THE DRAINAGE OF AIRPORTS

BY

W. W. HORNER
## CONTENTS

<table>
<thead>
<tr>
<th>I. INTRODUCTION</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Objectives of Airport Drainage</td>
<td>5</td>
</tr>
<tr>
<td>2. Source of Water</td>
<td>5</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>II. SUB-SURFACE DRAINAGE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>3. Necessity for Sub-surface Drainage</td>
<td>5</td>
</tr>
<tr>
<td>4. Disposal of Surface Water Through Ground</td>
<td>11</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>III. SURFACE WATER DRAINAGE</th>
<th>PAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>5. Removal of Surface Water</td>
<td>17</td>
</tr>
<tr>
<td>6. Rainfall</td>
<td>17</td>
</tr>
<tr>
<td>7. Frequency and Amount of Permissible Flooding</td>
<td>18</td>
</tr>
<tr>
<td>8. Inlets</td>
<td>21</td>
</tr>
<tr>
<td>9. Determination of Surface Runoff and of Required Capacity of Drainage Network</td>
<td>23</td>
</tr>
<tr>
<td>10. Hydrology of Surface Runoff</td>
<td>26</td>
</tr>
<tr>
<td>11. Design at Lambert Field, 1929</td>
<td>27</td>
</tr>
<tr>
<td>12. Design at Washington National Airport, 1939</td>
<td>28</td>
</tr>
<tr>
<td>13. Idlewild Airport</td>
<td>39</td>
</tr>
<tr>
<td>14. Drainage Design for Army Fields</td>
<td>46</td>
</tr>
<tr>
<td>15. Inlet Spacing and Field Shaping</td>
<td>48</td>
</tr>
</tbody>
</table>
DRAINAGE OF AIRPORTS

I. INTRODUCTION

1. Objectives of Airport Drainage.—The objectives of airport drainage may still be set out in about the same terms as stated by the Joint Engineering Committee in 1931.* "Drainage systems are proposed to remove surface water, intercept seepage, and stabilize ground through the lowering of ground water levels." Really, there are only two objectives:

(a) The reduction in soil moisture of the upper soil horizon, in the interest of better stabilized soil and of increased bearing power. This objective applies equally to the sub-grades of runway pavements and to the surface soils, either between runways or for all-over landing fields.

(b) The removal of surface runoff, during and shortly after periods of precipitation, to the extent necessary to permit landing and take-off safely. The actual extent and detail to which the achievement of these objectives should be undertaken must be determined separately for each airport after an analysis of probable operation and a full study of rainfall occurrence and soil characteristics. A good approach to a decision in this matter is contained in Table III of Chapter XXI, War Department Engineer Manual.

2. Source of Water.—The surface water for which drainage capacity must be provided consists of that part of the rainfall on the field which does not pass underground through the process of infiltration, and also surface runoff arriving at the field from other adjacent areas. The sub-surface water may be either that part of the rainfall on the field which has passed into the soil through infiltration, or may be migrating ground water below a general fluctuating water table.

II. SUB-SURFACE DRAINAGE

3. Necessity for Sub-surface Drainage.—The extent to which sub-surface drainage is needed or justified has been a matter of controversy between airport engineers from the beginning, and real engineering practice in this direction is only now emerging from the realm of generalized opinion. The engineering of the earlier airports, particularly prior to 1930, commonly involved the installation of sub-

surface drainage systems modeled largely on agricultural drainage practice. This was due in part to the availability, and presumed competence, of agricultural engineers who worked out this problem; it was probably due to an even greater extent to the promotional activities of the industries supplying pipe.

Many of the earlier fields were provided with gridirons of such pipe, which, in some cases produced results considered highly satisfactory. The principal detrimental effect of such installations was the concurrent assumption that they rendered any considerable surface water drainage unnecessary. Today engineers are fairly well away from the idea that surface drainage can be taken care of in any considerable part by routing it through the soil to sub-surface drains.

Sub-surface drainage need be and always should be considered in connection with site selection. The selection for an airport site of

---

DRAINAGE PLAN FOR IMPROVEMENT OF ST. LOUIS MUNICIPAL AIRPORT (LAMBERT FIELD)

FIG. 1. Drainage Plan for Improvement of St. Louis Municipal Airport (Lambert Field)

---

water-logged or swampy land, which would obviously require extensive sub-drains to lower the water table, would rarely be justified unless it were feasible to fill over it to a considerable depth. It is extremely rare to find any justification of the selection of a site that would require complete sub-surface drainage to produce a stable surface; consequently, sub-drainage for the purpose of lowering the water table will generally be carried out only where needed for some small portion of an otherwise satisfactory area.

Lambert Field at St. Louis is a good example of this situation. Originally Cold Water Creek lay across the middle of the airport. In the development plan, this Creek was detoured around one side. A small portion of the land near the original creek banks, where the soil was somewhat finer than average, had a high water table over most of the year. While this area was raised by 3 or 4 feet in the course of grading, it was considered desirable to install several lines of tile drainage prior to the filling (see Fig. 1).

On this project the field had a natural satisfactory slope of about 1 per cent. The soils to a considerable depth were silt loams largely from basal loess, with a very good transmission and percolation capacity. Normally, the water table in the wet season varied from 3 to 5 feet below the surface except in the area referred to, where the low transmission capacity forced it up. In the course of laying out the surface drainage system it was decided to refill all trenches with crushed stone, and this, among other things, has acted to lower the water table within the field materially; particularly along the edge of the apron, the relatively deep sewer line with crushed-stone backfill tended to cut off ground water migrating from the uplands outside the port area. The effect has been particularly satisfactory, and the soil surface at this port has a high stability throughout the year (see Fig. 2).

The question of sub-surface drainage may be approached also through soil types. It is almost invariably a fact that those soils which become unstable when wet have a high silt or silt clay content which results in their also having a low infiltration capacity and a low water transmission capacity. Therefore, infiltration during rainfall does not result in a great increase in soil moisture or any considerable amount of free water which can be removed by drainage system. It is further true for such soils that it is difficult to remove free water from them without permitting some of the soil to escape into the drains.

At the other extreme of the soil classes, the coarse-grained soils will generally have a high percolation capacity, and, if the water table
is at a reasonable distance below the surface, free water will percolate to it readily and the addition of under-drains would add little to the picture. With such soils no sub-surface drainage system would be justifiable unless for the purpose of lowering and maintaining the water table at a satisfactory depth below the surface.

On runway fields, handling planes of the heavier type, sub-drainage lines along the edges of the runways may occasionally be justified in order to reduce the possibility of lateral transmission of free water from the edge of the runway into the pavement sub-grade. For such fields, interrunway spaces are not in normal use, and no considerable expenditure can be justified to make them fully satisfactory for landing. The all-over type of field is useful only for training and light private planes, and a high soil-moisture content, for short and infrequent periods, would not involve a situation serious enough to justify large expenditures.

In the majority of cases the A horizons, or surface soils, because of their organic content, are more porous than the sub-soils. During prolonged wet periods infiltration into these upper horizons will be considerable, and will be in excess of the capacity of the sub-soil to dispose of the water through percolation. Quite commonly, therefore, the surface soils will be unstable during wet weather to a depth of 8 inches or more. Under normal conditions this excess water will be removed in part by percolation but primarily by evaporation and transpiration. It could not be removed by sub-drainage systems any more quickly unless the sub-drains were spaced at close intervals such as 10 feet or less, which is completely impractical. The best procedure for improvement of this situation is to build up a strong stand of turf at the earliest possible time, and to rely largely on transpiration for the removal of excess water in the top soil.

An example has already been given in the case of Lambert Field, where sub-drainage was encouraged in the interest of maintaining the water table at a considerable depth below the surface, and effectuated almost entirely by refilling the trenches of the surface drainage system with coarse stone. Two examples of more recent practice are interesting.

**Military Airfield “A”**

This field covers about 4 square miles, and was developed as a runway field in one section and as an all-over field in the other sections. The surface soil, to depths varying from 1 foot to 3 feet or more, was a fine-grain silt loam having a low bearing capacity in its natural state when wet. It was underlaid by a clay-pan formation throughout, which was practically impervious and had a very low moisture content throughout the year. The only water table was an ephemeral perched water table in the surface soil at the end of prolonged wet seasons (Fig. 3). The surface soil had a fair infiltration capacity rate when dry, but a very low rate after an initial rain, as shown by the results of infiltrometer runs, and the rainfall generally had to be disposed of...
as surface runoff (Fig. 4). Because of the shallow surface soil and a general absence of free water in the lower portion of it, it was obvious that sub-surface drainage would accomplish little, if anything, and would be extremely difficult and expensive to install in such a manner as to prevent the entrance of the soil into the joints. For the all-over field it was accepted as inevitable that the upper 6 inches of the soil would become unstable in prolonged wet weather, and reliance was placed on the turf to remove the moisture by transpiration as quickly as possible. This field is in an area where the soil would never freeze to any appreciable depth. This same surface soil when compacted at optimum moisture content had a fair California bearing ratio (10 to 15 per cent) even when saturated. While it was required to design the pavement for saturated sub-grade conditions, it was quite clear that, because of the low transmission capacity of the soil, the moisture content of the sub-grade would never reach unsatisfactory values, except in narrow strips immediately adjacent to the edges of the runway. Accordingly no sub-surface drainage was installed.

Military Field “B”

This field has a deep surface soil of sandy loam, to an average depth of possibly 2 feet, over a B horizon of sandy clayey silt 7 to 15 feet in depth overlying fine sand reaching to a depth of 50 feet or more (Fig. 3). The normal water table is in this fine sand at from 15 to 30 feet below the surface. Infiltrometer studies were made of this soil during a wet spring period and showed mean infiltration capacities during the first hour of application to average about 0.5 inch per hour. Continuation of these runs indicated that, after 2 or 3 inches of water had been infiltrated, the apparent capacity dropped off to an average value of about 0.1 inch per hour. This latter rate clearly indicated the rate of percolation in the B horizon. This percolation however, even under these conditions, was at about as high a rate as water could normally enter sub-drains, and it was clear that the installation of the sub-drainage system could not appreciably reduce the soil moisture during such prolonged wet periods. Since the water table was far below the surface, no sub-drainage for the reduction of its level was indicated. Therefore, as in the previous case, it was clear that the drainage system would have as its only objective the removal of surface water.

In conclusion, it has become more and more evident that, for properly selected airfields, need for a sub-drainage system generally applies to isolated and relatively small parts of the area; but some improvement in sub-drainage facilities, if it can be produced without too great expense, may be justified to lower somewhat further a water table that might otherwise during wet periods rise to within 3 or 4 feet of the surface.

4. Disposal of Surface Water Through Ground.—While it has become clear that satisfactory airport drainage cannot be accomplished through the use of sub-drains entirely, or in greater part, yet, on an airport project where the soils are largely sands and gravels, it is very tempting to try to dispose of the greater part of the surface water through the ground. This possibility had serious consideration in connection with the design of two large projects, namely, the Washington National Airport, and the New Idlewild Airport in New York City.

The Washington National Airport was built up on the Gravelly Point shallows of the Potomac River by hydraulic dredging. The filling was carried out first for the runways by trenching out the existing Potomac River mud down to the old gravel, and thereafter filling in along the runways with the sand and gravel from the channel of the river. These fills were made for a width of about 500 feet with mixed dredged material, ranging from fine sand to boulders of 12 inches or more. The intervening field pockets were then filled by dredging in Potomac River mud. The level of the field is about 10 feet above ordinary river level of the Potomac, and it was anticipated that the water table in the sand-gravel fills would be only slightly above the river level.
It seemed that no sub-drainage would be needed in the runway fills, and that none could be installed in the pocket fills, which were liquid mud and are still in the process of consolidation. The work was carried out under the Washington National Airport Commission of which Colonel Sumpter Smith was Chairman, and on which the Civil Aeronautics Administration was represented, and acted as advisors on general airport planning. The selection and design of runway pavements was carried out by Mr. H. H. Houk, Chief Engineer, for the Commission, and the preliminary plans for drainage were prepared by the writer, as consultant. The plans provided for a relatively complete system of surface drainage, with the initial lines laid in the sand and gravel shoulders, parallel to the runway pavements. The complete plan involved inlets and laterals in the field pockets, but it was not expected that these would be constructable for several years. The detailed plans and the general supervision of construction was under the U. S. Army District Engineer at Washington. After the design and general plan of the surface drainage system had been completed, the District Engineer questioned the necessity for a positive system of pipe drains, and expressed the opinion that the surface water could all be disposed of by infiltration into the sand and gravel runway fills, and transmitted through these fills as ground water through an outlet at the runway ends into the Potomac River. Members of his staff had demonstrated that a bucket of water poured on the porous fill disappeared almost immediately. It was quite obvious that no such disposition could be made of the amount of surface water involved, and that there was a lack of realization of the mechanics of infiltration and ground water transmission. For example, it does not occur to the layman and quite often not to the engineer, that water discharged on a small porous area, disappears quickly because the voids are empty and water can spread laterally in all directions, whereas water applied uniformly on large areas of this kind can only be disposed of by direct downward percolation.

A great deal of testing and research work had to be carried out before the local engineers became convinced that this idea had extremely limited possibilities. At the writer's suggestion, standard rainfall simulators were secured from the Soil Conservation Service and artificial rainfall was applied on 6 x 12-foot areas. The results were so interesting that they are rather completely summarized in Fig. 4.

It would seem that a sustained infiltration capacity of only a little over 1 inch was evidenced on the first application of rainfall, and that this dropped down to values comparable to those for an average soil on additional applications. Undoubtedly this reduction in capacity was due in part to the effect of rain impact in re-grading and choking in the fines; but it may also have reflected the limited storage available in the sand and gravel above a relatively high water table.

Actually it was not feasible to leave these sand and gravel shoulders exposed for infiltration purposes, and the original plan involved top soiling them. A second series of tests was run on an area where mud was incorporated with the sand and gravel in about equal amounts to a depth of 8 inches. The resulting infiltration curve is shown in Fig. 5. It is obvious that such a combination has a lower infiltration capacity than either the sand and gravel or the mud alone. A better method would be to place the mud top soil over the sand and gravel without mixing.

From these results it was clear that only a small part of the tributary surface runoff could be taken in through the sand and gravel surface, even if left exposed, and that the proposed system of pipe drains would be necessary and would be very little less in extent than originally contemplated.
The idea of disposing of surface water into the porous fill once having been proposed became extremely tenacious, and the District Engineer raised the second question, namely, if this water cannot be put in through the ground surface but must be taken into inlets, why not discharge it into the sand and gravel fill through porous pipes? Two objections were raised to this: (1) that the amount that could be so discharged would not be very great, and (2) that the sand and gravel fill had been shown to contain a large percentage of extreme fines, therefore porous pipe that could exude water might also under a reversal of conditions, say a high river stage and no rainfall, introduce water and sand into the drainage system. The second objection was never received with much conviction, but the first was subjected to a series of full scale tests. For these tests a wooden box inlet was constructed to which there were attached several lengths of perforated metal pipe. The pipe and inlet were filled with water and the discharge into the sand-gravel fill was determined by the drop of water level in the inlet.

These tests indicated that, with a head of 2 feet on the pipe perforations, on the average, water could be discharged into the sand and gravel fill at a rate of about 0.15 cubic feet per second per 100 feet of pipe length. This value is on the order of 10 or 15 per cent of the tributary inflow.

While the quantity of water shown to be distributable in this manner by those tests had to be discounted because of the possibility of lower heads and the regrading and further compaction of the fill, and while the amount was a relatively small proportion of the required pipe capacities, nevertheless the decision would have been to use perforated pipe if it had been found to be economical. Actually it was found that the increased cost of perforated pipe more than offset the reduction in size, and the proposal in this form was abandoned.

After construction was actually started it was found that, as the result of infiltration, a high water table was being permanently built up in the sand and gravel fill, and that some sub-drainage would be necessary to keep the water table appreciably below the surface. Accordingly, while no change was made with respect to the main sewers, the smaller lines along the runways were constructed with partially open joints. This was accomplished by chipping out a part of the spigot of the concrete pipe on the two sides of the pipe, leaving out the cement mortar at those points, and covering the joint with wire mesh. The system appears to operate in the following manner:

At the beginning of heavy runoff, water percolates out from the pipe into the sand and gravel; shortly this becomes saturated and percolation is reduced to a small quantity; between rains, however, water percolates into the pipes and prevents the water table from rising appreciably above the level of the sewers.

Idlewild Airport

This project now under construction is probably the largest civil airport proposed in the United States. It is constructed on the old marsh, on the Long Island side of Jamaica Bay, which was originally at about elevation +5 feet above mean low tide. The filling was carried out by hydraulic dredging from the Bay and the material is a fine beach sand similar to that at Rockaway Beach. The finished elevation of the field varies from 12 to 15 feet above mean low tide, thus requiring a depth of sand fill averaging 8 or 9 feet when compaction of the marsh mat is taken into account. Since the fill has a dimension of nearly 10,000 feet in each direction, the center of the sand fill is a long way from the outlet of ground water along the margin.

When the writer was called in as consultant on drainage, he recommended a positive system of pipe drainage to carry the whole of the expected surface runoff, and also called attention to the probability that a water table would be built up in the sand fill that would be very close to the surface. His function as drainage consultant was to determine the required capacities of the storm drains, actually to design one section of the storm drainage, and to set up a method of applying the design to the remaining sections. The details of this work are summarized in a later section of this paper.

As a result of the question which was raised about ground water, the engineer for the project, Mr. Jay Downer, installed, in September 1942, eight observation wells, and later installed eight additional wells. The ground water level was measured in these wells at intervals, and generally after each rainfall period. The character of the water table which was built up is shown in Fig. 6. As would be expected, this water table shows a slight but steady recession during dry weather, and a quick rise following rainfall. A 1-inch rain produces about 1 foot of rise in the ground-water table in the center of the field; a 3-inch rain is capable of raising it about 2 feet. The average water table in the center of the field is now about elevation 12, or very close to the intended finished grade in the field pockets. The engineers have under consideration the installation of some sub-surface drainage to accelerate ground water depletion.
In this project, as at Washington National, the possibility of disposing of rainfall into the fill appeared in an even more attractive form, since the whole of the interrunway space had been filled with porous sand up to the surface. Portable infiltrometers were set up on the undisturbed sand surfaces and infiltration capacity values were determined. The average results are shown in Fig. 6. Here again infiltration capacity was high on the first application but appeared to be reduced steadily thereafter. Actually these lower values occurred after a mounded water table had been built up around the infiltrometers, and really represented the rate at which the sand could distribute the infiltrated water, and not the true infiltration capacity of the sand surface. It seems probable that infiltration capacity would be sustained for several hours at rates of about 3 inches if storage space were available in the sand.

For this field there appears to be a very delicately-balanced condition, undoubtedly a large amount of rainfall can be disposed of into the sand provided the water table in the sand can be held down several feet by sub-drainage. It is entirely possible that all the rainfall on the sand surface can be satisfactorily disposed of in this manner. It is highly questionable, however, whether these sand areas can dispose of the run-off from the paved surfaces also.

III. Surface Water Drainage

5. Removal of Surface Water.—The common, and generally the primary, problem in airport drainage is the removal of surface water to the extent required to permit safe and satisfactory landing and take-off of airplanes. Almost invariably it is necessary to investigate the adequacy of the water courses in the vicinity to determine whether some improvement of them is necessary for the protection of the field from floods and also whether they can satisfactorily serve as outlets for the field drainage system. Many airports are situated on valley lands, and it is quite common to find that these lands in the past have been subject to flooding from adjacent water courses, and channel improvement or leveeing of the field may have to be undertaken. Each such situation, however, is a separate problem, and should be considered as coming under the head of “flood protection” rather than airport drainage itself. In the discussion which follows, it is assumed that the field has been rendered free from flood hazards and that an adequate outlet for the field drainage is available.

6. Rainfall.—The origin of the water to be removed by a surface drainage system is obviously the rainfall on the field surface, and the first investigation must necessarily be to determine the rates and character of rainfall for which the system should be adequate. The most extensive study of rainfall occurrence is that which was made by
Yarnell.* Municipal engineers have made more detailed studies for the larger cities; in Fig. 7 is shown a rainfall curve which was developed for Washington, D. C. However, similar curves can generally be prepared with sufficient accuracy from Yarnell's data. These curves must be interpreted as follows:

For a period of precipitation of 30 minutes, an average mean rate of rainfall of 2.55 inches per hour may be expected to be equalled or exceeded once in 2 years; note that this refers only to mean or average rate, and takes no account of the actual rates at which rain may occur within the 30 minutes.

In these studies it was found that precipitation practically never occurs at anything approaching a uniform rate for a period, say, of 30 minutes. The commonest type of precipitation involves a rate much higher than the average for a short part of the time, with this high rate occurring somewhat before the middle of the period. The actual pattern of rainfall occurrence is an important factor in determining the resulting rate of runoff as shown in Fig. 8.†

7. Frequency and Amount of Permissible Flooding.—A decision on this point is a pre-essential to the design of any drainage system, and is one of the criteria which has varied widely from time to time to meet the current judgment of airport managers and airport engineers. There appears to be a present concensus of opinion that field operation, of civil airports, will be satisfactory, provided that standing surface water does not encroach on the runway pavement oftener than once in 2 or 3 years, and provided, further, that water shall not encroach on the central 100-foot width of runways oftener than once in about 10 years. Such a situation apparently will not prevent a reasonably satisfactory operation for major air transport lines in and out of the field. Somewhat similar requirements have been used recently for military service airports and for training fields. Actually, for training fields, materially lower drainage standards are permissible, as training operations can be suspended for a few hours without any great damage or loss; whereas civil air transports approaching an airport must be able to land under practically all conditions.

These considerations may be summarized into a basic requirement that an airport drainage system must be adequate to remove surface water from rains occurring on the average once in 2 years to the extent necessary to prevent pondage on or within 50 feet of runways; the

---

*United States Department of Agriculture, Miscellaneous Publication, No. 204, August 1935.
There is a drainage problem, therefore, becomes one of determining the location and capacity of the drainage lines which will produce this result.

The first step in the development of the drainage system is the location of surface inlets, the provision of an adequate inlet capacity, and the laying out of the drainage network which will convey water from the inlets to the available points of outlet in the most direct or economical manner. The general form which such skeleton systems take for three major civil air fields is shown in Figs. 1, 9, and 19, and for certain training fields in Figs. 10 and 11.

8. Inlets.—In the earlier days of airport engineering, there was an idea that the field surface must be uniformly resilient throughout, and
that, therefore, manhole heads or cast iron inlet gratings were objectionable. To avoid such structures, the earlier attempts were to take surface drainage underground through exposed porous rock fills. This type of intake was installed initially at Lambert Field at St. Louis, only for it to be found, as in most other instances, that it was impossible to maintain a satisfactory porosity of the crushed rock surface. Invariably these became filled with grass cuttings and other fine debris, and with dust and silt. The initial intake installation along the apron of Lambert Field is shown on Fig. 12, and the intakes along the runways were the continuous rock-filled trenches. Within 2 years the intake system was entirely rebuilt with a continuous cast iron grating along the apron, and with positive inlet gratings at frequent intervals along the runways.

Another objection to the loose rock fills was their displacement under airplane operations. To meet this objection some engineers gave the rock in the upper part of the field a thin film of tar and asphalt with the idea of gluing the rocks together. Generally, however, this was found merely to accelerate the clogging of the pores. Today airport drainage almost invariably involves the construction of positive inlet gratings in adequate size at definite intervals. The inlet gratings first designed for the Washington National Airport are shown in Fig. 13, and the type of wooden grating which was recently used on military training fields in Fig. 14.

9. Determination of Surface Runoff and of Required Capacity of Drainage Network.—In municipal engineering practice prior to 1905 surface runoff was computed from various formulas that had been devised. None of these formulas did or could possibly give approximately right values over any considerable range of conditions. In 1907 a designing engineer for the City of St. Louis developed a method for determining probable storm runoff from rainfall adapted from the theory originally promulgated in 1887 by Kuichling. A description of this technique was published under the title of "A Rational Method of Storm Sewer Design." It met apparently an urgent need, and, with various modifications, was adopted quite generally in the United States under the retained title of "The Rational Method." The relationships back of this method are expressed in the formula $Q = cA$. This can be interpreted as stating that the runoff from any drainage area expected to occur with a particular frequency, as, for example, 2 years, is equal to a coefficient "$c$" multiplied by the mean rainfall rate from a 2-year frequency rainfall curve, for a duration period "$t$", which

*Engineering News, September 29, 1910.*
is equal to the time required for the water to flow from the watershed line to the point under design; this in turn is multiplied by the area in acres.

This theory was actually rational in some respects in that it gave a fair approach to a choice of the "Design Storm," but, as is now known, was somewhat irrational in respect to the means of determining the critical time \( t \), and could be approximately accurate only if somehow a table of values of the coefficient \( c \) could be acquired that would fit all of the conditions which were likely to occur.

---


---

In passing, the writer suggests that an adequate table of \( c \) values would probably require a large size pamphlet for their presentation.

In 1907 a series of \( c \) values was suggested that seemed to fit experience for average-developed urban areas in St. Louis. Later, after 20 years of sewer gagings, these were revised in part for the same use and were presented in a paper before the American Society of Civil Engineers. The only other good data with respect to the coefficient \( c \) were those worked up by C. H. Ramser in connection with studies of land drainage for agriculture areas.

In spite of this paucity of particular coefficients, the use of the rational method has continued in engineering practice. As late as 1938 an otherwise capable engineering office was attempting to design airport drainage by applying to a very flat airport the same coefficients...
10. Hydrology of Surface Runoff.—It would be well to stop at this point and see what the coefficient $c$ has been made to represent. In Fig. 15 there are shown the mechanics of the production of surface runoff for a rain of two uniform intensities. This diagram represents actual occurrence under carefully-controlled simulated rainfall. It will be seen that the first thing that occurs is the division of the rainfall, controlled by the infiltration capacity, into infiltrated water and surface runoff supply. The result of the application of the infiltration capacity values to the rainfall intensity shows the development of surface runoff supply indicated as $\sigma$.

The supply curve shows the rate at which runoff is produced on the ground surface. The differences in the ordinates between this and the runoff rate curve represent the rate at which detention storage takes place. The figures in the hatched areas show the actual inches in depth of surface water going into detention storage before equilibrium is reached. Two values of coefficient $c$ are shown. These, of course, could be used to give the rate of runoff at the lower end of a 12-foot plot. Many other things happen to change these coefficients before the water actually gets into the inlets.

As soon as surface supply exists and retention ponds are filled, surface runoff begins and is controlled by the laws of hydraulic flow. In order that runoff rates may increase, flow depths, and therefore storage, must also increase. The resulting hydrograph shows how runoff rates increase as surface detention increases, and the equilibrium finally occurs between runoff and supply at the peak of the hydrograph. The ratio of this peak runoff rate to the mean rainfall rates is the coefficient $c$. Approximately, $c$ is equal to $I - f$, but since both of the values of $I$ from the rainfall curve and the values of $f$ from the infiltration capacity curve, are functions of time, and since the time involved is a function both of velocity of flow and of the filling of storage, it can be shown that a real equation for $c$ looks something like the following:

$$c = \frac{K - K'}{K} K'' \sqrt{\frac{A}{S}}$$

where $K, K',$ and $K''$ are parameters of the equations of the rainfall curve and the infiltration capacity curve and of the hydraulic flow, formula $A$ is the area in acres and $S$ the controlling slope. This should illustrate very readily, the futility of attempting to find a few values of $c$ that can satisfactorily represent the relations between rainfall and runoff.

Engineers have reached the conclusion that there is no simple and royal road to the determining of surface runoff, and that the calculation of fairly representative values of runoff must necessarily involve as much detailed engineering computation as, for example, the stress analysis in a bridge.

11. Design at Lambert Field, 1929.—The writer reached this conclusion as early as 1929 in the course of a design of the drainage system for Lambert Field. This system was designed for a 1-year frequency rainfall with all surface water to be removed within 2 hours after the end of the rain. An extended study was made of the hydrology of one sub-system, during which there were calculated the infiltration, the build-up of surface detention, the storage in porous rock fills, and the time of travel and storage in the pipe system. For this
particular test section, it was found that the maximum capacity would be required for a rain of 45 minutes duration and that the maximum runoff would be 0.42 cubic feet per second per acre, or about 30 per cent of the mean rainfall rate. This value was made the basis for a runoff diagram where it was varied with the percentage of imperviousness, the size of the area, and the time of flow. What was actually done there was to determine the coefficient of $c$ by detailed hydrologic and hydraulic analysis.

12. Design at Washington National Airport, 1939.—In engineering practice there is usually little opportunity to pioneer in the development of new techniques. It is not surprising that in the 10 years following the design of the Lambert Field drainage system practically nothing was published, and apparently little was done to improve the methods of calculating surface runoff for air fields.

However, in 1939, the development of the National Airport at Washington, D. C., afforded an opportunity to make a rather detailed study of the facts affecting drainage design. This port was being developed under an independent engineering commission, imbued with the idea of making it the finest airport in the world. Among other things it was desired that the drainage system be adequate in every respect, but without any excess expenditures on that account. This indicated a need for and a possibility of a detailed engineering and economic analysis.

At that time there had also become available important new information on the infiltration capacity of soils, and new conceptions of the hydraulics of overland flow. In the studies which were carried out and which are set out in detail in the final report of the consulting engineer on “Drainage Facilities for the Washington National Airport,” full use was made of our improved understanding of the mechanics of surface runoff and of infiltration. Among the separate studies which were carried out, the following are the more important:

1. Probable infiltration capacities of hydraulic-filled sand and gravel areas covered with top soil
2. Probable infiltration capacities of dredged-in mud fills after consolidation, drying, and the development of turf
3. The rate-reducing effect, on runoff, of surface detention on paved surfaces
4. The rate-reducing effect on runoff, of surface detention, on turfed surfaces
5. The rate-reducing effect of routing surface runoff through flat turfed gutters of various lengths and slopes

(6) The development of a technique through which the result of these studies could be incorporated in the design practice.

In the space available here, it is impossible to describe these studies in much detail. With respect to infiltration capacity, it was decided that the maximum rate of 0.8 inch per hour for periods up to about 40 minutes could be applied to all top-soiled sand and gravel, and that a similar rate of 0.4 inch per hour would apply to the consolidated mud-filled inter-
runway spaces. An idea of some of the studies of the relation of detention to runoff can be gained from an inspection of the following figures:

Figure 16 shows how surface runoff would occur from the combination of a 50-foot pavement and a 50-foot turfed strip. The rising side of the hydrographs are shown for this condition for 4 combinations of rainfall intensity and duration; the addition of the hydrograph ordinates gives the maximum runoff from a unit width strip as shown by curves C and G. This is an academic study that assumes that the runoff from each 1-foot strip could enter the sewer system without further storage reduction. The difference between the rainfall curve and curve G, therefore, shows the rate-reducing effect of infiltration and surface detention during overland flow alone.

Figure 17 shows the maximum rates of inflow at inlets spaced 200 feet apart when 100 feet of runway drains over 50 feet of turf and meets the flow from another 50 feet of turf, having the opposite slope. In this figure there are shown both the hydrographs of surface runoff for a similar series of 1-foot strips entitled “runoff into the valley” and the hydrographs of inflow to the inlet entitled “runoff into the inlet.” For example, for a 3-inch rainfall rate, lasting 20 minutes, the maximum rate of inflow into the valley is 2.54; the maximum rate of flow into the inlet is 1.8 inches per hour. This brings out importantly the flow-reducing effect of storage in flat gutters.

Figure 18 shows the effect of different gutter lengths on reducing runoff rates. It is based on surface runoff from the same type of 200-foot wide strips, illustrated in Fig. 17. In Fig. 17 it will be noted, for example, that surface detention and infiltration alone reduced the 3-inch rainfall rate to 2.54. The storage effect of 100-foot gutter flow reduced this to 1.75, a 200-foot gutter flow to 1.15, and a 400-foot gutter flow to 0.75. It will be noted that both surface and gutter dete-
tention is natural detention, that is, it represents the storage required to permit the rate of flow to occur. It is interesting also that the maximum depth of flow at the inlet, even for the longest of these gutters, does not exceed 4 inches, and the edge of the stored water, therefore, does not reach the edge of the runway pavement. (This type of storage and rate reduction is quite different from the "forced pondage" described later.)

The relation of these studies to the actual design of the systems is described in the Report as follows:

On any such area as this, the rate at which surface water originates on the impervious runway and apron surfaces is equal to the rate at which the precipitation occurs. The rate at which water originates on the permeable turfed areas is the rate at which rainfall occurs minus the rate of infiltration, the latter, for design purposes, being the infiltration capacity rate. Therefore, the rate at which the water to be drained off originates is entirely independent of the drainage facilities or any part of them.

To permit runoff to occur, there must be an actual film or sheet of water over the surface. The creation of such a sheet of water involves an abstraction from the supply rate, and consequently this surface storage acts to reduce possible rate of runoff. Therefore, the rate at which water runs off an elemental section of airport surface, whether it is runway or turf, will in general be less than the rate at which surface water originates. This will be true, unless the time of rainfall is greater than that necessary to develop the surface sheet, and therefore is sufficient to permit a stable hydraulic condition to develop. The characteristics of runoff from plane surfaces are shown for the rising side of the hydrograph in the diagrams to which reference has been made. These show clearly how, for short rains, the effect of surface detention is to reduce the rate at which the water reaches the margin of the runway or the lower edge of a turfed surface slope. In any system of surface shaping, runoff as sheet flow becomes tributary to some form of rill, channel, or swale, and along it is conveyed to a storm water inlet and there introduced into the underground pipe system. The flow of water in such a channel or swale at any particular rate can only take place when the channel has been filled to a characteristic related depth, and therefore again a considerable part of the surface runoff goes into temporary surface detention in the channels.

Where the grades of the channels are extremely flat, as at this airport, considerable depths of water are required in them before the rate of flow reaches values characteristic for rainfalls in which we are interested. Consequently the channel storage on the surface, as, for instance, in the channels paralleling the runways, has an extremely important reducing effect on the rate of runoff, and this action develops runoff rates at the inlets in inches per hour that are much smaller than the rates at which surface water originates on the ground.

Obviously the farther the water has to flow as sheet flow over the surface, the lower will be the rates of runoff into the collecting channels. The farther the water has to flow in the channels, or the flatter the grades available to produce this flow, the greater will be the average depth in the channels, the greater will be the proportion of water stored in the channels, and the lower will be the rates of runoff into the surface water inlets.

From the foregoing it should be clear that there is a close, definite relationship between facilities for overland flow, before the water reaches the inlets, and the rate at which the water enters the inlets. This principle makes it possible to reduce the required capacity of the drainage system by flattening the grades of the collecting channels, or by increasing their length, i.e., by increasing the spacing of the inlets themselves. To the extent that this is done, the result will also be to require water to stand in greater depths in the collecting channels during the rain. In the extreme, these channels might be made so long and their grade so flat that water flowing to the inlet would actually rise onto the runway, and the runways themselves would be under small amounts of standing water during a precipitation period.

The application of these principles to the Washington National Airport naturally raises fundamental questions of policy, and a decision with regard to them was one of the most difficult to arrive at. In the chronological statement, the consultant has shown that the first plan which he presented involved the carrying of channel flow on the margins of the paved runways themselves, thereby facilitating this type of flow, holding water in transit to minimum depths, and requiring relatively large pipe systems to carry away the only slightly reduced flow from the inlets. This was strictly in accord with the earlier thought of the Engineering Commission that standing water on any part of the airport during a period of heavy precipitation should be held to the smallest depths possible.

Only after a tentative conclusion that the runways should not be widened, and after a preliminary idea of the expense of the system originally contemplated had been obtained, was the Commission's policy in this respect definitely modified. The final statement of policy by the Drainage Committee as outlined in the memorandum of the meeting of January 20th is extremely interesting from the viewpoint of operations requirement. This statement of policy in effect held that there would be no serious hazard to field operations, even under heavy landing traffic, if the water in the swales adjacent to the runway occasionally rose onto and even halfway up the crown of the runway pavement. Stated in another way, there was no serious hazard to operations so long as a hundred-foot width of all runways could be kept free of standing water during a short period of heavy precipitation.

This decision as to operating conditions radically modified the
of intake capacity are rendered ineffective by the lack of capacity of intakes. This is a detail that is often overlooked and perfectly designed systems requirements actually contemplated in the design of the sewer system. Obviously, also, the capacity of the inlets must be consistent with the longitudinal gutters sufficient to put some water over the runway margins, but should never result in making the un-inundated surface as narrow as 100 feet. The principle of balancing permissible depth of water in transit against the required capacity and cost of the drainage system is a fundamental one in any system of drainage for an airport, and the reducing effect on the size of sewers and on the cost of the system can be adjusted to any level compatible with the maintenance of safe water conditions on the field.

As has been pointed out, the flow quantities on which the drainage design is based are extremely critical with respect to the inlet spacing and the slope of the marginal gutters, and the design is only valid so long as the suggested spacing and slopes are actually developed under construction.

Obviously, also, the capacity of the inlets must be consistent with the requirements actually contemplated in the design of the sewer system. This is a detail that is often overlooked and perfectly designed systems of sewers are rendered ineffective by the lack of capacity of intakes.
Fig. 21. Typical Cross Sections of Runway and Gutter, Idlewild Airport
The actual runoff rates are exemplified in the specimen design sheet F-18. It should be kept in mind that for this field runoff is carried to the inlets through flat grassed gutters.

13. Idlewild Airport.—This project has been described briefly in Chapter II with relation to ground water conditions. At the time when the consultants were called in on drainage, it was the decision that rainfall on the interrunway spaces could be satisfactorily taken care of by infiltration into the sand surface, but that a positive drainage system would be installed to take the runoff from pavements. The general plan of this field is shown in Fig. 19, a plan of a portion of the drainage system in Fig. 20, and a typical cross section showing one-half of the 200-foot wide runway, and also the 50-foot wide stabilized gutter, is shown in Fig. 21.

It will be seen that it is proposed to locate inlets in the stabilized shoulders, and that these are to be warped to form flat longitudinal gutters. The cross section will vary uniformly between that at the inlet and that at the summit between inlets. It was proposed that the storm drains be laid just outside the shoulders with a manhole opposite each inlet.

Investigations which were carried out for this drainage project were therefore related entirely to flow over paved surfaces, and consequently involve relatively high rates of runoff.

For this project it was originally proposed to design the drainage for a five-year rainfall, but later this was modified to provide only for a two-year frequency. In the course of the investigation the following studies were carried out:

(1) The hydrographs of runoff from 150 feet of pavement into the gutter. These hydrographs were prepared for a series of rainfall rates corresponding to various durations.

(2) The determination of the water surface profiles in the flat gutters for various arbitrary rates of flow at the inlet. It could not be found that any hydraulic study of this condition, where inflow increased uniformly throughout the length of the channel, had ever been made. This study was carried out by an application of the energy equation to short sections of the gutter, and was facilitated for the use of functions and a logarithmic plotting of the functions. Typical profiles are shown in Fig. 22. Such profiles were calculated by various combinations of gutter flow lengths and grades. It is interesting to note that all involve backwater at the summits.
From these profiles, graphs were prepared showing the relation of

gutter storage to discharge at the inlet. These were made the basis of

flood-routing diagrams and the overland flow inflow rates to the gutter

were routed through gutter storage to determine the outflow rates from

the gutters, which became the inflow rates to the storm inlet. A typical

result of this type of computation is shown in Fig. 23. From this figure

it will be seen that while the gutter storage on these smooth concrete

surfaces, even with flat grades and considerable lengths of flow, does

not have a large peak-rate reducing effect, nevertheless it makes a

sharp difference in the character of the hydrograph of flow into the

inlet. This shape of hydrograph was found to effect flow rates in the

sewers downstream to an important extent.

As a preliminary study, the required sewer capacity was inves-
tigated by routing the inlet inflow graph through the first length of pipe,

thus determining the reducing effect of pipe storage, and thereafter

adding the inflow graph at the succeeding inlet. The summarized graph

at the second inlet was then routed through the succeeding pipe, etc.

This procedure was carried out for rainfalls of 10, 15, and 30 minutes

duration, and the resulting rates of runoff below each inlet are shown

in Fig. 24. The envelope curve indicates the maximum rates of runoff

below any particular inlet that could be produced by any isolated

period of rainfall conforming to the five-year frequency rainfall curve.
One of the most interesting results of this study is the showing that runoff rates have no specific relation to the rainfall rate for the so-called critical time. Actually for the total time of flow of 11 minutes at the first inlet, the maximum runoff rate would be produced by a rain lasting about 8 minutes. At inlet No. 4, where the time of flow is about 20 minutes, the maximum runoff would be produced by a rain of about 13 minutes duration, and at inlet No. 14, where the time of flow is about 40 minutes, the rain of 25 minutes duration will govern. This situation results from the actual passage of a flood wave through the sewer (see Fig. 25).

The reduction in runoff rates downstream from inlet No. 1 is due in part to the fact that lower rainfall rates become critical, and in considerable part to the rate-reducing effect of surface detention, gutter storage, and pipe storage.

The routing of these various hydrographs through pipe storage is a laborious and tedious process, and a method was devised under which the correct values can be very closely duplicated by offsetting the inlet inflow hydrograph from inlet to inlet by a time amount slightly less than the actual time of flow between inlets. It was found that this could be accomplished if the hydrographs were offset by a time equal to 0.8 of the actual flow time as computed by the Manning formula.

These preliminary studies were made for isolated rainfall periods, and therefore the pipe storage reduction was that related to a conduit initially empty. It was recognized that such critical rainfall periods practically never occur without some antecedent and some following precipitation. Therefore, there was substituted for the isolated rainfall period the pattern storm shown in Fig. 26, and the precipitation occurrence shown in this pattern was routed through surface detention and gutter storage for gutters of various lengths, resulting in a series of inlet inflow hydrographs shown in this figure. In the course of actual design the addition of the hydrograph ordinates was reduced to a tabular form; thus at each new point of inflow a hydrograph was chosen as properly representative of the tributary drainage basin, was offset by 0.8 of the flow time between the last two inlets, and its ordinates were added to those of the hydrograph at the next point upstream. Such a design table for one subsection of the drainage system is shown in Table 1.
### Table 1
**Typical Design Table for Drainage System**

See Fig. 20, Drainage System K, Idlewild Airport

<table>
<thead>
<tr>
<th>Line</th>
<th>M.H. or C.B.</th>
<th>Length</th>
<th>Area</th>
<th>Total Area</th>
<th>Gutter Length</th>
<th>$\Delta$</th>
<th>$\Delta^2$</th>
<th>0.9$\Delta^2$</th>
<th>Pipe Size</th>
<th>Hyd. Slope</th>
<th>Vel.</th>
<th>El. Hyd. Grade</th>
<th>Q</th>
</tr>
</thead>
<tbody>
<tr>
<td>152</td>
<td>46-47</td>
<td>150</td>
<td>1.62</td>
<td>0.73</td>
<td>0.73</td>
<td>0.58</td>
<td>6.03</td>
<td>18</td>
<td>0.00222</td>
<td>3.41</td>
<td>11.50</td>
<td>11.02</td>
<td>6.35</td>
</tr>
<tr>
<td>153</td>
<td>46-47</td>
<td>150</td>
<td>0.99</td>
<td>200</td>
<td>1.85</td>
<td>2.58</td>
<td>2.06</td>
<td>11.92</td>
<td>0.00275</td>
<td>3.70</td>
<td>11.02</td>
<td>9.86</td>
<td>12.55</td>
</tr>
<tr>
<td>154</td>
<td>47-48</td>
<td>420</td>
<td>3.21</td>
<td>1.04</td>
<td>1.04</td>
<td>1.04</td>
<td>1.04</td>
<td>1.04</td>
<td>0.00325</td>
<td>3.43</td>
<td>11.50</td>
<td>10.67</td>
<td>6.38</td>
</tr>
<tr>
<td>155</td>
<td>47-48</td>
<td>420</td>
<td>0.62</td>
<td>340</td>
<td>1.24</td>
<td>1.24</td>
<td>0.99</td>
<td>6.06</td>
<td>15</td>
<td>0.0055</td>
<td>3.90</td>
<td>11.50</td>
<td>10.95</td>
</tr>
<tr>
<td>156</td>
<td>47-48</td>
<td>420</td>
<td>0.50</td>
<td>140</td>
<td>0.43</td>
<td>0.43</td>
<td>0.34</td>
<td>4.79</td>
<td>15</td>
<td>0.0055</td>
<td>3.90</td>
<td>11.50</td>
<td>10.97</td>
</tr>
<tr>
<td>157</td>
<td>47-48</td>
<td>420</td>
<td>0.19</td>
<td>140</td>
<td>1.37</td>
<td>1.37</td>
<td>0.87</td>
<td>3.06</td>
<td>15</td>
<td>0.0055</td>
<td>3.90</td>
<td>11.50</td>
<td>10.97</td>
</tr>
<tr>
<td>158</td>
<td>48-49</td>
<td>170</td>
<td>6.14</td>
<td>0.62</td>
<td>3.20</td>
<td>2.56</td>
<td>22.44</td>
<td>30</td>
<td>0.0030</td>
<td>4.58</td>
<td>9.86</td>
<td>9.35</td>
<td>23.62</td>
</tr>
<tr>
<td>159</td>
<td>48-49</td>
<td>170</td>
<td>0.70</td>
<td>360</td>
<td>1.20</td>
<td>1.20</td>
<td>1.20</td>
<td>1.20</td>
<td>0.0030</td>
<td>4.58</td>
<td>9.86</td>
<td>9.35</td>
<td>23.62</td>
</tr>
<tr>
<td>160</td>
<td>48-49</td>
<td>170</td>
<td>0.19</td>
<td>140</td>
<td>0.43</td>
<td>0.43</td>
<td>0.34</td>
<td>4.79</td>
<td>15</td>
<td>0.0055</td>
<td>3.90</td>
<td>11.50</td>
<td>10.97</td>
</tr>
<tr>
<td>161</td>
<td>48-49</td>
<td>170</td>
<td>0.19</td>
<td>140</td>
<td>1.37</td>
<td>1.37</td>
<td>0.87</td>
<td>3.06</td>
<td>15</td>
<td>0.0055</td>
<td>3.90</td>
<td>11.50</td>
<td>10.97</td>
</tr>
<tr>
<td>162</td>
<td>49-50</td>
<td>100</td>
<td>1.37</td>
<td>0.43</td>
<td>0.43</td>
<td>0.34</td>
<td>4.79</td>
<td>15</td>
<td>0.0055</td>
<td>3.90</td>
<td>11.50</td>
<td>10.92</td>
<td>5.04</td>
</tr>
</tbody>
</table>

The plan shown on Fig. 20 was at one time tentatively adopted and the drainage was partly designed. Later this plan was abandoned and the runway pattern materially revised.

### Table 1 (Concluded)
**Typical Design Table for Drainage System**

<table>
<thead>
<tr>
<th>Line</th>
<th>Time in Minutes From Beginning of Rain</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>23.0</td>
<td>23.5</td>
</tr>
<tr>
<td>152</td>
<td>6.26</td>
<td>6.29</td>
</tr>
<tr>
<td>153</td>
<td>6.22</td>
<td>6.26</td>
</tr>
<tr>
<td>154</td>
<td>3.75</td>
<td>3.78</td>
</tr>
<tr>
<td>155</td>
<td>0.79</td>
<td>0.79</td>
</tr>
<tr>
<td>156</td>
<td>1.97</td>
<td>1.57</td>
</tr>
<tr>
<td>159</td>
<td>1.61</td>
<td>1.62</td>
</tr>
<tr>
<td>160</td>
<td>0.63</td>
<td>0.63</td>
</tr>
<tr>
<td>161</td>
<td>4.10</td>
<td>4.10</td>
</tr>
<tr>
<td>163</td>
<td>0.27</td>
<td>0.31</td>
</tr>
<tr>
<td>164</td>
<td>1.97</td>
<td>2.01</td>
</tr>
<tr>
<td>165</td>
<td>1.97</td>
<td>1.97</td>
</tr>
<tr>
<td>166</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td>168</td>
<td>22.97</td>
<td>23.16</td>
</tr>
<tr>
<td>169</td>
<td>22.71</td>
<td>22.87</td>
</tr>
<tr>
<td>170</td>
<td>2.23</td>
<td>2.27</td>
</tr>
<tr>
<td>171</td>
<td>1.86</td>
<td>1.89</td>
</tr>
<tr>
<td>172</td>
<td>4.87</td>
<td>4.91</td>
</tr>
<tr>
<td>173</td>
<td>4.81</td>
<td>4.87</td>
</tr>
<tr>
<td>174</td>
<td>4.81</td>
<td>4.87</td>
</tr>
</tbody>
</table>

Note: The original table shows calculations for each half minute of time from 22 minutes to 30.5 minutes. Only 6 time columns from the central part of the original table are reproduced here for illustrative purposes.

*Length of gutter.*

†Design figures used for pipe design calculations.
It would seem that this procedure, while involving a little more arithmetic than the application of the rational method, can be carried out rapidly once the inlet inflow hydrographs have been prepared. The method involves no coefficients of runoff, rests entirely upon the application of hydraulics, and unquestionably gives an accurate basis of design.

While in this instance all the runoff was from paved surfaces, and infiltration capacities are not involved, the method is equally applicable to the determination of runoff from combinations of pavement and turf, once a satisfactory series of inflow hydrographs have been prepared.

It is interesting to compare the results of this design with that for the Washington National Airport. The rainfall rates are not appreciably different, the runway grades are equally flat, and the grades of the sewers are on about the same order. An inspection of corresponding systems indicates that the storm sewers for Idlewild Airport are about the same size as those for Washington National, indicating that the extra width of pavement (300 feet) at Idlewild has been compared with that at Washington National (200 feet) approximately offsets the fact that at Idlewild no drainage capacity is provided for the inter-field spaces.

This of course is not the whole story, as the rate-reducing effect on the turfed gutter at Washington is tremendously greater than that of the paved gutter at Idlewild.

With respect to this project it should be noted again that the storage which is active in reducing runoff rates is natural storage, that is, there is no “forced pondage” on the surface.

14. Drainage Design for Army Fields.—Under pressure of increasing defense need, in connection with the construction of Army airfields, the preparation of a manual of airport construction was undertaken by the Office of the Chief of Engineers early in 1941. The development of the engineering section on airport drainage was carried out by a staff of hydraulic and hydrologic engineers under the supervision of Mr. Gail A. Hathaway. The resulting procedure was issued first as an Engineer Bulletin in June of 1941, and later was subjected to several revisions as Chapter XXI of the Engineering Manual. The substance of the procedure was contained in a paper which Mr. Hathaway presented to the American Society of Civil Engineers in January of 1943. The procedure embodied in the Manual followed in many respects that which was developed for the Washington National Airport and included in the consultant engineer’s report of August 1940. For example, it made a similar use of rainfall rate curves and of the hydrograph of overland flow for paved surfaces and turfed surfaces; it included the use of supply curves for pervious areas prepared by deducting infiltration rates from rainfall rates. In the original draft the design procedure required the application through successive trials to determine the critical supply rates. The original draft contained the following statement:

Two practical methods may be employed to reduce the peak rate of runoff from an area:

(a) The distance of flow over turf may be increased, thus taking advantage of the retarding effect of the turf; and

(b) The capacity of inlet ports may be restricted to cause temporary ponding of runoff in excess of a specific rate.

There then followed a discussion to the effect that unless the stand of turf was well maintained, gullying might occur and storage be materially less than expected. Under the second alternative, attention was called to the danger of clogging of restricted inlet ports, but it was suggested that partial clogging occasionally might not constitute a serious problem.

In the later edition of the Engineering Manual, design procedure is related almost entirely to forced pondage produced, not by restricted inlet ports, but by restricted sewer capacity. This has been the basis of design of most of the military airfields, and is apparently satisfactory for that purpose. It seems preferable not to use it in connection with civil airports unless it is reasonably certain that a good stand of turf could not be maintained. Turf produces a great rate-reducing effect on flow over turf, particularly on very flat slopes. In general, the capacity of inlets and the detailed sloping plan can be worked out without forced pondage so as to get a large measure of storage reduction.

The procedure outlined in the Engineering Manual was naturally devised in the interest of shortening design work, and involved a number of approximations which should be used with considerable discretion. For example, the supply curve for a sub-area containing both pervious and impervious surfaces is obtained by weighting the respective rainfall and the rainfall minus infiltration curves in proportion to the areas, somewhat as was done in the old rational method. This gives a representation of water application which is neither the correct one for the pervious nor for the impervious sections.

The method further largely neglects the surface detention on paved surfaces, and the time of flow on paved surfaces, as being small in comparison with similar values on turf. However, it provides for the use of overland flow curves having a length of only the flow over turf and
uses a roughness coefficient that is a weighted value between that for pavement and that for turf. This procedure might give nearly the correct surface detention in some one special situation, but widely incorrect results in others. It apparently gives a satisfactory approximate value where the inlets are located at a considerable distance from the nearest part of the paved area.

15. Inlet Spacing and Field Shaping.—Probably the most important result that has come out of the experience in designing a large number of military fields has been the knowledge that the drainage system can be greatly reduced in capacity by shaping the field into properly devised inlet areas with flat summits between them.

If the maximum reducing effect is to be secured from natural surface detention out of overland flow and through grassed gutters, the inlets should be spaced as widely as possible. On very flat grades it has been found that natural storage in grassed gutters of 200 to 250 feet in length may involve depths of from 3 to 5 inches without using restricted openings or arbitrarily restricted sewer capacity.

Where storage reduction is to be secured by forced pondage a somewhat closer inlet spacing is desirable in order to divide the storage ponds into smaller units, and also to make it possible to install dividing ridges where the cross slope of the field may be 1 per cent or even greater.

In the earlier days of airport drainage there was a general conviction that the drainage system should remove water, if possible, as fast as it fell. Today it is recognized that water on the surface for periods of several hours need not be antagonistic to satisfactory operation of the fields, but the depths of standing water, whether produced by forced pondage or by natural detention storage, should preferably be kept under 6 inches for civil airports, and under 12 inches for training fields. Also, the margin of such storage or pondage areas should preferably be kept 50 feet or more from the edge of the nearest paved surface in order to avoid anything approaching saturation of sub-grade.

Ten years ago engineers could find no well-defined precedent for design of drainage for airfields, and no very definite criteria as to the conditions that must be met. During the past five years engineering techniques have been greatly advanced and the objectives of the drainage system fairly well agreed upon. Of course, careful note will have to be made of further changes in aeronautical design, and objectives will have to be modified from time to time as newer types of airplanes come into use.