HIGHWAY RESEARCH BOARD

Research Report 6-B

SURFACE DRAINAGE OF HIGHWAYS

REPORT OF COMMITTEE

AND

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1948

PRESENTED AT THE
TWENTY-SEVENTH ANNUAL MEETING

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HIGHWAY RESEARCH BOARD

DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH

NATIONAL RESEARCH COUNCIL

Washington 25, D. C. December, 1948

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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~~ includes 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

$$Q = \frac{A\sqrt{2gH}}{\sqrt{1 + 0.16D^{0.6} + \frac{0.106L}{D^{1.2}}}} \qquad (2)$$

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 culverte 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_o. Secondly, there is the boundary layer segment, i.e. the segment in

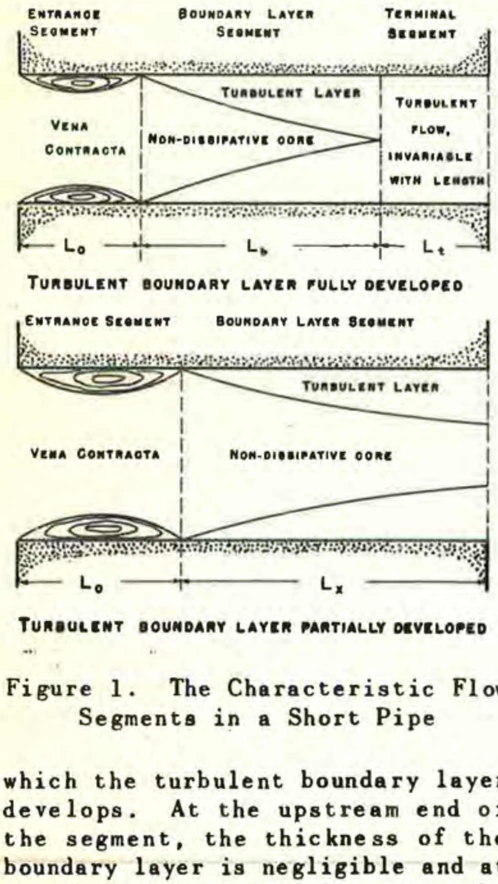

Figure 1. The Characteristic Flow Segments in a Short Pipe

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

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

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

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

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

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

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.

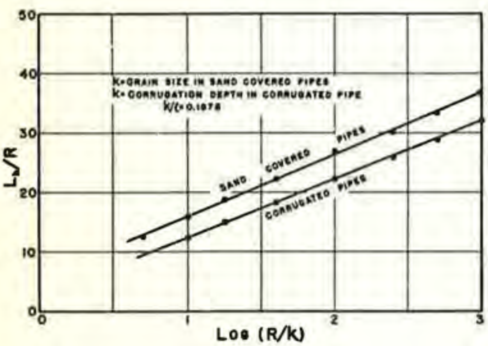

Figure 2. Length of Fully Developed Boundary Layer in Rough Pipes

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

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

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

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

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

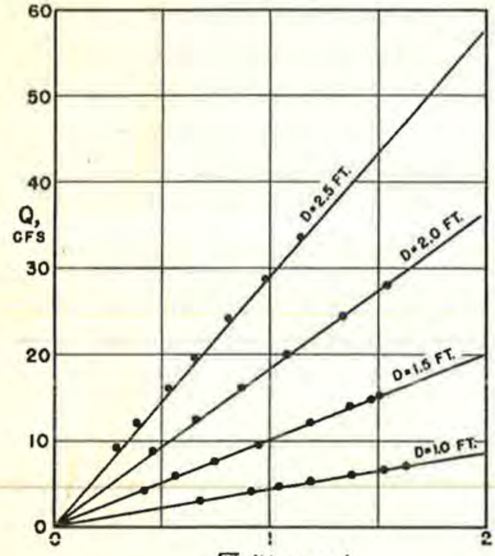

Figure 3. Iowa Test Discharges in Concrete Pipes with Square-cornered Entrances

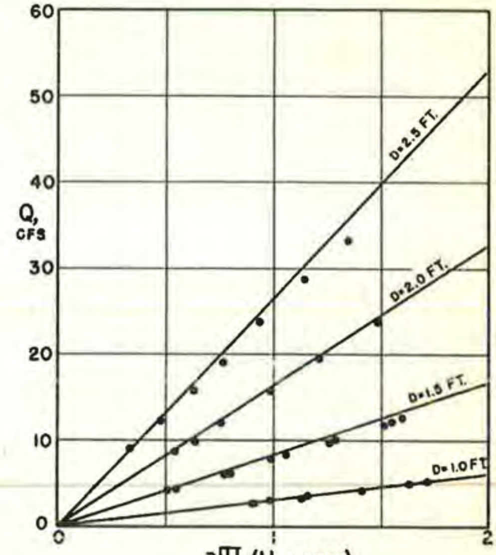

Figure 4. Iowa Test Discharges in Corrugated Pipes with Square-cornered Entrances

the supposition of a non-dissipative central core and the condition of continuity enabled ~~one to~~ THE determin~~e~~ATION OF 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

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

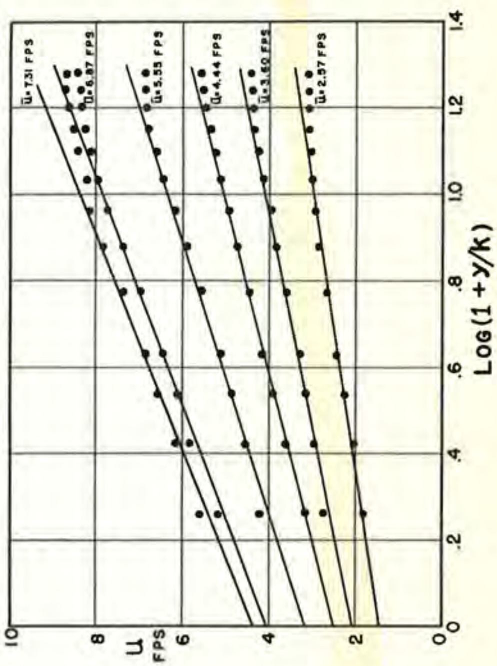

Figure 5. Velocity Distribution in Corrugated Metal Pipe; Iowa Tests

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 $k/_l$ = 0.1875.

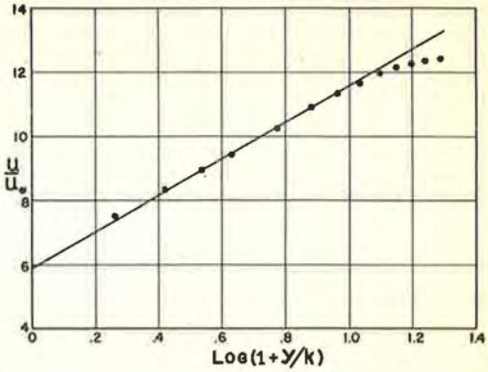

Figure 6. Universal Law of Velocities for Corrugated Metal Pipe; $k/_l$ = 0.1875

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

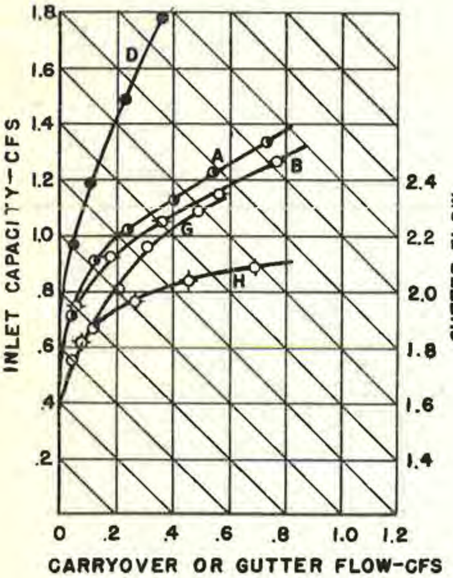

Figure 1. Rating Curves at Velocity Index of 17.2

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

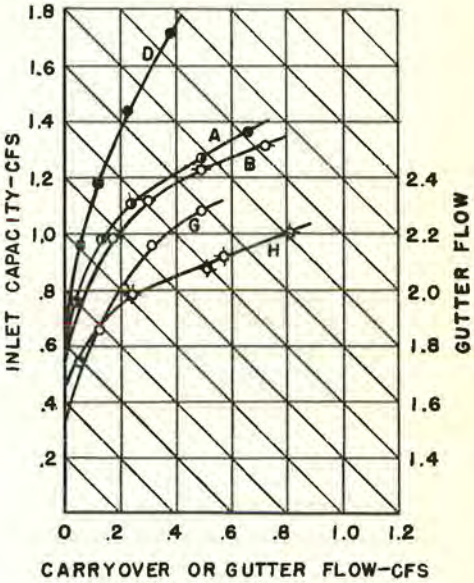

Figure 2. Rating Curves at Velocity Index of 14.0

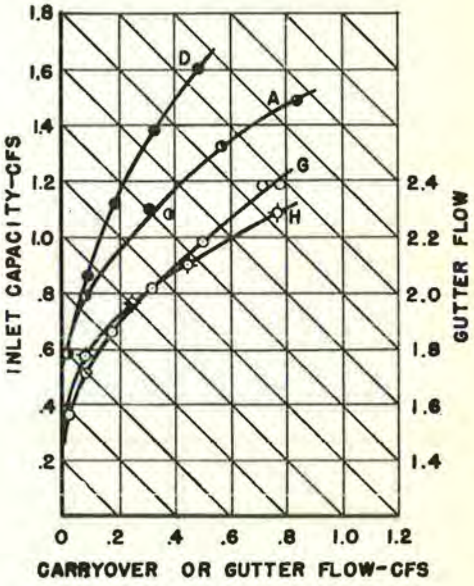

Figure 3. Rating Curves at Velocity Index of 9.8

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

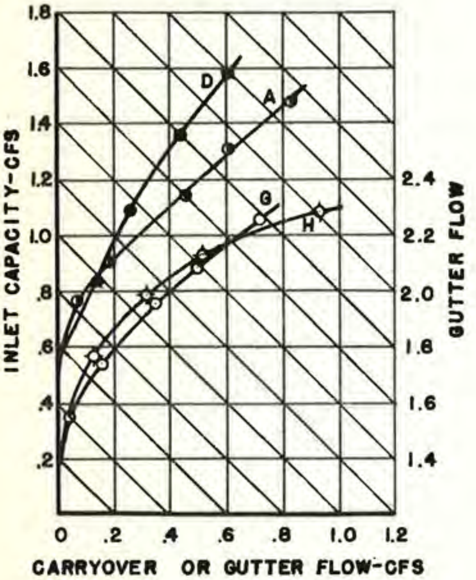

Figure 4. Rating Curves at Velocity Index of 6.6

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

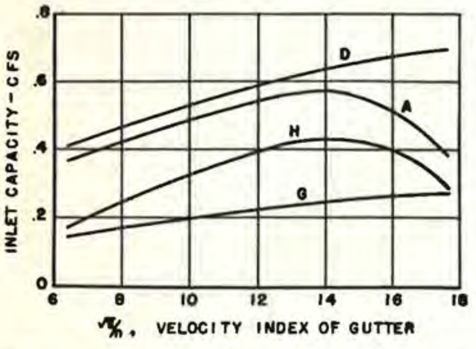

Figure 5. Inlet Capacities with No Carryover

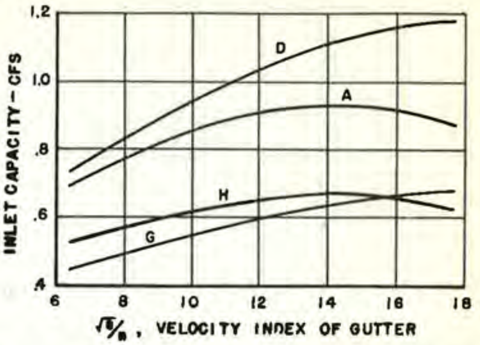

Figure 6. Inlet Capacities with Carryover of 0.10 cu ft per sec

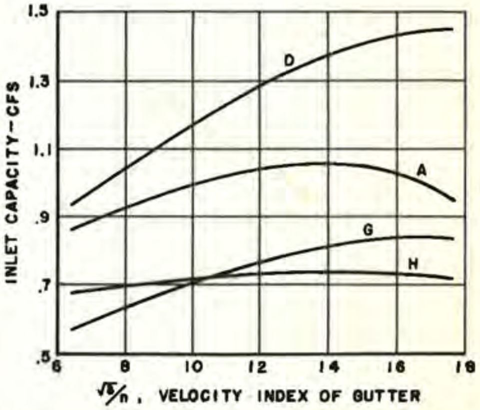

Figure 7. Inlet Capacities with Carryover of 0.20 cu ft per sec

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 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 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

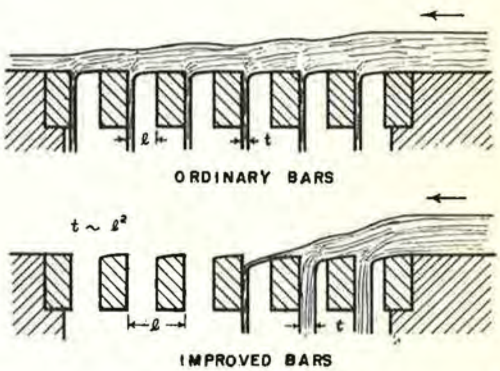

Figure 12. Flow Over Ordinary and Improved Grate Bars

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 in- 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:

$$w = 20.6\ y \qquad (4)$$

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.

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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

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

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 $\sqrt{s}/n \doteq 14$ 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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