bridge construction.

Stormwater Drainage for Urban Highways - A Subcommittee on Stormwater Drainage for Urban Highways has been formed and is preparing a list of problems on which research is believed to be needed.

Experimental work on inlet capacities is in progress at the University of Illinois, and at the University of Minnesota, as reported elsewhere in this bulletin. Research at the University of Illinois will also include statistical analysis of data for about 15 recording rainfall gages in the vicinity of Chicago to provide intensity-frequency duration curves and related information for use in design of stormwater drainage on express highways.

Hydraulic Research on Culverts - A fundamental investigation of the hydraulics of culverts is underway at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota as a cooperative project of the Minnesota Department of Highways and the Public Roads Administration. The first phase of this investigation is a thorough search of engineering literature to learn what has been done previously. An annotated bibliography covering about 100 of the more important articles is in preparation. The second phase of the project is the construction of a tilting channel in which model culverts may be installed for testing on various gradients. The plans for this apparatus have been approved and construction is expected to start soon. The third phase will be measurement of flow through model culverts with various approach and outlet channel conditions simulating the situations commonly encountered in the field. One of the initial objectives will be to develop entrance and outlet sections for pipe culverts which will operate efficiently over a wide range of flows, decreasing the headwater elevation for peak flows, and minimizing the erosion at both inlet and outlet. The resulting designs will be modified as necessary to facilitate mass production by precasting, or prefabrication, thereby eliminating the need for cast-in-place concrete headwalls and securing greater economy in first cost as well as lowered maintenance costs.

The hydraulic model tests at the St. Anthony Falls Hydraulic Laboratory will also enable further experimental verification of the theory of flow through short tubes as developed by Dr. Keulegan. The analytical study by Dr. Keulegan, abstracted in this bulletin, will be of great interest to research engineers since it reveals clearly the inadequacies of the empirical formulas previously developed from limited experimental data.

Proposed Research on Underscour at Bridge Piers and Abutments - The committee endorses the proposal made by the Joint Committee on Floods of the American Society of Civil Engineers for a fundamental investigation of the mechanics of scour around bridge piers and abutments. In many sections of the country it is necessary to build bridge foundations in alluvial stream beds. At flood stages deep scour occurs around these obstructions placed in the stream and occasionally a pier or abutment will fail by undermining. A more fundamental understanding of the forces involved in the scour phenomena, particularly the effect of local eddies, will make it possible to design substructures which will be safe against scour without excessive cost for either construction or maintenance.

(Editors Note: Since the committee meeting a research project has been financed by the Iowa State Highway Commission and the Public Roads Administration for model investigations of scour around bridge piers. The work will be done by the Iowa Institute of Hydraulic Research of the University of Iowa at Iowa City, under the direction of Dr. Hunter Rouse.)

The committee is gratified with the widespread interest now being shown on surface drainage of highways. Substantial progress is being made on the research problems endorsed by the committee. No new objectives were set forth, it being felt that efforts for the next year should be devoted to support of projects underway.

The committee will welcome suggestions as to research needed in the field of highway drainage.

DESCRIPTION OF APPARATUS AND PROCEDURE FOR TESTING FLOW IN GUTTERS AND STORM DRAIN INLETS

JOHN C. GUILLOU, *University of Illinois*

The Highway Drainage Research Project was undertaken by the University of Illinois at the request of the State of Illinois Division of Highways and the Public Roads Administration. Design engineers in the Illinois Highway Department have long realized that the data for design of highway drainage facilities is far from adequate. The State Division of Highways, faced with the task of building hundreds of miles of express highways upon which traffic stoppage, due to high or ponded water would be extremely costly, decided to undertake a study and to accumulate data for the proper design of drainage facilities for these high-speed roadways.

This project financed jointly by the State of Illinois and the Public Roads Administration, provides for fundamental research to develop improved designs of the various pertinent hydraulic features of such a high-speed roadway. The actual investigation will be carried out by the University of Illinois under an agreement between the University and State of Illinois, Division of Highways. The purpose of the study is "to compile information and to make investigations and tests relating to hydraulics and hydrology as involved in highway design, construction and maintenance with the objective of improving efficiency, ecomony and safety."

This objective directs that the studies be not only analytical, but that to a large extent they involve the construction and testing of hydraulic models of proposed designs and the development of new designs. The features of greatest concern to the Highway Department are, flow in gutters, flow through gutter inlets of various designs and flow through a collection system to the point where the collection system enters the main drain. In addition to this laboratory work, an analytical study is to be made of hydrology as it affects run-off on the highway project. It will include the study of storm paths and patterns relative to the drainage area shape, including a comprehensive review and analysis of existing rainfall data. A model reproducing any given rainfall hydrograph will be used in the laboratory in connection with these hydrologic studies.

The superhighway project facing the Illinois State Highway Department in the future is the Congress Street Superhighway, officially known as the West Route, in the city of Chicago. By directing all immediate laboratory study to the Congress Street Superhighway and obtaining results before the actual final design is completed on the project, it will be possible to use the Congress Street roadway as a 'pilot' project. An extensive study can then be made of such problems as driver behavior and psychology and other variables not readily adaptable to either laboratory or purely analytical study.

The Congress Street project is of the divided highway type with each directional roadway consisting of four 12-ft. lanes for high-speed traffic and one emergency parking lane. Each directional roadway is symetrical about its center line except that the emergency parking lane is provided on the right side of the roadway. The emergency parking lane is separated from the high-speed roadway by a mountable-type curb. The purpose of this curb and gutter is to channelize traffic and remove light precipitation run-off from the roadway without its first passing over the emergency parking lane. On the extreme right hand side of the emergency parking lane is a large circular section gutter which is designed to carry the bulk of the run-off to the collection system. On the other, or left side of the directional roadway is a barrier-type curb and gutter which carries the water from that half of the roadway to the collecting system. The collection system then carries the water from the three inlet boxes to a junction and thence to the main drain.

Because of the broad scope of the laboratory investigations, several types of hydraulic models will be employed in the study. First a 1:3 model of a portion of the Congress Street project will be built. This model will extend from the center line of the roadway to the outside edge of the toe-of-slope gutter in width, and in length will reproduce a prototype distance of 140 feet. By varying the amount of water supplied to the toe-of-slope gutter at the up-stream end of the model any gutter inlet spacing may be simulated. That is, the model will always represent the down-stream 140 feet of the inlet spacing distance. This 1:3 model will be used initially to study such design features of the Congress Street Superhighway as velocity distribution in gutters, efficiency of inlet gratings, sheet flow across the highway slab and a study of the intersection pattern of the sheet flow with the gutter flow. The model will be of the adjustable slope type capable of longitudinal slopes varying between zero and six percent and transverse slopes varying between zero and ten percent.

The model itself will rest upon a trussed model support frame which in turn will be supported by hydraulic jacks at alternate panel points. The model is of simple design, to facilitate remodeling and consists essentially of 2-by 10-in. transverse templates with a plywood surface. It has been decided to use molded Lucite sheets for the gutter portions of the model to facilitate observation in the velocity distribution phase of the work. The rest of the model area, representing the roadway lanes, will be covered with 3/8-in. thick Marine plywood. The plywood will be treated so as to obtain the proper roughness for hydraulic similitude. The lucite is naturally of the required roughness.

The water supply for this model will be pumped from a below floor level reservoir, or chase, into a constant head tank and then to one of three outlets, first it may flow to the up-stream end of the model and be introduced as gutter flow, as from an up-stream pavement area which is not a part of the model. It will then flow down either the mountable-curb gutter or the toe-of-slope gutter to the inlet basins and thence return to the chase. Second, the water may leave the constant head tank in one of two horizontal pipes whose purpose it is to simulate, in one case, the rainfall on the highway slab, and in the other case, the run-off from the cut slope to the side of the prototype roadway. Both of these pipes are horizontal and directly above, in case one, the center line of the roadway and in case two, the toe-of-slope gutter. Flow will take place through orifices in the sides of the pipe. The orifices will be about six inches on center for the entire length of the model. The water leaving an orifice will impinge upon a sheet of plastic screen material which will distribute the flow evenly as it flows down the sheet, and finally will release the water at the bottom edge of the screen to the model itself. Between the distribution pipes and the constant head tank the water will flow through valves which are operated by a pair of Selsyn motors so that any desired rainfall hydrograph may be reproduced in the model. By using continuous outflow measuring devices on the discharge lines from the catch basins it will be possible to accurately measure the dentention period or time lag which occurs between the change in rainfall intensity on the highway slab and the discharge rate at the catch basin outlets.

The second form of model which is to be employed in this investigation is of a large scale, probably 1:1½ or even 1:1. This model, actually three models in one, will consist of three gutters and catch basins complete with collection system. The only gutter sections that will be built on this model will be those necessary to accurately reproduce the entrance and tailwater condition at the inlet itself. This model, too, will be of the adjustable slope type. However, it will be subjected to a longitudinal slope change of only zero to four percent and will not be capable of any change in transverse slope. The gutters of this model will be constructed of light weight aggregate concrete to ease the change of slope problem. The grating inlet bars, catch basins and the junction boxes will all be made of Lucite of varying thicknesses. This again is done to facilitate the observation of flow characteristics.

This model is expected to yield data for the more efficient design of catch basins, junction boxes and grates, data for the elimination, or at least great reduction in the amount, of air entrained by the water in the catch basin. Finally a check test on the proper size and location of collecting system pipes which drain the catch basins or inlet boxes will be made. The piping in this model also, will be made of Lucite material artificially roughened for hydraulic similitude. In this study, too, a system of measuring devices will be used to show the time lag between the various points of inflow and outflow from the model. Design features developed in this model study will be reproduced at 1:3 scale and will then be inserted in the 1:3 model of the roadway itself. There they will be subjected to tests on the integrated structure to determine what effect any particular change may have on the general problem. By adhering to this procedure the 1:3 model will always be available for tests and demonstrations of the latest designs.

The third type of model to be employed in this study is not, strictly speaking, a hydraulic model. It is designed in accordance with the laws of hydraulic similitude but rather than use water for the testing medium a Bentonite solution will be used. Bentonite is actually a colloidal clay, which, when in proper suspension and subjected to stress, exhibits a remarkable birefringent characteristic. That is, when subjected to circularly polarized light, those portions of the suspension under different stresses show different colors. The difference in the stress and, therefore, the color, is caused by the existence of an acceleration gradient through the liquid at the point under question. This acceleration, of course, is indicative of the change in velocity. Therefore, the color is a direct measure of the velocity at any point. The light supplied to this model is passed through a polaroid screen and then a quarter wave plate. This quarter wave plate causes the light leaving the quarter wave plate to be circularly polarized. The circularly polarized light encountering the different refractive stream-lines in the model suspension is transformed in such a way that the net result when observed through proper polaroid plates is a chromatic velocity picture.

This model is expected to yield data depicting exactly what flow conditions do exist between adjacent bars of the inlet grating. It will also show exactly what the flow condition is in the catch basin itself. From this data a much more efficient inlet grating and catch basin design may be developed. The importance of the hydraulics of the jet of water flowing between the bars of the inlet grate cannot be over emphasized when considering air entrainment in the catch basin and the discharge pipe. Air entrainment is probably one of the major problems in the proper design of catch basins and inlet boxes. With the aid of the Bentonite model the design of catch basins is expected to be greatly simplified.

The space provided for this project is on the campus of the University of Illinois, in Urbana. The main laboratory is on the first floor of the Sanitary Engineering Building and contains approximately 1500 square feet. It is in this room that all the 1:3 model studies will be carried on. Another laboratory, containing about 1800 square feet of floor space is being readied for construction of the large scale model tests. This new laboratory will also contain facilities for testing component parts of the project, without it being necessary to use either the 1:3 model, or the large scale model of the catch basins and collecting system. This arrangement will make it possible to study numerous design features simultaneously, and thus avoid costly delay while awaiting completion of a test already in progress.

The above studies are being carried out by the Engineering Experiment Station at the University of Illinois. Dean M. L. Enger is director of the Engineering Experiment Station. The personnel involved in this particular study is composed of students and members of the Civil Engineering staff in the College of Engineering. Professor W. C. Huntington is head of the Civil Engineering Department and is in charge of all Civil Engineering research. Professor J. J. Doland is in specific charge of the Highway Drainage Research Project as well as other hydraulic engineering research. Mr. John C. Guillou is in charge of the model tests and other laboratory studies. Several research graduate assistants are employed and work half time on the project. It is expected that the preliminary tests on the Congress Street Superhighway will be completed by May of 1948. The large scale model tests should begin by mid February, 1948 and the Bentonite model should be in operation by mid April, 1948.

THEORY OF FLOW THROUGH SHORT TUBES WITH SMOOTH AND CORRUGATED SURFACES AND WITH SQUARE EDGED ENTRANCES

GARBIS H. KEULEGAN, *National Bureau of Standards*

The University of Iowa tests on the flow of water through culverts, made some twenty years ago, are well known for the extensiveness of the measurements undertaken. The main purpose of the investigation was the establishment of the formulas of discharge for culverts. The investigation proposed the relations

$$Q = \frac{A\sqrt{2gH}}{\sqrt{1 + 0.31D^{0.5} + \frac{0.026L}{D^{1.2}}}} \qquad (1)$$

and

$$Q = \frac{A\sqrt{2gH}}{\sqrt{1 + 0.16D^{0.8} + \frac{0.106L}{D^{1.2}}}} \qquad (2)$$

as the formulas of discharge for concrete and corrugated metal pipes, respectively, the pipes having square-edged entrances and flowing full. In these expressions Q is the discharge, A the cross-sectional area of the pipe, H the difference in the elevations of the water surfaces at the two ends of the pipe, L the length, and D the diameter. Quantities are measured in feet and second units. Although the experiments were made with culverts of small length and size, the investigations suppose that the formula are equally valid for culverts of any length and size.

That these formulas faithfully represent the discharges actually met with in the tests of the particular culverts employed there can be no doubt. On the other hand, the validity of the formulas can certainly be questioned for culverts of greater length and size. It is the particular structure of the formulas proposed which invites criticism. Under the radical sign of the denominators of these formulas are found three terms which in their order are associated with the loss of kinetic energy at the exit end, the loss at entrance, and the loss throughout the entire length of the tube. The criticisms to be made are in regard to the forms of the last two terms. First, the dimensions of these terms are not correct. Second, the particular exponents of the diameters in these terms imply that both the entrance loss and the so-called Manning's n for the entire culvert length vary with the culvert size. With the increase of diameter, these two quantities increase. There is, now, no hydrodynamical basis for this implication to be true and, therefore, the apparent contradictions involving the formula proposed require an examination.

In dealing with a short tube, it is better to divide it into three segments. See Figure 1. There is first the entrance segment where the losses occur in the surface of discontinuity of the gradually vanishing vena contracta. Here the losses occur in the body of water and the frictional shear at the wall is negligible. According to this view the loss is independent of the size of tube or the character of the surface. We shall denote the length of the segment by L_0. Secondly, there is the boundary layer segment, i.e. the segment in

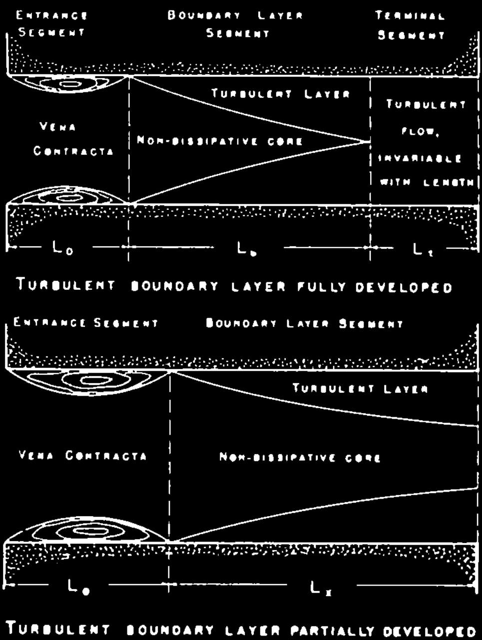

Figure 1. The Characteristic Flow Segments in a Short Pipe

which the turbulent boundary layer develops. At the upstream end of the segment, the thickness of the boundary layer is negligible and at the downstream end, when the tube is of sufficient length to assure the full development of the layer, the thickness equals the radius of the pipe. The local friction at the wall is large where the thickness of the layer is small. In the boundary layer segment, wall friction commences with a large value and is gradually reduced to a limiting value with distance. The central core outside of the boundary layer is non-dissipative and there the velocities and the pressures conform to the law of Bernoulli. For the full development of the layer a distance L_b will be required. Finally, if the length of the pipe exceeds $L_o + L_b$, there is the third segment, the terminal segment, where the velocity pattern does not change with distance along the pipe axis. We shall denote the length of the terminal segment by L_t. Now if the pipe is not sufficiently long, the length of the segment in which the turbulent layer develops will be less than L_b, and the corresponding length in that case may be denoted by L_x, where $L_x < L_b$.

THE THEORETICAL FORM OF DISCHARGE FORMULAS

Suppose that a tube of diameter D and length L connects two reservoirs. Let H be the difference in the elevations of the water surface in the two reservoirs. The Weisbach formula corresponding to the case is

$$H = (1 + f_e \frac{L}{D}) \frac{V^2}{2g} \qquad (3)$$

and the discharge will be given by

$$Q = \frac{A\sqrt{2gH}}{\sqrt{1 + f_e \frac{L}{D}}} \qquad (4)$$

Here, in these two expressions, f_e is the effective resistance coefficient. The proper evaluation of f_e requires that the losses are first individually determined for the three segments above mentioned.

Consider the case for which $L < L_o + L_b$. Denoting the loss in the entrance segment by H_o and in the boundary layer by H_b,

$$H = H_o + H_b + \frac{V^2}{2g}$$

In terms of the velocity head

$$H_o = f_o \frac{L_o}{D} \cdot \frac{V^2}{2g}$$

and

$$H_b = f_b \frac{L_x}{D} \cdot \frac{V^2}{2g}$$

where f_o is the coefficient of friction in the entrance segment and f_b is the coefficient of friction in the boundary layer segment. Thus

$$H = (1 + f_o \frac{L_o}{D} + f_b \frac{L_x}{D}) \frac{V^2}{2g} \qquad (5)$$

or comparing with equation 3,

$$f_e = f_o \frac{L_o}{L} + f_b \frac{L_x}{L} \qquad (6)$$

Consider next the case for which $L > L_o + L_b$, that is when

$$L = L_o + L_b + L_t$$

We now add to the right hand member of equation (5) the term H_t,

$$H_t = f \frac{L_t}{D} \frac{V^2}{2g}$$

where f is the coefficient of resistance in the terminal segment. Comparing the resultant expression of H with equation (3), it is seen that

$$f_e = f_o \frac{L_o}{L} + f_b \frac{L_b}{L} + f \frac{L_t}{L} \qquad (7)$$

Entering the expressions for the effective coefficient of friction from equation (6) or equation (7) into equation (4), we have for the discharges,

$$Q = \frac{A\sqrt{2gH}}{\sqrt{1 + f_o \frac{L_o}{D} + f_b \frac{L_x}{D}}} \qquad (8)$$

when $L = L_o + L_x$, and

$$Q = \frac{A\sqrt{2gH}}{\sqrt{1 + f_o \frac{L_o}{D} + f_b \frac{L_b}{D} + f \frac{L_t}{D}}} \qquad (9)$$

when $L = L_o + L_b + L_t$.

These are the theoretical discharge formulas. Comparing the general form of these expressions with the forms of the Iowa formulas, equations (1) and (2), the following facts are revealed. First, there is no unique formula which will express the discharge through short and long tubes with equal correctness. Secondly, by introducing the concept of the varying resistance in the laminar boundary layer, one dispenses with the idea of variable Manning's n. Actually, it is supposed that the variation of resistance in the critical initial part of a short tube is brought about not by the variation of the surface characteristics, since these are invariable, but by the gradual change in the thickness of the developing turbulent boundary layer.

THE EVALUATION OF THE EFFECTIVE COEFFICIENT OF RESISTANCE

To evaluate the effective coefficient of resistance f_e, the quantities f_o, L_o, L_b or L_x, f_b and f must be first known.

The examination of entrance losses in the Iowa experiments shows that for concrete and corrugated metal pipes with square-edged entrances, the entrance length L_o equals 3.5D and the coefficient of friction is

$$f_o = 0.152 \qquad (10)$$

As regards f_b our analysis for the turbulent boundary layer shows that this quantity is proportional to f, the coefficient of resistance in the terminal segment, the factor of proportionality being a function of L_x/L_b and the relative roughness R/k, where R is the radius of the pipe and k is a roughness factor. We have deduced from various published data on concrete pipes that for concrete $k = 0.005$ ft. on the average. This really is the equivalent sand grain size for concrete. If a smooth surface is covered with sand of this size, the flow distribution will be the same as that of concrete under the condition that mean flows and diameters are equal. In corrugated metal pipe k may be identified as the corrugation depth, that is the vertical distance between the crests and troughs of the corrugations. In the corrugated metal of Iowa tests $k = 1$ inch.

Following the above explanations, the effective frictional coefficient in the developing boundary layer may be written mathematically

$$\frac{f_b}{f} = N\left(\frac{L_x}{L_b}, \frac{R}{k}\right) \tag{11}$$

This functional relationship was investigated for the surfaces that can be interpreted as sand covered surfaces and also for corrugated metal surfaces of the type used in the Iowa tests. In these corrugated metal surfaces the corrugations are nearly sinusoidal in shape with the ratio of corrugation depths to the wave length being $k/l = 0.1875$. Since the universal law of velocities for concrete and corrugated metal surfaces are not the same, the function N was determined separately for the two surfaces. The result of computations are given in Tables 1 and 2, where f_b/f values are tabulated against L_x/L_b and R/k.

TABLE 1

EFFECTIVE COEFFICIENT OF RESISTANCE OF THE TURBULENT BOUNDARY LAYER IN SAND-COVERED PIPES

($k = 0.005$ ft. for concrete)

R/k	10	18	40	100	250	500	1000
L_x/L_b	f_b/f						
0.1	1.538	1.676	1.852	1.961	2.000	2.000	2.001
0.2	1.424	1.517	1.613	1.679	1.692	1.687	1.680
0.3	1.351	1.418	1.490	1.536	1.538	1.534	1.528
0.4	1.296	1.349	1.404	1.446	1.443	1.438	1.428
0.5	1.257	1.300	1.349	1.377	1.377	1.371	1.361
0.6	1.224	1.261	1.302	1.325	1.325	1.320	1.311
0.7	1.198	1.229	1.264	1.284	1.284	1.280	1.272
0.8	1.175	1.203	1.234	1.252	1.252	1.248	1.241
0.9	1.157	1.181	1.209	1.227	1.227	1.222	1.215
1.0	1.141	1.163	1.189	1.203	1.203	1.199	1.193

TABLE 2

EFFECTIVE COEFFICIENT OF RESISTANCE OF THE TURBULENT BOUNDARY LAYER IN CORRUGATED PIPES

($k = 0.0417$ ft. $k/l = 0.1875$)

R/k	10	18	40	100	250	500	1000
L_x/L_b	f_b/f						
0.1	1.798	1.902	2.112	2.209	2.254	2.306	2.277
0.2	1.598	1.660	1.785	1.852	1.868	1.858	1.834
0.3	1.480	1.530	1.621	1.663	1.659	1.655	1.633
0.4	1.397	1.440	1.506	1.547	1.539	1.531	1.533
0.5	1.339	1.371	1.428	1.457	1.451	1.446	1.432
0.6	1.294	1.319	1.369	1.392	1.388	1.374	1.370
0.7	1.258	1.280	1.322	1.342	1.338	1.335	1.324
0.8	1.228	1.249	1.285	1.300	1.299	1.296	1.286
0.9	1.204	1.225	1.259	1.271	1.266	1.265	1.254
1.0	1.184	1.205	1.229	1.244	1.240	1.238	1.228

As regards L_b, the length necessary for the full development of turbulent boundary layer, our analysis has shown that it depends on the type of surface, the diameter of pipe and the relative roughness. The dependence of L_b/R on R/k for the two surfaces can be read from the two curves in Figure 2.

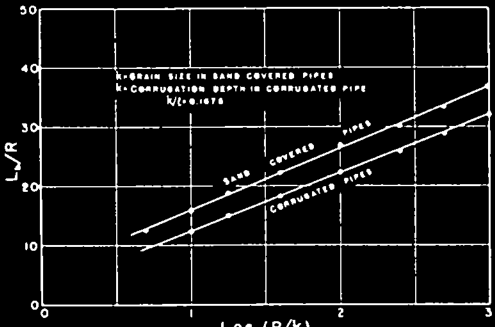

Figure 2. Length of Fully Developed Boundary Layer in Rough Pipes

Finally, for the coefficient in the terminal segment, f, we have

$$\sqrt{\frac{8}{f}} = 4.75 - 2.5\frac{k}{R} + 5.75\log\left(1 + \frac{k}{R}\right)^{2} \cdot \log\left(1 + \frac{R}{k}\right) \qquad (12)$$

for a pipe surface the hydrodynamic action of which can be simulated by a surface covered with sand of grain size k. As remarked above, for concrete k may be taken as equalling 0.005 ft. For corrugated metal of the type used in the Iowa tests

$$\sqrt{\frac{8}{f}} = 2.11 - 2.5\frac{k}{R} + 5.75\log\left(1 + \frac{k}{R}\right)^{2} \cdot \log\left(1 + \frac{R}{k}\right) \qquad (13)$$

The forms of these friction formulas differ slightly from the logarithmic forms ordinarily given in the texts on hydraulics. The difference results from the convention adopted for measuring distances in the pipe transverse sections. We measure wall distances from the top of the sand asperities or from the crests of corrugations. Accordingly, the inner diameters are measured between the crests of the roughness asperities.

IOWA EXPERIMENTS

The discharges observed in the Iowa tests for the corrugated metal and concrete pipes with square-cornered entrances are represented by small circles in Figures 3 and 4. The curves shown in the figures are theoretically obtained, using the method of computation explained above. The agreement between theory and experiment may be considered as satisfactory.

Due to the satisfactory agreement between theory and observation noted for the discharges obtained in the small size culverts of the Iowa tests, like computations were also made for eight-foot diameter corrugated metal and concrete pipe and of varying lengths up to three hundred feet. The results of the computations were compared with the similar results of the Iowa discharge formulas. It was noted that the Iowa formulas gave smaller values than the theoretical results and that the disparity in the two results increased with increasing lengths of pipe. Furthermore, the increase was larger for the corrugated metal pipe than for the concrete.

BASIS OF ANALYSIS

The analysis undertaken to determine the quantities of the boundary layer proved to be very lengthy. The details of the computations will not be given here. For the purpose of orientation, however, it may be helpful to make a few remarks about velocity distributions in general.

The momentum law, together with

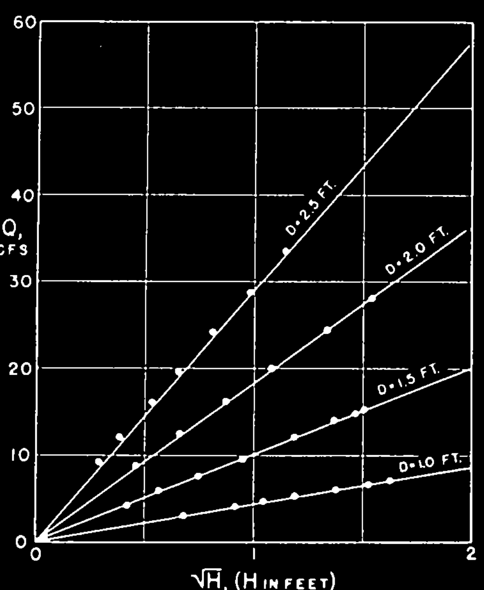

Figure 3. Iowa Test Discharges in Concrete Pipes with Square-cornered Entrances

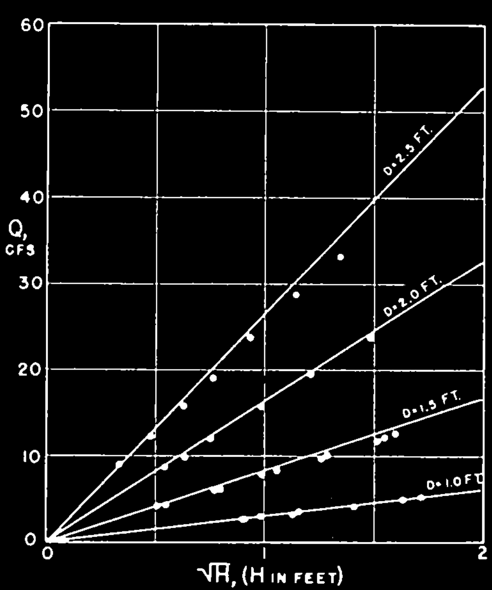

Figure 4. Iowa Test Discharges in Corrugated Pipes with Square-cornered Entrances

the supposition of a non-dissipative central core and the condition of continuity enables one to determine the variation of boundary layer thickness and wall shear with distance for a given relative roughness R/k. The computations are carried out by supposing that for each type of surface a universal velocity law exists of the type

$$\frac{u}{U_v} = a + 5.75 \log(1 + \frac{y}{k}) \qquad (14)$$

where u is the velocity at the point y, y being measured from the crests of corrugations and the tops of the hypothetical asperities of sand. The quantity U_v is the so-called shear velocity given by the relation

$$U_v = \sqrt{\frac{\tau_o}{\rho}} \qquad (15)$$

where τ_o is the shear at the wall and ρ is the density of the fluid. The quantity a is a numerical constant characteristic of a given surface. The formula is very simple and states that the velocity at a point depends solely on the shear at the wall and the distance from the wall. The extent and the limits of the fluid have no relation whatever to the velocities. Thus, the velocity law is independent of the dimensions of the central core or of the size of the pipe.

In our computations we have assumed that for a concrete surface we may take a = 5.85 and k = .005 ft. These suppositions put the concrete surface in the same class as the sand covered pipe surfaces of the Nikuradze experiments. That is, for concrete

$$\frac{u}{U_v} = 5.05 + 5.75 \log(1 + \frac{y}{k}) \qquad (16)$$

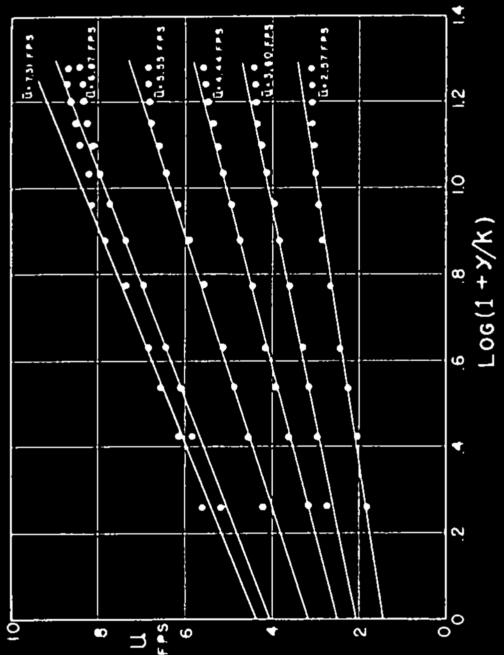

Figure 5. Velocity Distribution in Corrugated Metal Pipe; Iowa Tests

The law of velocities for a corrugated surface does not appear to have been known previously. A determination was made using a series of velocity distribution from the Iowa tests. The method of analysis in short is as follows. For any type of surface we may write

$$u = A + B \log(1 + \frac{y}{k})$$

where $A = aU_v$

and $B = 5.75\, U_v$

Thus, if u is plotted against (1 + y/k) and the plot gives a straight line, the intersection point A at the ordinate axis and the inclination B determine U_v and a. The value u/U_v may now be formed and may be plotted against log (1 + y/k). For the corrugated metal pipes the application of this method gives first the data of Figure 5 and Figure 6. According to the latter a = 8.5, and thus the law of velocities for the corrugated metal pipe is

$$\frac{u}{U_v} = 8.5 + 5.75 \log(1 + \frac{y}{k}) \qquad (17)$$

where k is corrugation depth and $k/_1$ = 0.1875.

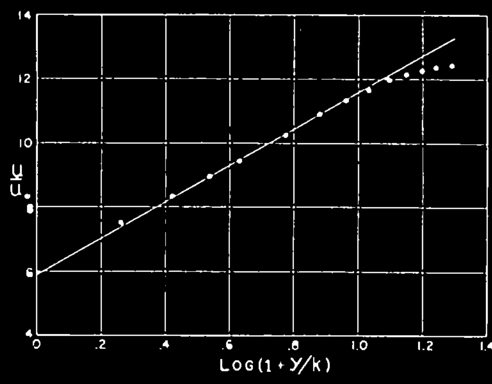

Figure 6. Universal Law of Velocities for Corrugated Metal Pipe; $k/_1$ = 0.1875

EXPERIMENTS ON FLOW THROUGH INLET GRATINGS FOR STREET GUTTERS

CURTIS L. LARSON, St. Anthony Falls Hydraulic Laboratory

In designing surface drainage facilities for streets and highways, the highway engineer has been handicapped by the general lack of data on the capacity of grate inlets. The limited data available indicates that for many of the grate inlets now in use, the capacities are quite low, particularly on moderate and steep grades. In addition, clogging of grate inlets with paper, leaves, and other debris continues to be a serious maintenance problem. For the purpose of alleviating these problems, an experimental investigation was undertaken at the St. Anthony Falls Hydraulic Laboratory of the University of Minnesota, under the sponsorship of the Minnesota Department of Highways.

A test gutter with a cross-slope of 20.6 to 1 and with a nearly vertical curb was constructed in the 36-in. tilting channel of the Laboratory. Near the end of the gutter, a test section was provided, in which full-scale inlets and curb openings of any shape could be installed. Tests of various grate inlets were conducted at several slopes, using a wide range of discharges at each slope. In all tests the entire flow was introduced at the upper end of the gutter. Measurements were taken of the depth and discharge in the gutter, and of the quantity of water passing over and around the inlet, which was termed "carryover." The portion of the flow intercepted by the inlet, referred to as "inlet capacity," was, of course, the difference between the gutter flow and the carryover. Tests were also made with simulated debris added to the flow.

In tests conducted at the North Carolina Engineering Experiment Station (*1*)[1], N.W. Conner found that deflecting slots in a gutter are self-cleaning when set at an angle of 45 deg. with the direction of flow. In an attempt to improve the self-cleaning ability of grate inlets, an experimental inlet was constructed with its bars and openings set at this angle. Tests were made of this inlet both with and without a curb opening. This experimental inlet was then improved by rounding the surface of each of its bars. Standard inlets tested included a Minnesota Highway Department inlet, which has openings parallel to the flow, and a city street department inlet, which has openings normal to the direction of flow.

Since the test gutter was considerably smoother than the average gutter, differences in roughness must be considered in applying the test results to grate inlets in actual gutters. In addition, one may wish to apply the data to inlets in various gutters having different degrees of roughness. For these reasons, the test results are not presented on the basis of slope alone, but rather on the basis of the quantity $\sqrt{s}/n$, in which s is the highway slope and n is the Manning roughness coefficient. This factor is a constant for any given gutter. Since this index is pro-

[1] Italicized figures in parentheses refer to the list of references at the end of paper.

portional to velocity for a given depth of flow, it will be referred to as the "velocity index." The four test slopes selected gave a range in velocity index from 6.6 to 17.2, resulting in super-critical flow within the entire range. This range includes gutters of ordinary roughness at slopes of 1 to 6 percent.

The data obtained in the capacity tests are presented in Figures 1 through 4, in the form of "rating" curves. In these curves, inlet

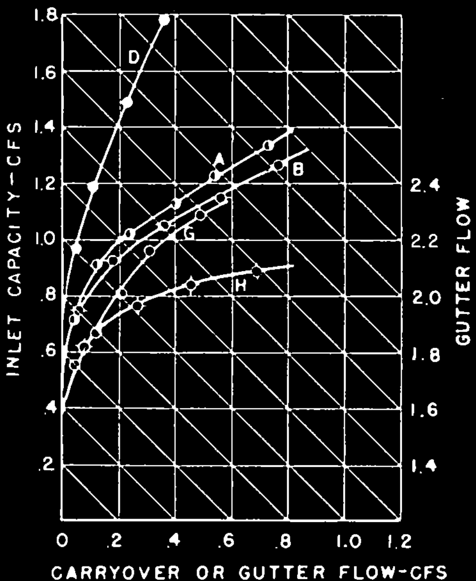

Figure 1. Rating Curves at Velocity Index of 17.2

capacities are plotted as ordinates, and carryovers as abscissas. For any point on these curves, the corresponding gutter discharge can also be determined directly by following the sloping lines to the carryover scale. The letter designations on the figures indicate the various inlets or inlet setups, as follows:

A. Experimental inlet, with curb opening
B. Experimental inlet, without curb opening
D. Improved experimental inlet
G. Highway Department inlet
H. City inlet

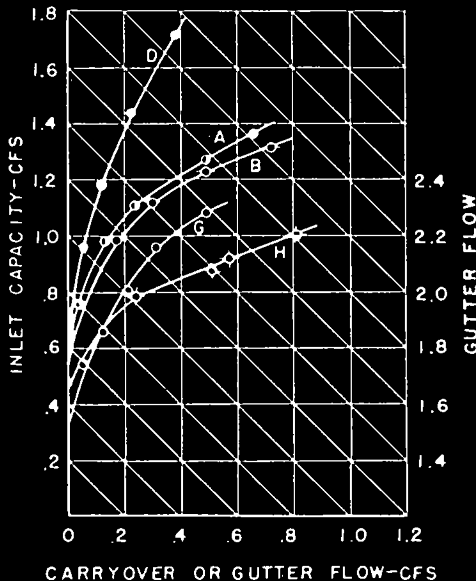

Figure 2. Rating Curves at Velocity Index of 14.0

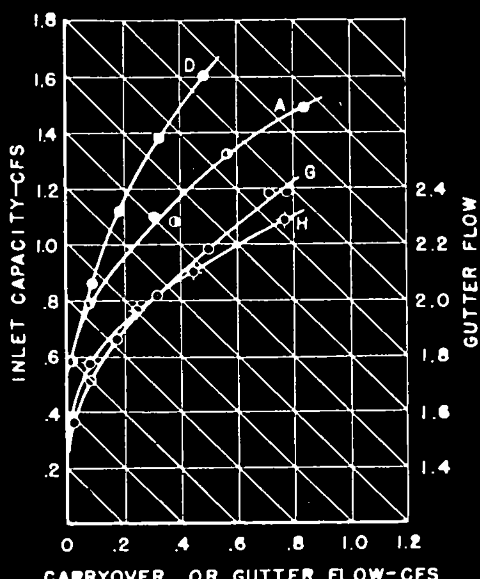

Figure 3. Rating Curves at Velocity Index of 9.8

Each of Figures 1-4 contains the data obtained at a certain test slope, and is therefore applicable only for a particular velocity index.

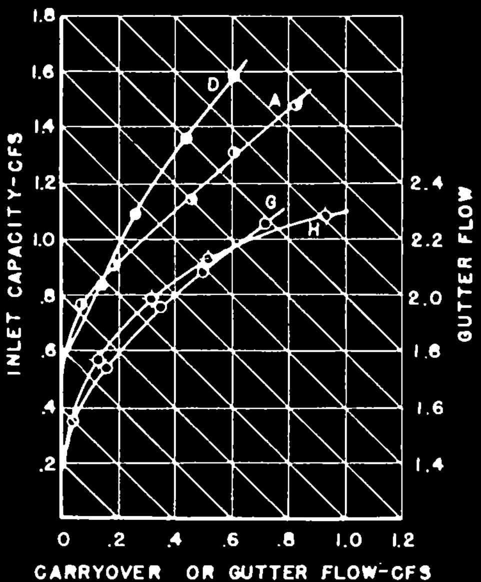

Figure 4. Rating Curves at Velocity Index of 6.6

Figures 5, 6, and 7 are plots of velocity indexes versus inlet capacities corresponding to several carryovers. For these carryovers then, one can determine the corresponding inlet capacity at any

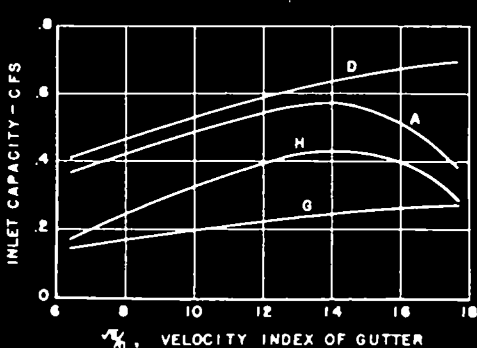

Figure 5. Inlet Capacities with No Carryover

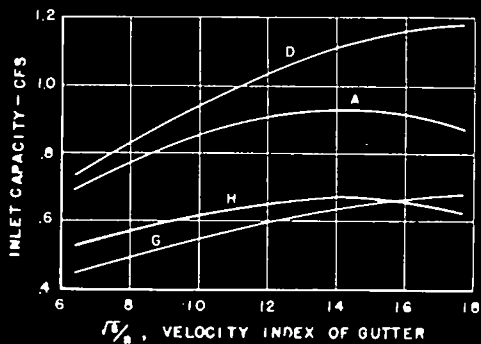

Figure 6. Inlet Capacities with Carryover of 0.10 cu ft per sec

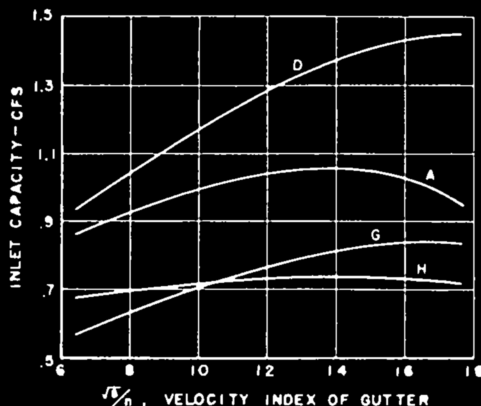

Figure 7. Inlet Capacities with Carryover of 0.20 cu ft per sec

velocity index within the range of the tests. These curves also indicate the manner in which the capacity of any of the inlets varies with slope.

Visual observations during the tests indicated that the data do not fully explain differences in behavior between the various inlets. To supplement the data, therefore, a number of photographs were taken of the inlets in operation. Figures 8 through 11 show Inlets A, D, G, and H operating with approximately the same gutter discharge.

RESULTS OF CAPACITY TESTS

Perhaps the most important fact developed by these tests is that the capacities of grate inlets can be greatly increased by permitting a small amount of carryover. This statement appears to apply to any grate inlet. The rating curves show that the capacities of most of the inlets tested are approximately doubled by allowing carryovers from 0.10 to 0.20 cu. ft. per sec. In the case of inlets in series, these small carryovers from inlet to inlet produce no ill effects other than a slight increase in the gutter flow, since carryover is not cumulative. Greater carryovers produce diminishing returns. Thus a carryover in the range of 0.10 to 0.20 cu. ft. per sec. appears to be the optimum for inlets in series in the ordinary case where the gutter discharge is a limiting factor.

The capacity test data show that the capacity of a grate inlet is affected both by the characteristics of the inlet and by the characteristics of the approach flow. Furthermore, variations in the nature of the approach flow produce varying and sometimes opposite effects upon inlet capacity, depending on the characteristics of the inlet. Of primary importance in determining inlet capacity are the following inlet characteristics: the width of the inlet, and the efficiency of the inlet openings.

The width of the inlet measured normal to the direction of flow, is an influential factor in that the carryover in almost every case is either partly or wholly composed of water which passes around the inlet. In other words, no inlet can be expected to intercept a large portion of the flow unless it extends well into the path of the flow. The importance of width can be seen from an inspection of the rating curves for Inlet D, the improved experimental inlet, and for Inlet G, the Highway Department inlet, both of which take water readily. Inlet D, being 24 in. in width, has a high rating curve, while Inlet G, which is 17 in. wide, has a low rating. Thus, it appears worthwhile to make grate inlets at least 24 in. wide for a gutter of this shape, and perhaps wider for highways with flatter crown slopes.

The efficiency of grate inlet openings was found to depend mainly on the effective length of the individual openings, which, in all cases, is measured in the direction of flow. The importance of this characteristic is well demonstrated in a general way by the test results. Since it has 1 3/16-in. transverse openings, the city inlet, Inlet H, permitted an appreciable portion of the flow to pass directly over the openings. The rating curve for this inlet therefore rises slowly. In the Highway Department inlet, Inlet G, 1 1/4-in. by 11-in. openings are placed parallel to the flow, making their effective length 11 in. The photographs show that these openings allowed no water to pass over the inlet, and a steeper rating curve was the result. The narrower width of Inlet G, however, caused its capacity to fall below that of Inlet H in the region of no carryover.

During tests of inlets with transverse bars, it was observed that only a thin sheet of water was di-

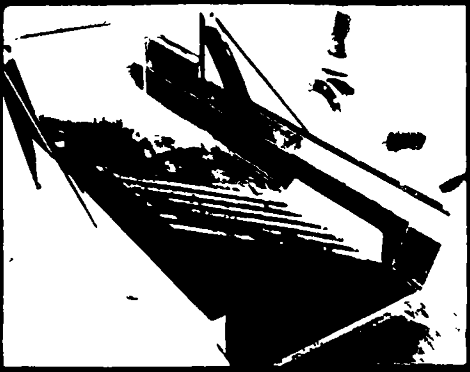

Figure 8. Experimental Inlet, $\sqrt{s}/n$ = 14.0, Q_G = 1.02, Q_I = 0.92, Q_C = 0.10 cu ft per sec

Figure 9. Improved Experimental Inlet, Series D, $\sqrt{s}/n$ = 14.0, Q_G = 1.05, Q_I = 0.98, Q_C = 0.07 cu ft per sec.

Figure 10. Highway Department Inlet, Series G, $\sqrt{s}/n$ = 14.0, Q_G = 1.00, Q_I = 0.81, Q_C = 0.19 cu ft per sec

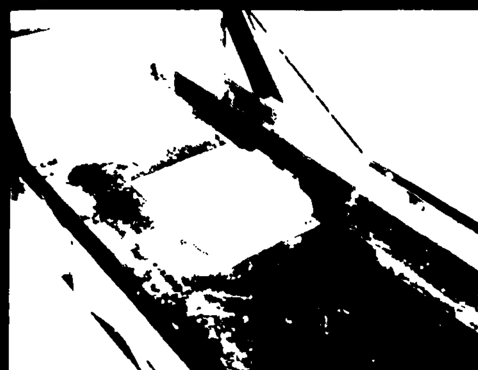

Figure 11. City Inlet, Series H, $\sqrt{s}/n$ = 14.0, Q_G = 1.00, Q_I = 0.77, Q_C = 0.23 cu ft per sec

Figure 12. Flow Over Ordinary and Improved Grate Bars

verted downward at the face of each bar. Theoretically, the thickness of this sheet of water varies as the square of the overfall distance (effective length of opening) for flow of a given velocity, if the path of the water crossing the opening is assumed to be that of a freely falling body. For this reason, it would appear highly desirable to increase the effective length of opening in any way possible.

In Series D, the length of the openings of the experimental inlet was increased by rounding the grate bar surfaces. The surface of each bar was rounded to conform approximately to the shape of a free overfall from its leading edge, as shown in Figure 12. In effect, this change moved the beginning point of each overfall from the trailing edge to the leading edge of the bar. Since the bar thickness was equal to the bar spacing, the overfall distance or effective length of opening was approximately doubled. Thus, the thickness t of the sheet of water diverted by each bar, as well as the capacity of each opening, was theoretically quadrupled. Although no measurements were made of the capacity of individual openings, the photographs demonstrate that this improvement actually is very effective. Figure 8 shows that six of the openings failed to intercept all of the water flowing over the original experimental inlet, while with the improved grate bars, Figure 9, almost the entire flow was intercepted by the first two openings. This simple improvement appears to be applicable to any inlet with transverse bars, and would result in little, if any, increase in the cost of casting this type of inlet.

The use of curb openings with grate inlets was found to produce little or no increase in capacity, depending on the efficiency of the inlet. The B series of tests was conducted with the experimental inlet as it was used in the A series, except that the curb opening was replaced by a section of curb. Comparison of the rating curves, Figures 1 and 2, shows that only a small percentage of the inlet capacity, less than 5 percent, can be credited to the curb openings. In this case, the curb opening intercepts some water which would otherwise flow over the inlet. In the case of inlets with more efficient openings, which permit no water to flow over the inlet, it is evident that a curb opening provides practically no increase in capacity, unless the inlets are affected by backwater.

The characteristics of the approach flow were also found to have a pronounced effect on the capacity of grate inlets. The tests showed that high velocities tend to decrease the capacity of an inlet by increasing the tendency for water to flow or spray over the openings. On the other hand, high velocities tend to increase the capacity of an inlet by concentrating a greater flow in a given width of gutter. Figures 5, 6, and 7 show that either of these opposing tendencies may be predominant, depending on the width of the inlet and the efficiency of the inlet openings. Within the range of the tests, these curves also show that the improved experimental inlet and the Highway Department inlet, which have efficient openings, operate with increasing capacity as the slope and velocity are increased. The original experimental inlet and the city inlet, which have less efficient openings, increase in capacity with increased velocity indexes up to approximately 14, but decrease in capacity for velocity indexes higher than 14.

DEBRIS TESTS

In order to have a quantitative basis for comparing the self-cleaning abilities of the various inlets, an arbitrary procedure for debris tests was adopted, using as debris pieces of paper 1 by 2 inches in size. Since no attempt was made to duplicate actual gutter debris, the results of these tests were not intended to indicate the percentage of actual debris which a given inlet will handle. However, the results are believed to serve as a basis for comparing the various inlets tested.

The original experimental inlet was found to pass only 20 to 30 percent of the test debris, and would therefore probably clog quite easily. It was hoped that this inlet would be self-cleaning as a result of the component of flow along the axis of each bar, but this component was not strong enough to remove the test debris. Rounding the bars of this inlet, however, permitted approximately 70 percent of the test debris to pass through the inlet openings.

Because its openings are parallel to the flow, the Highway Department inlet handled the test debris as easily as it did water, having a debris efficiency of about 95 percent. However, it should be noted that for larger debris, this inlet might clog as easily as any other. The city inlet, which is a rough casting with the bars normal to the flow, passed only 17 percent of the test debris.

Of the inlets tested then, only the Highway Department inlet, which has openings parallel to the flow, can be considered highly efficient in passing this type of debris. The debris tests also indicate that improving the hydraulic efficiency of inlet openings increases the ability of the inlet to pass debris.

APPLICATION OF RESULTS

For a given set of design conditions, the results of this investigation can be used to determine the required spacing for inlets of any of the types tested. Moreover, the data can be used to predict the operating capacity of any individual inlet, either under the design conditions or under other circumstances, such as rainfall intensities higher or lower than the design intensity, or clogging of one or more inlets in a series.

If one of these inlets is to be used in a location where no carryover is permissible, it is necessary merely to select the inlet capacity which will give no carryover at the appropriate velocity index. In such a location, however, the inlet may be affected by backwater from intersecting streets or from changes in grade, in which case the capacity of the inlet will probably be greater than the capacity found in the tests.

In a series of inlets where some carryover is permissible, a considerably greater inlet capacity, and correspondingly, a greater inlet spacing can be used. In designing such a series of inlets, the "design" or "normal" inlet capacity, corresponding to a suitable carryover, can be selected from Figures 5 through 7, or from rating curves. For a series of uniformly spaced inlets, it can be shown readily that, if succeeding inlets operate with equal carryover, the flow intercepted by each inlet will be equal to the runoff per inlet. Thus, the required inlet spacing can be found by equating the design capacity to the runoff per inlet, if the rate of runoff can be expressed in terms of the dimensions of the drainage area and the rainfall intensity. For the idealized case of a rainfall of uniform intensity for a period longer than the time of concentration, assuming no infiltration, the expression thus obtained for the inlet spacing L in feet is:

$$L = \frac{43{,}200 \; Q_I}{bI} \qquad (1)$$

in which Q_I is the design inlet capacity in cu. ft. per sec, b the width of street drained in feet, and I the rainfall intensity in inches per hour. The depth and width of flow in the gutter upstream of each inlet can then be computed if desired. The gutter flow Q_G, is given by:

$$Q_G = Q_I + Q_C \qquad (2)$$

where Q_C is the design carryover. For the gutter under consideration, Manning's formula may be applied to obtain the following depth discharge relation:

$$Q_G = 9.5 \frac{\sqrt{s}}{n} y^{8/3} \qquad (3)$$

in which y is the depth of flow in feet at the curb. If it is to be used repeatedly, this relation can be plotted as a family of curves for various values of $\sqrt{s}/n$, the velocity index. Since the cross-slope of the experimental gutter is 20.6 to 1, the maximum width of flow, w, is given by:

$$w = 20.6\ y \tag{4}$$

An example best illustrates the use of these data or similar data in a design problem. In a gutter of the same shape as the test gutter on a 3.5 percent grade, the roughness coefficient n is estimated to be 0.015. The velocity index is then 12.5. A rainfall having a uniform intensity of 5 in. per hr. is to be drained from a 24-ft. width of paved street or highway by inlets of Type A. Assuming that a carryover of 0.20 cu. ft. per sec. is permissible, it is seen from Figure 7 that, at a velocity index of 12.5, the corresponding inlet capacity is 1.05 cu. ft. per sec. The required inlet spacing can then be obtained by use of Equation (1):

$$L = \frac{43{,}200 \times 1.05}{24 \times 5} = 378 \text{ ft.}$$

The gutter flow just above each inlet is given by:

$$Q_G = 1.05 + 0.20 = 1.25 \text{ cu. ft. per sec.}$$

The depth of flow can be found by substitution in Equation (3):

$$y = \left[\frac{1.25}{9.5 \times 12.5}\right]^{3/8} = 0.18 \text{ ft.}$$

and the maximum width of flow is found to be:

$$w = 20.6 \times 0.18 = 3.7 \text{ ft.}$$

If it appears advisable to consider the effects of gutter storage and storms shorter than the time of concentration, the inlet spacing cannot be determined directly by an equation such as Equation (1). The actual gutter hydrograph can be determined, however, by a method originated by Horner and Jens[2]. This method was verified experimentally and developed further by Izzard[3]. Further development of this procedure is necessary to determine the effect of carryover on the gutter hydrograph.

A series of inlets possesses a valuable attribute in its ability to adjust its capacity to any rate of runoff within a considerable range. To demonstrate that each inlet in a series tends to operate at a capacity equal to the runoff per inlet, another example will be given. In a gutter having a velocity index of 14.0, ten of the improved experimental inlets, Type D, are spaced to receive 1.00 cu. ft. per sec. of runoff per inlet, which results in a normal carryover of 0.07 cu. ft. per sec. By some unusual circumstance, Inlet No. 5 becomes completely clogged. The gutter discharge is therefore considerably more than normal at Inlet No. 6, and is less than normal at the beginning of the series. The discharge intercepted by each inlet of the series can be determined, however, by use of the appropriate rating curve, as shown in Table 1.

Beginning at Inlet No. 1 of this series, the gutter discharge is 1.00 cu. ft. per sec., since there is no carryover from a preceding inlet. The rating curve for $\sqrt{s}/n$ = 14.0, Figure 2, shows that with this gutter flow, 0.94 cu. ft. per sec. is intercepted and 0.06 passes by the inlet as carryover. This carryover results in a gutter flow of 1.06 cu. ft. per sec. at Inlet No. 2 and the rating curve is referred to again to determine the flow intercepted and the carryover. This procedure may be followed on through the series. In this example, the normal inlet capacity, equal to the runoff per inlet, is reached at Inlet No. 3, and all succeeding inlets will normally operate at this capacity. Clogging of Inlet No. 5 upsets this equilibrium, since none of the flow is intercepted by this inlet. The

TABLE I

COMPUTATION OF INDIVIDUAL CAPACITIES OF TYPE D INLETS IN SERIES AT A VELOCITY INDEX OF 14.0

Inlet Number	Runoff	Condition	Q_G	Q_I	Q_C
1	1.00	Clean	1.00	0.94	0.06
2	"	"	1.06	0.99	0.07
3	"	"	1.07	1.00	0.07
4	"	"	1.07	1.00	0.07
5	"	Clogged	1.07	0	1.07
6	"	Clean	2.07	1.71	0.36
7	"	"	1.36	1.23	0.13
8	"	"	1.13	1.05	0.08
9	"	"	1.08	1.01	0.07
10	"	"	1.07	1.00	0.07

result is a carryover of 1.07 and a gutter flow of 2.07 cu. ft. per sec. to Inlet No. 6. This gutter flow, however, is quickly reduced at succeeding inlets, and normal inlet capacity is again reached at Inlet No. 10. This example shows that if the gutter flow at any inlet happens to be more or less than the normal amount for the series, the flow intercepted by succeeding inlets will increase or decrease, as the case may be, until the normal inlet capacity, equal to the runoff per inlet, is reached at some inlet downstream.

This investigation is limited chiefly by the fact that the data obtained are applicable only to inlets in gutters having cross sections identical to that of the test gutter, that is, with a uniform cross-slope of 20.6 to 1. However, it seems likely that many of the general findings of these experiments will apply, in greater or lesser degree, to grate inlets in gutters having other cross-slopes.

In planning the tests and preparing the data, the effects of lateral inflow on the flow conditions in an actual gutter were neglected, since the side inflow per foot of gutter will normally be only a fraction of a percent of the gutter flow near an inlet. Any resulting discrepancies would therefore be small, and would be reflected mainly in the velocity index scale, which in practice is subject to an error of several percent in the estimation of the roughness coefficient.

Of the standard and experimental inlets investigated, none is believed to represent the best solution to the requirements of capacity, self-cleaning ability, and economy in grate inlets. Nevertheless, the tests have developed considerable evidence of the relative importance of various inlet characteristics. It is possible that the best features of the test inlets can be combined to best satisfy these requirements, for a gutter of the shape used. Further tests are being planned for this purpose.

SUMMARY OF RESULTS

The results of this investigation are summarized briefly in the following conclusions, which are applicable to a continuous gutter having a cross-slope of approximately 20 to 1, and a velocity index within the range of these tests.

1. The capacity of a grate inlet can be greatly increased by allowing a small amount of carryover.

2. The capacity of a grate inlet is determined mainly by its width normal to the flow and by the efficiency of its openings.

3. The efficiency of grate inlet openings depends largely on the effective length of the openings in the direction of flow.

4. The capacity of inlets with transverse bars and openings can be increased substantially by rounding

the top surface of each bar.

5. In the normal range of application, inlets with efficient openings operate with increasing capacity as the slope of the gutter is increased.

6. Except where capacity is provided by ponding, curb openings are of little or no value in increasing the capacity of a grate inlet.

ACKNOWLEDGEMENTS

The writer wishes to acknowledge his indebtedness to Mr. A.W. Verharen, Hydraulic Engineer of the Minnesota Department of Highways, who suggested this investigation and who has followed its progress closely, and to Dr. Lorenz G. Straub, Director of the St. Anthony Falls Hydraulic Laboratory, under whose direction the experiments were conducted. For authority to publish these test results, the writer is indebted to the Minnesota Department of Highways, Mr. O.L. Kipp, Chief Engineer, and to the Director of the Laboratory.

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DISCUSSION

CARL F. IZZARD, *Highway Research Engineer, Public Roads Administration* - The paper by Mr. Larson is a valuable contribution to an understanding of the hydraulics of inlet gratings for street gutters. The reader will be glad to know that Mr. Larson has conducted tests on modifications of the inlets described and that a complete report will be published by the University.

In interpreting the data in this paper the effect of the cross-section of the approach gutter must not be overlooked. For example, the capacity of Inlet G, as reported, will be reduced nearly 30 percent if the transverse slope of the gutter is flattened to a 50 to 1 slope.

The capacity of Inlet G, or of any other grating having efficient openings, can be closely approximated by assuming that all the water flowing within the width of the grating will be intercepted, while the water flowing on the pavement beyond the outside edge of the grating is the "carryover" discharge. Hicks[1] made this assumption in 1944 and the data in Larson's paper may be used as verification.

The rating curve for Inlet G may be computed with a maximum difference of 3 percent in a range of gutter flow from 0.5 to 2.0 cubic feet per second using equations (3) and (4) to estimate depth and width of gutter flow and Hicks' flow distribution curve to estimate flow within the width of the grating. However, it is not necessary to use the latter curve as will be shown.

For gutters having a triangular cross-section, equation (3) can be generalized by making the numerical coefficient equal to 0.468 z, where z is the ratio of width of flow to depth of flow (20.6 in Larson's experiments). The factor 0.468 is taken directly from equation (11) in Reference 3. The general equation is then

$$Q_G = 0.468\, z \frac{\sqrt{s}}{n}\, y^{8/3} \qquad (5)$$

Substituting 20.6 for z gives a numerical coefficient of 9.64 instead

[1]Hicks, W.I. "Runoff Computations and Drainage Inlets for Parkways in Los Angeles," *Proceedings*, Highway Research Board, Vol. 24 pp 138-147 (1944).

of 9.5 as in equation (3) because Reference 3 ignores friction on the curb face in order to simplify the derivation, the error having no practical significance in working with shallow depths. Also, because of this fact, the same equation may be used to estimate the carryover discharge by taking y as the depth at the outside edge of the grating. Assume Q_G = 1.0 cubic feet per second and take $\sqrt{s}/n$ = 14 as in Figure 2. Then equation (5) reduces to $Q_G = (9.64 \times 14)y^{8/3} = 135\ y^{8/3}$ from which y = 0.159 feet for the assumed gutter flow. The depth will be 1.42/20.6 = 0.069 feet less at the outer edge of the grating or 0.090 feet. Substituting this depth in the same equation $Q_C = 135\ (1/11.1)^{8/3} = 0.22$ cubic feet per second. (Note: reciprocals are easier to work with than small decimals; use tables of fractional powers in hydraulic handbook to facilitate computation.). Then from equation (2), Q_I = 1.0 - 0.22 = 0.78 cubic feet per second which agrees closely with the observed value of 0.80 cubic feet per second read from Fig. 2.

From equation (5) it follows that the width of flow for a given discharge in a triangular gutter on a given grade will vary as $(z)^{5/8}$ while the depth varies inversely as $(z)^{3/8}$. Thus when the transverse slope is flattened to 50 to 1 making z = 50, the width of flow in the gutter is $(50/20.6)^{5/8}$ = 1.74 times that in Larson's tests. The depth for z = 50 would be $(20.6/50)^{3/8}$ = 0.717 times that in Larson's tests.

The trend in drainage design on urban highways is to space inlets so that the width of flow in the gutter for a design rainfall intensity will not exceed an arbitrary amount for frequent storms. The design rainfall intensity, for example, may be the average intensity for a duration of 20 minutes and a frequency of one or two years. The intense rainfall of shorter duration obscures vision so that traffic is forced to move slowly or even stop, but with adequate inlets the roadway will clear rapidly within a few minutes after the intense rainfall ceases. Thus the traffic delay will probably not be serious, particularly since these occurrences will be infrequent. The storm sewer sizes should be based on, for example, a 10-year storm for durations corresponding to the respective times of concentration so that water will not be ponded on the roadway because of inadequate outlet capacity except for the extreme storms for which it is not considered economical to design.

Rating curves similar to those in Figure 2 may be computed for any given width of inlet with any value of z in equation (5), assuming the inlet to have efficient openings. From such curves computed for various grades inlet capacity curves for different rates of carryover, similar to Figures 5, 6 and 7, can be drawn. These will show, as Larson ably demonstrates, that a small amount of carryover greatly increases the inlet capacity. Since the spacing of inlets by equation (1) is directly proportional to the inlet capacity, the spacing also increases with the amount of carryover, thereby reducing the initial cost. Charts may also be drawn for gutter capacity in relation to grade of roadway for various widths of flow. These can be used to check inlet spacing by the criterion established for width of flow in the design storm, which may be found on the flatter grades.

A common practice on express highways is to provide a 2-foot gutter on a one-inch per foot slope outside the edge of the 12-foot traffic lane. Equation (5) can be used to compute capacity of this type of cross-section as follows. Assume steeper slope to be extended, compute discharge for a given depth on one percent grade and subtract discharge computed for depth at point where slope changes. Then, for the latter depth, compute discharge on the flatter slope. Add this discharge to that computed for gutter to obtain total discharge. Repeat computations for other depths. Then plot discharge against depth or width with grade as parameter (discharge on other grades will vary with square root of grade), or plot discharge against slope with depth or width as parameter. Since the inlet grating is usually the same width as the steep portion of the gutter, the discharge computed for the latter will also be inlet capacity if inlet can be assumed as having efficient openings. This type of cross-section enables carrying a given discharge with much less encroachment on the traffic lane in comparison to a section with the curb at the edge of the traffic lane.

In applying equation (5) to estimating inlet capacities for gutter sections differing from that used by Larson, study must be given to his experimental data in judging whether or not a proposed grating has efficient openings which can be depended on to intercept all the flow over the grating. In general it appears that a grating with bars parallel to the approaching flow and a clear length of opening sufficient to permit the falling jet of water to clear the far end of the grating will have satisfactory characteristics. A length of opening in the direction of approach flow of about 18 inches is sufficient for maximum velocities likely to be encountered on express highways, based on a free-fall drop of 0.5 feet in the time required for the water to move the length of the opening. A length of 24 inches would provide some factor of safety to allow for debris accumulating on the downstream end of the bars. A greater length gives no increased capacity except when ponding occurs as at sag vertical curves.

In the past widely-spaced bars parallel to the curb have been frowned on because of the hazard of wheels on narrow-tired vehicles, such as buggies and bicycles, dropping through the openings. On limited access highways where there is little possibility of such traffic this objection doesn't apply, nor is it necessary to give consideration to high heels on women's shoes in determining the maximum width of opening. Where bicycle traffic may be encountered diagonal bars with rounded tops as in Inlet D may be used.

Attention is called to the fact that the increased capacity of Inlet D over Inlet G, which has bars parallel to the curb, is due almost entirely to the width of 24 inches within the range of velocity index tested. This can be proved by computing flow in a width of 24 inches as compared to 17 inches by the method previously illustrated. The length of opening, 11 inches, for Inlet G would begin to restrict capacity at greater depths and velocities of flow, but Inlet D may also fail to intercept all the flow in its width under similar conditions.

Gutter storage probably has no significant effect on required inlet capacity as used in equation (1) if the rainfall intensity used is the average for a duration of about 20 minutes which is the present trend of design practice as previously noted. This time is in excess of the time of concentration for most cases so that the outflow hydrograph at each inlet would have reached equilibrium with the inflow hydrograph for the drainage area.

Larson deserves great credit for developing his theory of the manner in which grating-type inlets in series on a continuous grade will adjust to the rate of runoff because of the characteristic of increased inlet capacity with increased carryover. By applying Larson's method for determining inlet spacing, satisfactory results can be obtained with fewer inlets than would be required for the assumption that each inlet on a continuous grade should intercept all the flow in the gutter.

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