ride-area warden to co-operate with similar wardens of other plants and business firms. Eventually every area will have ride wardens from every plant. These men can handle the work of correlating riders and will be given OCD certificates for their work.

We feel that the seriousness of this rubber shortage, even for men in war-production plants, is going to necessitate the continuance of the Gary War Transportation Committee's work until the supply of both natural crude and synthetic rubber is sufficient to maintain every war worker's car so that its owner can do his share in production of war material. If this is not done, and the rubber supply remains as critical as today, we can foresee the moving into city homes, within walking distance of our industrial plants, of hundreds of our industrial workers, or the additional alternative of employee housing projects provided by the industry and/or the government.

From our experience in Gary and knowledge of other cities' work, we can say that results can be had, satisfying results too, by the operation of these swap-ride programs. They are necessary. They are patriotic measures. They are the one and only method we can see to keep America rolling on rubber, so that America and our Allies may exert every effort to win this war and our eternal freedom.

BRIDGE AND CULVERT FLOW AREAS

F. William Greve, Professor of Hydraulic Engineering,
Purdue University

Captain Petty has asked me to review briefly some of the hydrological factors which influence the size of culverts and small bridge openings. No distinction will be made hereafter between these two types of waterway, and, for the sake of clarity, references to culverts will also be applicable to small bridge openings.

A culvert is a right-of-way. The flow through it differs fundamentally from the traffic on a highway in that hydraulic flow is subject to the laws of nature in contrast to the movement of vehicles, which is under the control of man. Congestion of human traffic on a road will, at the worst, cause inconvenience and doubtless much profanity; while at a culvert, congestion will produce ponding of water which, in turn, may cause not only the inconvenience of a flooded roadway, but also property damage incident to submerged adjacent lands, erosion of the bed and banks of the stream, and possible destruction of both culvert and highway.

Future vehicular traffic can be estimated from present trends. The maximal flood flow which may pass down the stream, however, lies buried in its entire past history, which, unfortunately, is not a matter of record. How often one hears statements made to the effect that distinct climatic changes are taking place within the relatively short span of a lifetime, and that rainstorms and stream flows are higher, lower, or both, than they have been in the past. Recorded data, on the other hand, give evidence that there has been no distinct climatic change in this country during approximately the last two centuries. Undoubtedly there has been some change in the frequency and intensity of minor floods, caused by man-made alterations in the vegetal cover of the earth's surface, construction of flood planes, and construction projects of assorted types. Major floods, on the other hand, will occur, man's work to the contrary, whenever an abnormal precipitation falls on ground which is frozen or is saturated.

It behooves engineers not to accept as fact the memories of the elder generation with regard to the record of streams. Almost everyone can remember the brooks and other streams of his boyhood fishing days. They have become magnified through the lapse of time, and the fish caught were giants, so it seems, compared to the pigmies of today. Illusions of youth furnish some of the spice of life and provide source material for good-natured bragging. And if one would preserve these illusions, let him shun the streams of his youth, and under no circumstances permit his technical judgment to be swayed by recollections.

Man's record of a stream is not necessarily and probably never a true reflection of its entire past behavior. The record is but a point on the line of time, which extends back for at least thousands of years. The assumption that a stream has gone through all its paces during the period of record leads to the application of the law of probability. This mathematical rule is based on the assumption that history does repeat itself. The difficulty of obtaining factual evidence owing to the scarcity of run-off data for culvert-size streams has already been stressed. On the other hand, usually at hand are records of trend, intensity, and volume of highway traffic upon which to base estimates of future capacities and speed. Hydraulic traffic, under the premise of no significant climatic changes, can have no trend. Yet it does have one other important characteristic, and that is frequency. Unfortunately, frequency is not periodic. So, presuming that the maximal flood which will occur once in a given period of time can be computed within reasonable limits of accuracy, the problem is still in a sense indeterminate because the time of arrival of the highest waters remains unpredictable. The destructive flood may arrive on the first day of the period, or the last day, or at any time in between.

Run-off

Factors affecting run-off from small watersheds include: (1) form, intensity, and duration of precipitation; (2) season of the year; (3) type of vegetal cover; (4) type of soil; (5) topography; and (6) percentage of the surface occupied by natural storage basins in the form of ponds and marshes. In the study of surface flow from large areas, it is necessary to include such additional hydrological factors as direction of the storm, size and shape of the watershed, seepage, evaporation, synchronization of tributary flow, etc.

There are several methods for the estimation of run-off, most of them of the rule-of-thumb, or empirical, order, and frequently quite suitable for the purpose at hand. Most highway and railway engineering staffs have a generous supply of such empirical formulas on tap, each one of the equations limited in application to a restricted region of the country. A brief description of the several methods follows:

1. Estimation of the flow by comparison with that of a similar stream in the same locality for which data have been obtained by measurement.
2. For a stream which is at all times confined within its banks, the cross-sectional area of the waterway corresponding to maximal peak flow can be approximated, which approximation, multiplied by a presumed velocity of flow equal to 10-ft./sec., will give as a product the probable flood discharge. Clues to the maximal height of the stream surface frequently can be detected in cornstalks and other debris lodged in trees along the banks.
3. A method often used employs the application of one of many empirical formulas of the general type, $Q = C\ R\ D$, where Q is the discharge in cubic feet per second, C is a variable coefficient dependent upon the physical characteristics of the watershed, R is the total rainfall in inches, and D is the area of the drainage basin in square miles. The accuracy of results will depend very largely upon how well the engineer estimates, or judges, the value of C. (It is, psychologically, bad practice for an engineer to admit that he ever guesses.) Probably no two engineers would evaluate C alike for any one watershed; and it is readily understood that values of C obtained from data established on one drainage basin should be applied with extreme caution to other watersheds in a different locality. The equations of the general empirical form mentioned are further handicapped by the fact that no factor is included to express the intensity of rainfall, which may be and often is of paramount importance.

Some of the better known formulas include Talbot's, Peck's, Myers', and the Tidewater. Talbot's formula is based on data collected from areas of less than 50,000 acres in the Mississippi Valley. The following examples illustrate the wide variation in results obtained by application of several of these formulas to two watersheds in California. Myer's formula applied to a watershed of 100 acres indicated a flood flow 2.5 times that computed with the Tidewater equation, while for an area of 100,000 acres, the maximal run-off calculated with the Peck formula was 8.5, 15, and 53 times, respectively, that computed by the Tidewater, Talbot, and Myer equations.

4. The so-called rational method for computing run-off takes into consideration both the intensity and duration of rainfall, the elapsed time for water to reach a given point on the stream from various parts of the watershed, and the ratio of run-off to rainfall. The total rate of discharge is then computed from a combination of these several factors. H. K. Barrows says that "usually surface flow on lateral slopes will move with a velocity of from 0.50 to 1 mi/hr., on small branches the velocity may be from 2 to 4 mi/hr., depending on the amount of fall." He also states that "the percentage of rainfall flowing off under flood conditions is likely to be from 50% to 75%, although with conditions of sandy soil and flat slopes this may be considerably less." A maximal ratio of run-off to rainfall equal to 75% seems too low, for it is conceivable that the run-off may equal or even slightly exceed the precipitation if a concentrated warm rain falls on a rough terrain covered with deep snow. The stream flow in such an instance is determined not by the precipitation alone, but by the combined rainfall and melted snow. Discharge determined by the rational method is usually calculated by substitution in an equation of the form $Q = C\ I\ A$, where Q is in cubic feet per second; C represents the ratio of run-off to rainfall, sometimes termed the run-off coefficient; I is the intensity of rainfall; and A is the area of the watershed.

5. Accurate measurement of the stream flow can be made with a current meter if a suitable metering station is available. The first step in the procedure is to divide the vertical cross-section of the stream into component areas. The mean velocity in each component area or section is then obtained by manipulation of the meter in one of several ways, by: (a) moving the meter slowly and at a uniform speed from the water surface to the bottom and up again, the mean velocity being obtained by integration of the meter readings; (b) averaging the velocities measured at given intervals of depth; (c) assuming the mean velocity equal to the average of the velocities measured at 0.20 and 0.80 of the depth; and (d) assuming the mean velocity to occur at 0.60 of the depth. The number and size of the component sections must be decided by the judgment of the engineer, which will be guided by the relative regularity of the stream bed. The integrating method is seldom used because of the difficulty of maintaining uniform speed of movement. The multiple point method is dependable, its reliability being influenced to a large extent by

the lengths of the intervals employed. This method will require more time than any of the other three. The 0.2-0.8 method is the one preferred by most engineers. The 0.6 method will lead to approximate results only. The meter registers the number of revolutions and hence it is necessary to employ a calibration curve, which correlates the velocity of flow at a given depth with the corresponding number of revolutions per unit of time.

6. Reliable data of discharge are obtainable with measurements made at a weir, or dam, placed across the stream, the rate of flow being determined by the size and shape of the weir and the depth of water over or through the structure.

All these methods, except the two which make use of the current meter or the weir, require measurement or else close approximation of the rainfall. The number of rainfall gages is being increased yearly, but the time is far distant when the number will suffice for hydrological purposes. And until such time arrives, the correlation of stream flow to precipitation must rest in large measure on the engineer's judgment.

Until about a decade ago, many, if not most, engineers assumed a peak flood flow of 600 sec.-ft./sq. mi. of drainage area was adequate for design purposes in the central section of the Mississippi Valley. At the present time, however, an increasing number of the profession believe that hydraulic structures on small streams should be capable of handling a flood of from 2,000 to 2,500 sec.-ft./sq. mi. One approach to the problem is to suppose that any given area may be subjected to the greatest storm which has ever occurred on any similar basin in the same region. Assumption of run-off equal to rainfall will lead to conservative results, which will be more reliable than those obtained from a study of a stream's past record, mainly because rainfall data are more numerous and generally more dependable than are those for stream flow.

THE HYDRAULIST VS. THE HYDROLOGIST

The previous discussion of the relation of run-off to rainfall is the expression of a hydraulist, or hydraulic engineer. The hydrologist, however, treats this relationship from quite a different viewpoint. He makes use of no run-off coefficient. Rather, he prefers to evaluate the stream flow as a difference between the precipitation per period of time and the corresponding amount of such precipitation which filters into the ground, is evaporated into the atmosphere, and is taken up by vegetal growth. Theoretically, the premises of the hydrologist are the better of the two. His handicap arises from the difficulty in determining these so-called water-losses, all of which vary with change in the seasons of the year and with particular location on the watershed. For instance, in order properly to compute the seepage, he must know: (1) the character of the ground, which varies laterally and with depth throughout the watershed; (2) the porosity of the soil; (3) the depth to the water-table; and (4) the slope of the hydraulic gradient. There is no precise instrument nor method for determination of the evaporation from land surfaces. Moreover, only in rare instances would the rate of evaporation be constant over an entire drainage basin. And so it would seem that the hydrologist is as much dependent upon assumption and judgment as is the hydraulist, who leans so heavily upon the coefficient of run-off.

Culvert Design

Many designers of culverts and small bridges compute the required cross-sectional areas of waterway by the use of empirical formulas similar to the expressions previously discussed for computing run-off. Both sets of equations possess about the same advantages and disadvantages. The rational approach to the problem is to treat the culvert as a short pipe, or tube, flowing full at peak discharge. It can be easily proved, with application of the Bernoulli theorem, that $q = A\,C\,\sqrt{2\,gh}$, where q is the discharge in cubic feet per second; A is the cross-sectional area in square feet; C is the coefficient of discharge, determined experimentally; g is equal to 32.2 feet per second; and h, the head, is the difference in elevation in feet between the water surfaces at inlet and outlet. The value of C will be influenced by the shape, cross-sectional area, and length of the culvert; the material of the culvert; the head; form of entrance and outlet; and depth of submergence. Values of C are included in various handbooks, notably King's *Handbook of Hydraulics.* More detailed information is available in the pamphlet, *Flow of Water through Culverts,* by Yarnell, Nagler, and Woodward, published by the State University of Iowa.

The prime prerequisite in the design of a culvert is to provide sufficient hydraulic capacity at all times. Failure to do so may produce excessive ponding, which in turn may cause: (1) damage by flooding to land and other property upstream and even to the roadway itself; (2) excessive lateral currents along the right-of-way, necessitating the upstream approach embankments to be paved with riprap or otherwise protected against erosion; (3) protective works at the outlet to guard against scour caused by turbulence; and (4) restriction of the waterway as a result of the lodgment of debris at or near the inlet. Curvature of the stream in the vicinity of the inlet acts to restrict flow and, in extreme instances, may cause the stream to cut a path directly through the approach embankment. Protection against erosion will be required wherever a stream flows parallel and close to a roadway. So, in the final analysis, the engineer will be called upon to make decisions based on his experience and judgment quite apart from correct mathematical applications.

SIMPLIFICATION OF HYDRAULIC COMPUTATIONS

W. E. Howland, Professor of Sanitary Engineering,
Purdue University

The solution of some of the hydraulic problems that the city engineer meets has been facilitated by various methods recently developed. I am going to mention two of these. The first is the computation of flow or pressure drop in a pipe network. Methods are now available which enable the engineer to reduce certain apparently complicated systems to a single equivalent pipe of definite length. One pipe may be said to be equivalent to another or to a system of others when it can carry the same total flows at the same over-all pressure losses. When once this equivalent length has been found, then the answer to any one of a number of questions may be read directly from pipe-flow tables or charts, which are readily available.

In solving such problems it is usually desirable, first, to reduce all pipes in any given system to individual, equivalent single pipes of a single size. This may be done by the use of Table 1. This table shows, for example, that one foot of 6-inch pipe is equivalent to 4.06 feet of 8-inch pipe. Hence, if in a given system there were 1,000 feet of 6-inch pipe, this could be replaced by 4,060 feet of 8-inch pipe.

The next step in the solution of a network problem is to combine the pipes in pairs whenever possible and then to find the equivalent lengths of these pairs. This may be done by means of Fig. 1. First find the ratio of the lengths of the two pipes (now converted into equivalent lengths of the same size of pipe). This ratio will be called L_2/L_1. L_2 will be the smaller of the two lengths. From the righthand side of one of the lines in Fig. 1, locate this value; then read the corresponding value on the lefthand side of the line. This is the value of the ratio of the equivalent length of the pair, L_x, to the length of the stated one, L_2. Finally, multiply the smaller length L_2 by this Ratio L_x/L_2 to find the length of the pair, L_x. This will be illustrated by Example No. 1.