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A Stormwater Overflow Control Device

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BY

RICHARD DUKE STALKER
BASE, University of Florida, 1965

RESEARCH REPORT

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ABSTRACT

A STORMWATER OVERFLOW CONTROL DEVICE

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On Lake Eola, stormwater runoff has been identified as a major source of pollution. Other lakes in Central Florida are experiencing similar decay due to stormwater runoff. A device has been examined for diversion of the initial flows to treatment before discharge into the lake. A graphical aid was developed to select the proper volume required for the device and was applied to a Lake Eola existing collection basin. A laboratory model was designed and constructed based on the scaled-down version of a collecting basin on Lake Eola. This model was used to demonstrate the concept, as well as, indicate the effects of several critical design variables. Recommendations on design for a Lake Eola device were made for possible improvements in the system itself.

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INTRODUCTION

Lakes are transitory water holding basins on the earth's landscape. Natural processes tend to destoy lakes and man's actions help accelerate their destruction. Eutrophication is the term given to the build up of nutrients and impurities mainly caused by the combination of natural changes and man-made changes in the quality of our lakes. Sources of our lake problems come from man taking for granted the environment in which he lives.

One of the most obvious sources of lake eutrophication is the stormwater runoff from rainfall being directed into lakes. This source of pollution is only one of many important sources; however, it is a major source where the results can be studied and documented to indicate its degree of influence. This particular type of study is taking place currently on Lake Eola in Orlando, Florida. This lake has just been restored and is currently under a post-restoration monitoring phase. During the pre-restoration investigation of the lake, it was determined that stormwater runoff from the streets were draining directly into the land-locked lake depositing debris, silt, leaves, and any other materials transportable by stormwater runoff. This caused a slow decomposition and collection of undesireable material on its bottom indicating that stormwater is a significant source of pollution.

The objective of this report is to pursue research for the development of a stormwater control device that will divert the

initial few minutes of stormwater to a treatment facility and to allow the remainder to discharge into a lake for recharge purposes. This device will be primarily examined in context to the modification of an existing collection system that presently affects Lake Eola. The control device should be capable of being utilized for stormwater control in areas that exhibit problems similar to those on Lake Eola.

I. BACKGROUND

Although lakes are temporary features of the landscape, they are vital to our nation's water use and management. It has been estimated that about three fourths of the water used by man comes from surface water which includes lakes. Many irrigation and large power withdrawals depend on lakes. Municipalities and industries depend on standing water as a direct source or upstream sources. Reservoirs and natural lakes throughout the United States fulfill these requirements. Lakes are one of the most dependable water supplies. Water shed areas with numerous lakes usually have moderate stream flow fluctuations. Our population lives in contact with fresh surface water sources of which 90 percent by area are lakes. (1)

Our lakes have been deteriorating not only because of the natural aging of lakes but also because of the continuous and increasing quantities of pollution from the activities of man. Run-off from mismanaged land developments carries large quantities of sediments and nutrient rich top soil and fertilizer into nearby lakes. Some areas use lakes as a community dump. Wastewater treatment plants often contribute to eutrophication problems. Terrestrial deposit of waste materials can affect a lake where drainage from dumps and our many streets transport debris, vegetation, undesirable nutrients and bacteria to water sources. Industrial plant locations can create many problems because waste from some industrial sites are direct sources of nutrients.

Review of Lake Restoration Methods

When one looks to restoration of a lake, it is generally approached either to reduce the nutrient concentration of the lake below the level causing overproductive growth or to improve the water quality in order to restore certain recreational or esthetic uses. These conditions can be met by either direct treatment of the water or indirectly by controlling the bottom sediments and the growth of aquatic plants. (2)

One direct method would be to prevent pollutant discharge into lakes by proper land zoning and rigidly enforced regulations on land development. It is obvious with this type of treatment, pollution abatement, may not improve the water quality; however, further deterioration is reduced and a possible nutrient balance created. The extent of improvement would be determined by the rate at which the water body is being replaced with new water and the quality of the influent water. A second direct method to improve the water quality of a lake is to replace the existing water with higher quality water. Replacement can be by introducing high quality water directly into the water and displacing an equal volume of low quality water, or by first draining a given volume of the lake and refilling it with higher quality water. The latter method is preferred since it allows for less dilution due to mixing, thus achieving a higher water quality per unit volume of water. However, this is generally the more expensive technique.

The best direct method to be examined is water treatment by external treatment processes. In this process, any degree of treatment can be achieved depending on the result required. Adequate

improvement can be obtained by just chemical coagulation following plain sedimentation. Such treatment could remove large amounts of color, turbidity, phosphorous, algae, bacteria, and some dissolved organic impurities. Higher degree of treatment could be utilized by using granular filtration and activated carbon absorption, however, the costs jump rapidly.

The indirect method of controlling bottom sediments which can contain large quantities of nutrients are divided into three techniques, they are:

1. Sediment covering - using liner type materials, such as, polyethylene sheets and particulate materials (clay, fly ash and sand).

2. Dredging - although dredging presents a permanent means of removing sediments from lakes, the environmental impact of the operation could be significant.

3. Oxygenation - based on the fact that pollutional releases from the bottom sediments are much greater under anaerobic conditions than under aerobic conditions.

Finally, techniques are available for control of aquatic plants which can be considered as indirect methods since the aquatic plants contribute to bottom sediments, they are:

1. Chemical - addition of chemical compounds to lakes.

2. Mechanical - harvesting aquatic plants.

3. Biological - stocking lake with cray fish, manates, freshwater snails, and white amur

4. Physical - draw down water and expose bottom to sunlight.

Proposed Method

The method developed is to control stormwater runoff so that the drainage water discharged to the receiving lake has a minimal amount of impurities. In evaluating a water sample for its pollutional

content the parameters examined are divided into six basic categories: nutrients, biological, dissolved substances, suspended materials, microbiological and physical. (3) The nutrient parameters include phosphates (Ortho, poly and total), nitrates, nitrites, ammonia, nitrogen and organic carbon. The biological parameters include such things as algae count and identification and primary productivity. Dissolved substances include dissolved solids, chlorides, hardness, sodium, calcium, potassium, iron, etc. Suspended solids include such items as turbidity, volatile suspended solids, suspended solids and total solids. Micro-biological contains items such as human pathogen analysis, sediment studies, fecal and total coliforms and anaerobic plate count. Finally, the physical category includes such items as temperature, water level, rainfall and flow rate of discharges. In addition to these, BOD (Biochemical Oxygen Demand), (TOC) Total Organic Carbon, dissolved oxygen (DO) and pH are equally important parameters in measuring the pollutional content of a lake.

The method proposed here is a modification of an existing drainage collection basin presently in use on Lake Eola. It allows stormwater from the streets and adjacent areas to enter from a curb side drain which is the input to the stormwater overflow device. The initial flow which is usually heavily polluted is directed into either a sanitary sewer line or a small on site treatment facility for treatment before returning to a lake for recharge purposes. The selection of one or a combination of the two is beyond the scope of this report, but would be based on an economic present worth analysis. Since the flow into the drainage basin during heavy or peak flows will be greater than the amount being sent into the treatment line, the

stormwater will tend to build up in the drainage basin until a height is reached equal to the height of the baffle wall. When this level is exceeded, the cleaner top surface of the stormwater will spill over into the second compartment which will then be discharged into the lake by the natural action of differences of head. After a storm is over, any stormwater remaining in the first compartment will be drained into the treatment line thus removing most of the water from this compartment and leaving only the amount of water equal to the level of the lake in the second compartment. A flapper valve is also provided at the outputs of both the line going into the treatment line and the one into the lake itself. The first valve prevents the sanitary sewer from backing up into the drainage basin when the line becomes full. The second valve prevents water from entering the basin if the level of the lake is higher than the basin itself which would occur, for instance, if flood conditions prevailed.

This particular method could be categorized as a combination of two of the direct methods previously described. In essence, it works on the replacement principle by removing the initial pollutant load which is greatest during the first few minutes of a storm and allowing the cleaner water to overflow into the lake. It also satisfies the direct method of water treatment by external treatment processes by allowing the water to enter either a sanitary sewer or an on-site treatment facility. One additional technique is utilized: that of the elimination of pollutants entering the water from controllable sources.

The catalyst for this report has been the recent Lake Eola restoration program for which the lake is under the post-restoration

monitoring phase. It is during this time that all the control measures implemented are monitored and further possible improvements examined. In recent years, obvious deterioration of the lake was observed until recent restoration plan was undertaken. The drawdown method was used to lower the level of the lake to expose the bottom sediment for eventual removal to restore good quality water to the lake. The storm sewer system during this time underwent some changes by the addition of some stormwater drainage basins. Lake Eola is a land-locked lake receiving stormwater runoff through storm sewers serving approximately 350 acres of the surrounding urban area. There are at least twenty-two (22) storm sewer pipes emptying into the lake. (4) The lake contains about 100 million gallons of water with a surface area of 28.5 acres and its level is controlled by three drainage deep wells. Runoff and dry weather flows from the street drains vary in both quantity and quality. Prior to the lakes drawdown, build up of sand and muck deposits around the outlets into the lake were visibly noticeable from the shores bank.

Water quality data were collected by the Orange County Pollution Control Department and FTU. Most of the data indicated that the stormwater from the storm sewers entering the lake were the most probable source of silt, sand, and vegetation fround in the lake. (5)

Control of only the point sources of pollutional discharge will not necessarily result in restoring the Lake to the desired level. (6) However, once these point sources are isolated and identified, control measures could be implemented such that these sources would have minimal effect and the water quality of the lake

remain status quo. This assumes that the natural biochemical action in the lake is adequate to handle the impurities and nutrients being grudgingly discharged into the lake.

When rain starts, the solids close to the curb will immediately start a transport process to the nearest drain intake which eventually enters the stormwater catch basin. The transported material will depend on such parameters as street surfacing material, rainfall intensity, time of last rain, size of particles and street condition. It has been previously determined that the transport of pollutants follow an exponential function which indicates that most of the impurities from the street are transported during early part of a storm. (5)

II. STORMWATER CONTROL

Review of Regulating Devices

Basically three types of regulating devices are in use today, the mechanical automatic regulator, overflow devices, and fixed orifice structures. (7)

The mechanical regulator's function is usually in sewage flow to prevent surcharge of an intercepting sewer by closing an automatic gate on one line thus cutting off the flow and forcing it to flow to another outlet. However, it has a second function, that of regulating the flow of a storm drain so that discharge during a storm from one storm sewer carrying a heavily polluted storm flow may be directed into an interceptor in a greater proportion than that from another sewer carrying a more dilute storm.

The automatic mechanical regulators of either the float operated or the flapper-valve type are considered impractical from the standpoint of small flows because of the small waterways and the consequent possibility of clogging. Due to the size of structure necessary and the high cost of equipment, mechanical regulators are not considered economically justified for flows under 2 cfs or 1.3 mgd. Of equal importance, all these devices require periodic inspection, careful maintenance and careful adjustments by technically knowledgeable workers.

Secondly, stormwater overflows are widely used as a means of reducing the additional load imposed on the treatment facility. They

are designed to allow the excess sewage flow above a definite rate to escape from one sewer in which its flowing over a weir into a secondary sewer. Storm overflows are classified into the following types: side-flow weirs, baffled weirs, transverse weirs, leaping weirs, and siphon spill ways.

Side weirs are commonly used to bypass excess storm flows in combined sewer systems. They can be simply visualized by considering a section of pipe cut away of which the length and height of weir must be determined for the particular application intended. Baffled side weirs are ones in which a baffle has been placed across the main channel for the purpose of increasing the capacity of the side weir. Transverse weirs removes the uncertainty in the use of side weirs by placing the weir directly across the path of flow and (deflecting) the sewer to one side, thus reversing the arrangement used for side weirs. Leaping weirs are formed by a gap in the invert of a pipe or sewer so that the standard flow drops through the gap and passes to the interceptor. At times of increased flow, the increased velocity causes most of the flow to leap the gap and pass into a second outlet. Siphon spill ways have the advantage of providing a means of regulating the water surface elevation in a sewer with small variation in high-water level than can be obtained with the other devices. By utilizing all the available head the siphon discharges at a higher velocity than overflow weirs. They work automatically without any mechanical devices. However, they are relatively new and information on the operation and design is limited. Another effect is the possible noise and vibration from the sudden starting and stopping of the siphon. The basic drawback of all the above mentioned storm overflow

methods/devices is their inefficient operation in preventing the escape of solids to the bypass flow. A final and rather unique overflow regulating device involves the use of a computer for monitoring and remote control of inflatable dams placed inside the sewer. It can also be controlled manually, but has the unique feature of being able to regulate the height of a weir or dam. The obvious drawback to this would be the cost on a small scale basis and only becomes justifiable in major sewer systems. (8)

Thirdly, fixed orifice structures employ a fixed diameter orifice for the purpose of diverting relatively small rates of dry weather sewage from an existing combined sewer into an interceptor and during peak load flows due to storms limits the discharge of stormwater to the interceptor without the use of mechanical regulators. In operation, the hydraulic gradients in the various diversion chambers will rise; and the total discharge of each sewer will be divided between the stream and interceptor in proportion to the orifice size and outlet size. The orifice flow is dependent on the height or level of the water in the chamber, as well as, the orifice diameter.

Lake Eola Application

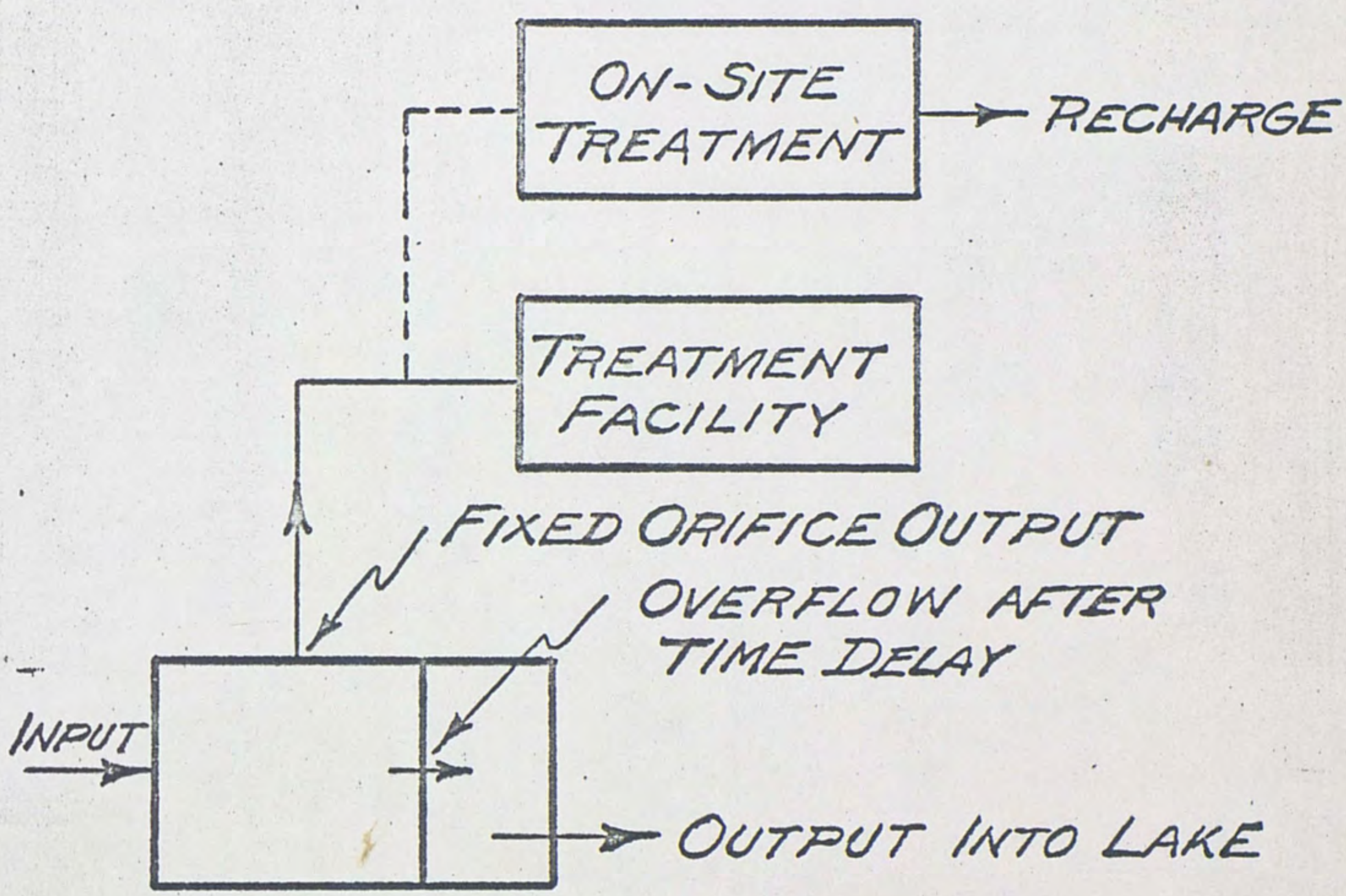
Research and study conducted by the American Public Works Association for the Federal Pollution Control Administration determined that street refuse and litter has contributed to the pollutional strength of storm runoff. (9) The report separated street wase loadings into pollutional parameters such as BOD, COD, nitrogen, phosphate, total bacteria, caliform bacteria, and enterococci. It was concluded that the pollutional load of the first flush of storm was much stronger than normal sewage, because it contained more

organic matter. Therefore, if stormwater from the streets is allowed to run freely into lakes or streams without control, these bodies of water will eventually show signs of eutrophication.

Lake Eola, located in downtown Orlando, Florida, is a lake that recently has been restored. Lake Eola is a land-locked lake which receives stormwater runoff directly through storm sewers serving approximately 350 acres of the surrounding city from about twenty-two (22) storm sewer pipes located around the lake. The problem arose during the restoration of Lake Eola of what type of solution can be utilized to control the inevitable repollution of the lake without some sort of stormwater control. During the drawdown some changes of the storm sewer system were made mainly screening devices placed at the outlets to prevent large street solids, leaves, and litter from entering into the lake. However, the high initial pollutional load still remains and at present is allowed to enter the lake with only screening devices. A diversion technique for controlling the initial flush of stormwater from entering the lake would be beneficial.

Principle of Operation

The method uses the combination of fixed orifice and weir to accomplsih the intended diversion of initial storm flow. The functional block diagram, as shown in Figure II-1, is a pictorial representation of how the process works. As stormwater enters the collection basin, flow will immediately begin to exit through the orifice. As the quantity of stormwater increases, the water will raise above the orifice to the height of the weir where, if the flow is great enough, it will overflow and enter the lake. The time it takes to reach



Functional Block Diagram of Stormwater Overflow Device

FIGURE II-1

this height should be sufficient to handle the initial flush of stormwater. This time has generally been given as 30 minutes, however, it is variable with intensity of the storm. The water level in the basin regulates itself until it reaches the level in the Lake minus any minor head losses. During stormwater flow periods, the water in the basin, builds up forcing the stale water (low dissolved oxygen and high in BOD) into the Lake along with most of the dissolved impurities. The proposed method will prevent the surge of stale water from entering the Lake by diverting initial storm water flows to a sanitary treatment plant. In the existing collection basin, during dry weather periods, water sits relatively stagnant at the level of the Lake. During this period the water loses most or all of its dissolved oxygen creating an anerobic condition which if allowed to remain long enough would emit an objectionable odor. However, in the proposed method there will be two chambers being separated by a baffle wall which acts as a weir. Only in the front chamber will water be allowed to sit at the level of the Lake. In the rear chamber, controlled by the orifice, water will be drained completely from the basin into the treatment line. This allows greater utilization of the basin by providing a greater volume for collection than the existing collection basin which remains half or three-quarters full all the time. Due to this fact, the existing basin won't hold the normal stormwater runoff without overflowing through the access covers. The proposed method greatly reduces this possibility by simply providing the entire volume except for volume in the forward chamber for collection while at the same time directing the flow to a sanitary treatment facility. Also, during low flow or dry weather

flows, all of the stormwater will be diverted to the sanitary sewer. Another advantage with this method besides the ones already mentioned is the fact that there are no moving parts. However, a flapper valve is utilized at the output of the orifice as a precautionary measure to prevent the treatment line from backing up into the basin and ultimately into the lake. Continued screening must be applied and regular clean out of the basin performed. Clean out of the proposed basin will be easier than the existing, since the chamber to be cleaned out will be dry, whereas, the other, was half or better filled with stale water.

There are two fundamental cases that must be considered in examining this device. The first case is where Q(in) is less than or equal to Q(orifice), where Q(orifice) is defined as the flow through the orifice at a head equal to the height of the weir. [Q(in) $\leq$ Q(orifice)]. In this situation, the flow in the orifice and chamber will obtain a certain level in the chamber where the head reaches the necessary height to allow the flows through the orifice to equal the inlet flow. This head height will also be less than the baffle weir height. Therefore, $\int_0^t \delta(Q(in) - Q(orifice)) = f(h)\delta t$ where: h is a vector of variables such as, head and surface area, but at a certain time, t, it will reach a maximum and will remain constant at that value as long as Q(in) remains constant.

The second case is where Q(in) is greater than Q(orifice) [Q(in) $\geq$ Q(orifice)]. In this case, the flow rate into the orifice chamber is greater than the flow rate out of the orifice at its maximum allowed head which is the height of the baffle weir. Thus, the equation obtained is:

$$\int_0^t \delta(Q(in) - Q(orifice)) = f(\phi)\delta t$$

where: ϕ is a vector of variables which now include weir configuration, sharpness of weir edge, roughness of weirs crest, solids, etc., as well as, head and surface area. Now, the condition exists where the storage capacity has been exceeded and overflow occurs. The time it takes to exceed this storage volume is t but depends on the allowed head and surface area (volume).

The first point to be considered is the amount of flow that is involved. In estimating stormwater quantities, it must be remembered that flow into stormwater conduits is mainly by gravity from collecting surfaces of various characteristics such as rough or smooth, pervious or impervious, through swales and gutters, into inlets and finally into the collecting basin. The rate of stormwater runoff to be used in the design of the collecting basin is difficult to evaluate. The approach to be taken here is that of computing runoff as related to rainfall through a proportionality factor as applied in the rational method. For the rational method, the frequency of the stormwater runoff is assumed to be equal to that of the frequency of the average rainfall intensity selected. Experience has shown this method to yield satisfactory results for relatively small areas. The rational method is based on three assumptions. (10) First, the peak rate of runoff at any point is a direct function of the average rainfall intensity during the time of concentration to that point. Second, the frequency of the peak discharge is the same as the frequency of the average rainfall intensity. Third, the time of concentration is the time required for the runoff to become established and flow from the most remote part of the

drainage area to the point under design.

Therefore, in applying the rational method to the Lake Eola area by the formula, $Q = CiA$ in which Q is the peak runoff rate in CFS, C is a runoff coefficient dependent on characteristics of the drainage area, i is the average rainfall intensity in in/hr. and A is the drainage area in acres. The intensity (i) to be used is equal to .25 inches/hr. which was found to be the intensity occurring 75 percent of the time in Orlando. (11) The area is 350 acres of which 22 storm sewer pipes empty into the lake. For illustrative purposes, assume that the drainage basin for one storm sewer is 20 acres. The runoff coefficient (c) used is equal to 0.90, the range for asphalt and concrete was in a range between 0.70 - 0.95 since only street runoff is considered. (10) Therefore, Q can now be estimated; $Q = 0.90 \times 0.25$ in/hr $\times$ 20 acres = 4.5 CFS, which represents the amount of input into the collection basin.

It is necessary to determine the volume to store this quantity of water for a recommended thirty minutes.

$$V = Qt$$

where: Q = input flow in CFS

t = time seconds

V = storage volume

The volume is: $V = 4.5$ CFS (30 min.) (60 sec/min) = 8100 Ft^3

This only determines the amount of storage space required to collect that amount of water over a thirty (30) minute interval.

Now the next step is to determine the amount of water removed by a specific diameter orifice at a specified head. The general equation for flow through an orifice as presented in The Civil

Engineering Handbook, (12) is:

$$Q = Ca\sqrt{2gh}$$

where: Q = discharged flow through orifice (CFS)

C = coefficient of discharge (Dimensionless)

a = area of orifice (Ft^2)

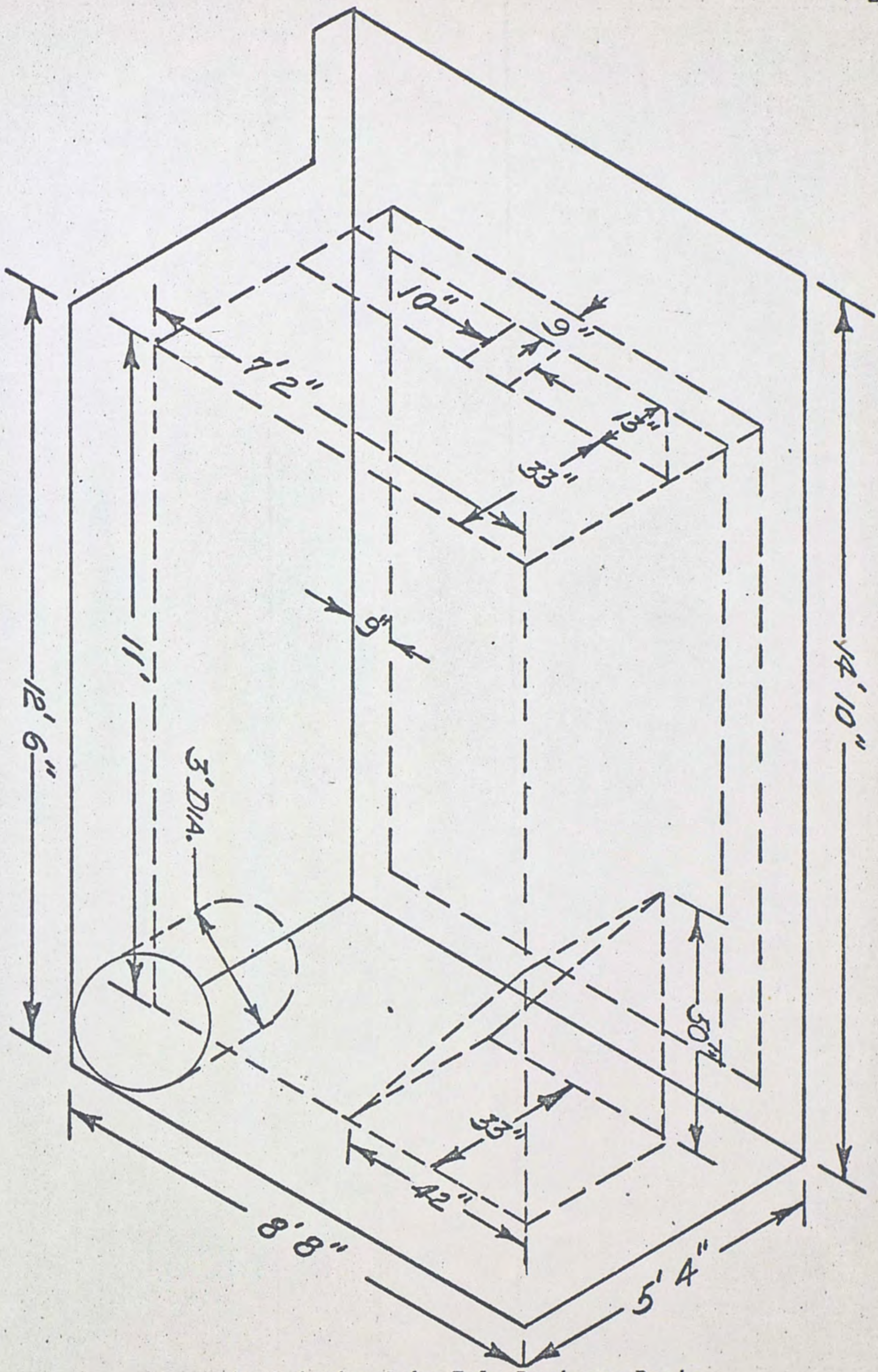
g = gravitational constant (Ft/sec^2)

h = head or height of fluid (Ft)

The selection of the proper coefficient is important since it is the product of the coefficient of velocity (C_v) and coefficient of contraction (C_c). The effect of water temperature causes the viscosity or density to vary which affects the coefficient of discharge as does velocity and the orifice diameter. For this reason, the Reynolds number is used as the coordinating factor which will show the effect on "C" of the four factors mentioned. This has been demonstrated by Lea (13) and further varified by Medaugh and Johnson (14). Thus, one can determine the value for C from a plot of $\frac{d\sqrt{gh}}{\mu}\rho$ versus coefficient of discharge "C". By knowing the "C" value then the diameter can be determined that is necessary to create a steady state flow condition for any specified height of weir by setting the Q(in) equal to Q(orifice) (Q(in) = Q(orifice)). Performing this calculation for the above conditions with a head of three (3) feet represents a diameter equal to 9.97 inches. This would allow for a storm of 4.5 CFS to build up in the storm collection basin to a height of 3 feet. This determines the maximum level the incoming stormwater will obtain at that steady state condition. However, this condition will exist given any surface area with the same height only the time to reach that height will vary. Therefore, it is necessary to

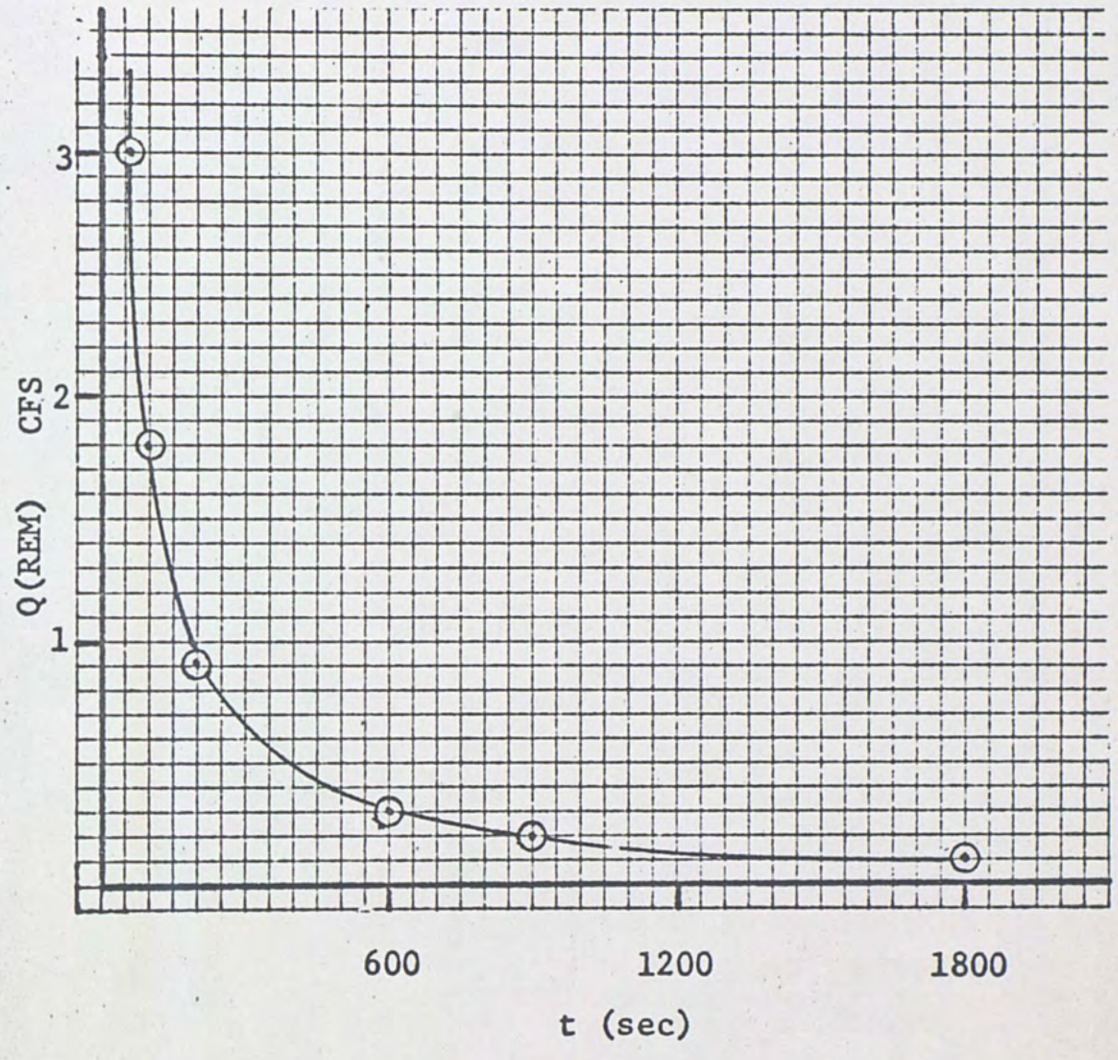
determine the surface area required to restrain a flow that just exceeds this steady state flow for the recommended time of 30 minutes. Thus, knowing that Q(in) - Q(out) = Q(remaining) in the collecting basin a flow equivalent to obtaining a Q(remaining) of 0.1 CFS was used to obtain the volume of the collection basin to restrain the design flow for thirty minutes before overflowing the weir. From the equation $V = Qt$, the volume was found to be 180 Ft^3 and by dividing by the height of head (3 ft) obtained a surface area of 60 Ft^2. Applying this to the existing basin on Lake Eola, which has the dimensions as shown in Figure II-2 of 11 ft. X 7.167 ft. X 3.833 ft., the three (3 Ft) weir or baffle would be placed $L = \dfrac{180\ Ft^3}{3\ ft \times 7.167\ ft} =$ 8.4 ft from inlet, or $\dfrac{8.4}{11} \times 100 = 76\%$ of the existing surface area.

The effect of increasing the Q(remaining) as shown in Figure II-3 indicates a rapid reduction in detention time. However, this is based on the assumption of having the peak flow immediately, where in actuality the storm runoff will build up gradually depending on the intensity of storm, imperviousness of area and the acreage served. This effect would therefore tend to increase the detention time even longer than that present here. Figure II-4 illustrates the effect that this device is trying to achieve. For a low intensity storm, the peak time of flow will be reached at a much greater time than a high intensity storm; however, the pollutional effects of the low intensity storm usually will be spread out over a longer period of time relative to a high intensity storm. If flow over the weir can be diverted or retained long enough so that in both the low intensity and high intensity storm the peak BOD loading has been passed, the



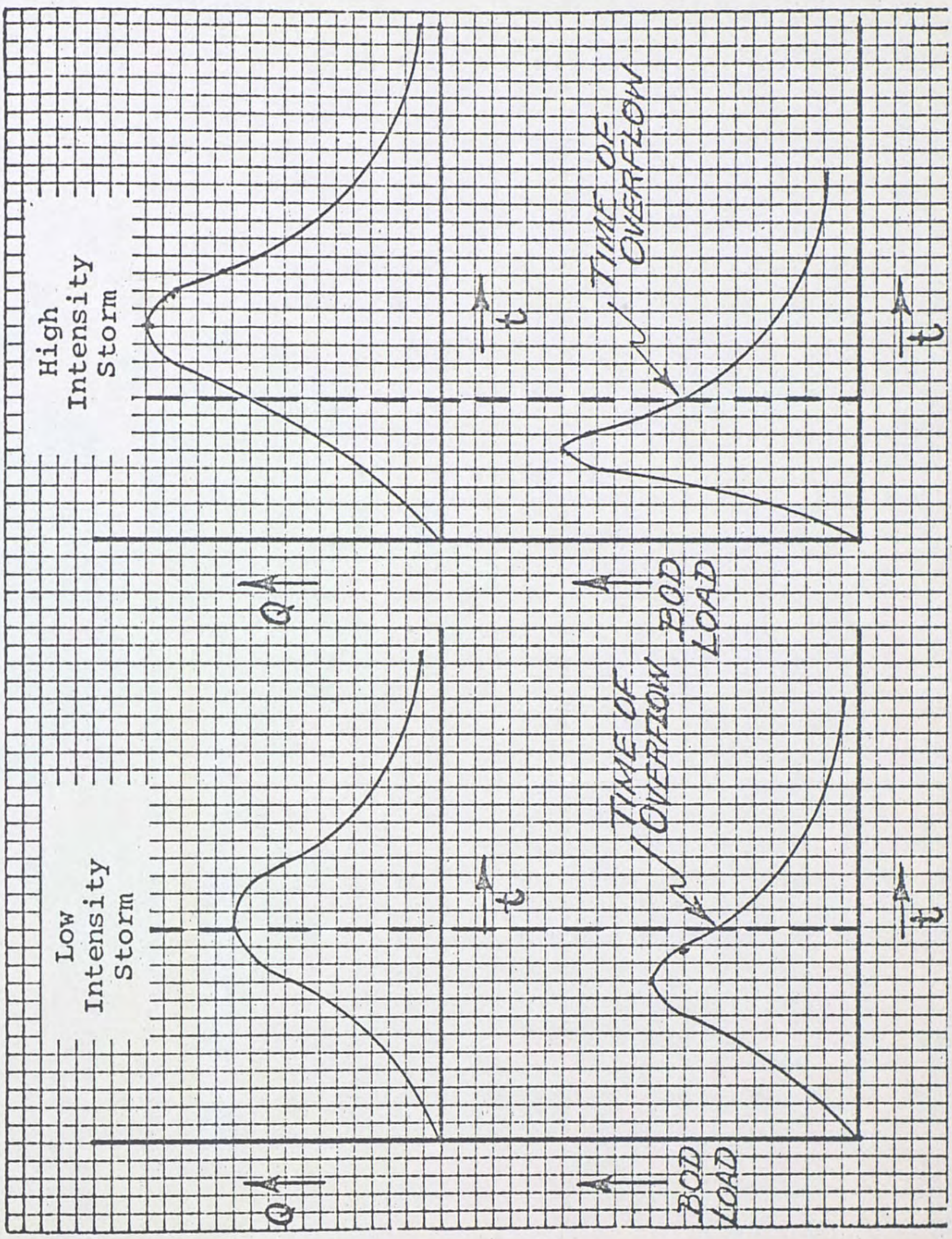
An Existing Lake Eola Drainage Basin

FIGURE II-2



Detention Time Versus Q(remaining) For Fixed Design Volume and Orifice Diameter for Steady State Condition at 4.5 CFS.

FIGURE II-3



Comparison of Intensity With B.O.D. Loading Versus Time
To Demonstrate The Storm Water Overflow Device's Goal

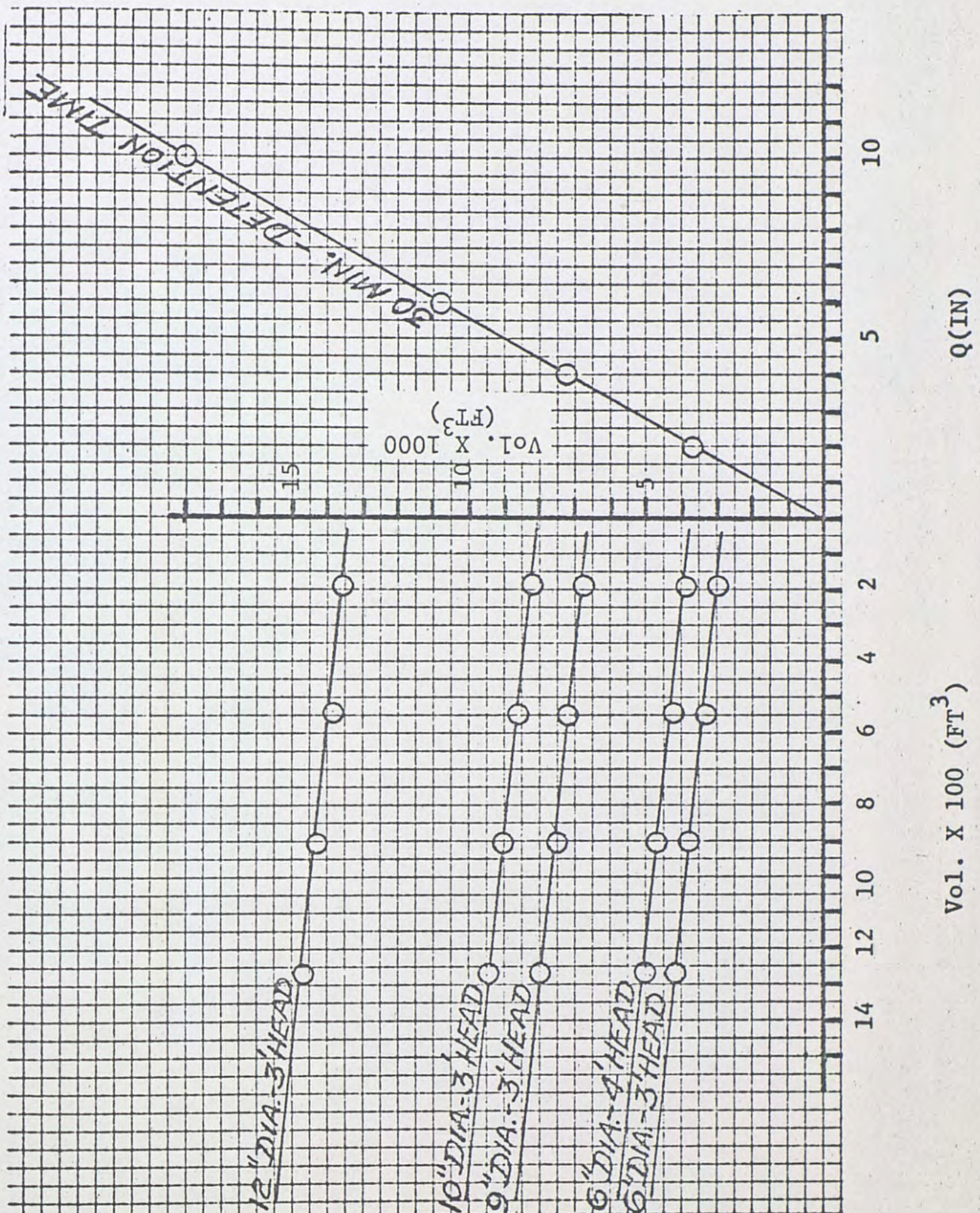
FIGURE II-4

operation of the overflow control device is performing its function of preventing the higher BOD level of stormwater from entering the lake. This is based on the fact that during higher intensity storms greater quantities of stormwater are flowing with a higher velocity, therefore capable of transporting a greater amount of surface debris in a shorter time interval. This in effect cleans the street much faster, thus the rapid decrease of BOD at the tail of the curve.

The steady state conditions for 3 various size orifices and heads are given in Figure II-5 as an indicated aid for determining the design steady state volume for a given detention time for a specific Q(in). A family of curves can be generated in this way for various orifice diameters and operating heads, however, only a few are shown to indicate the general trend. By going to any Q(in) and detention time of thirty minutes the design volume of the collecting chamber can be determined by reading over to the applicable orifice sizes and heads. Graphs similar to this for various detention times could be generated using the same approach as used to generate Figure II-5. If the volume was found to be large, a further evaluation of the area as to increasing the number of storm overflow control devices, should be performed to obtain reasonable volumes.

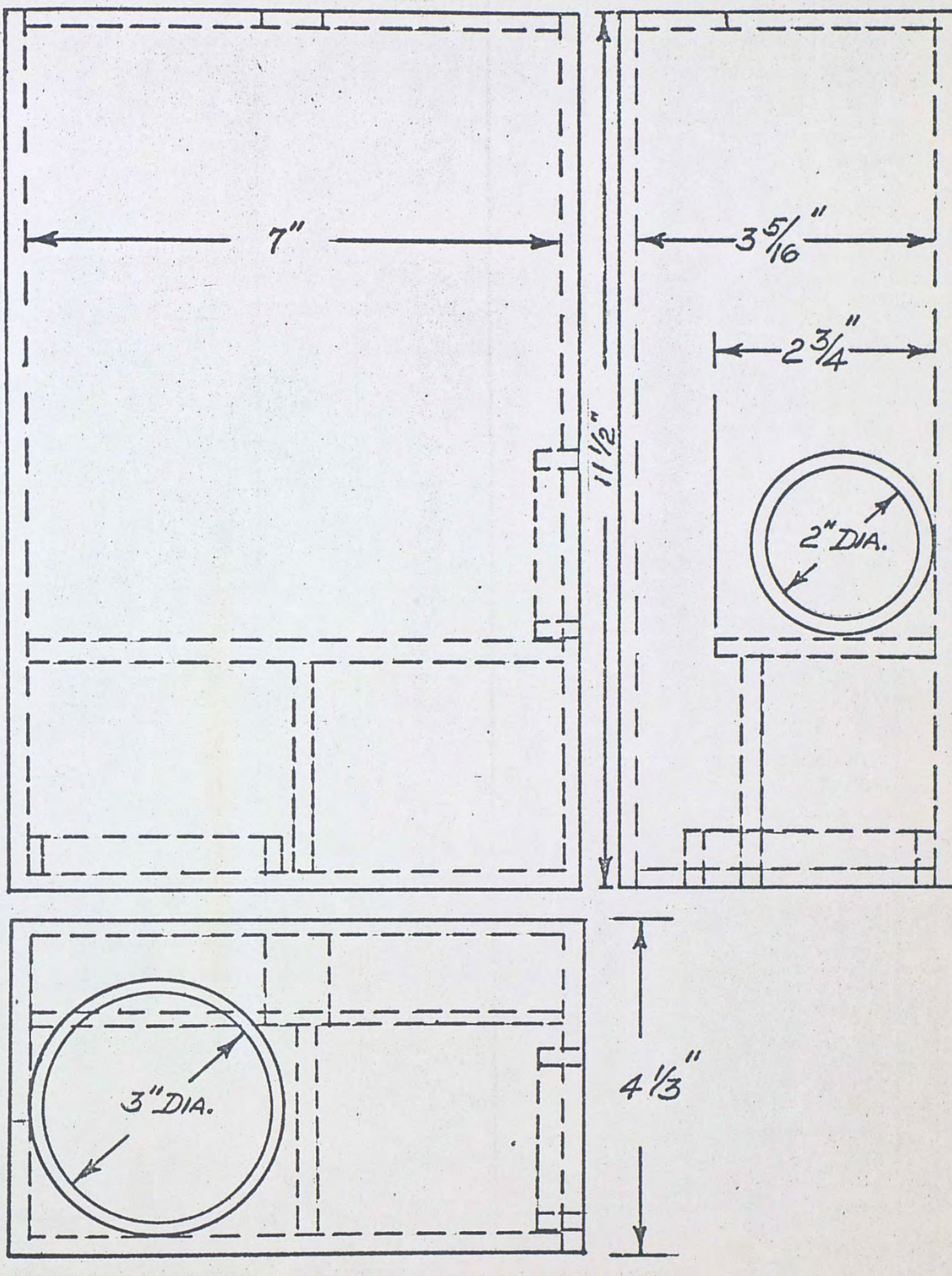
Description of Model, Construction and Results

A model was built in order to demonstrate the orifice weir combination. It was modeled after an existing stormwater collecting basin in use on Lake Eola (see Fig. II-2) and modified according to the design two-dimensional drawing in Figure II-6. The model was made on the scale of 1:12 or one inch equals a foot. The actual



Q(IN) Versus Storage Vol. Versus Required Steady State Vol. Using a Specific Orifice and Head.

FIGURE II-5

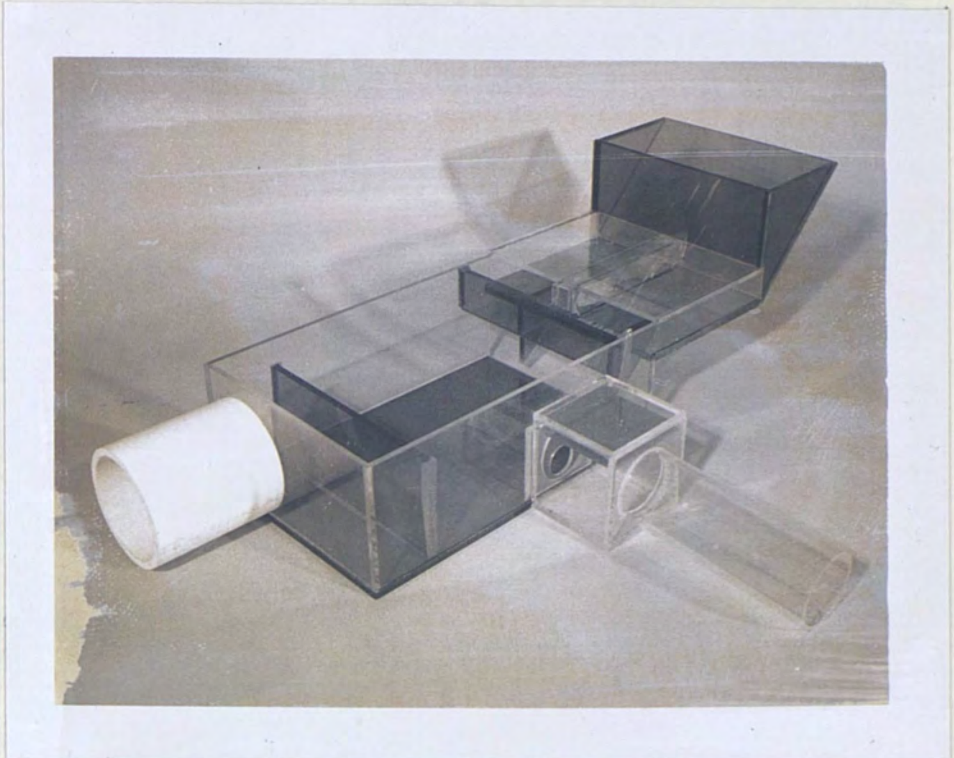


Two Dimensional View of Stormwater Overflow Control Device

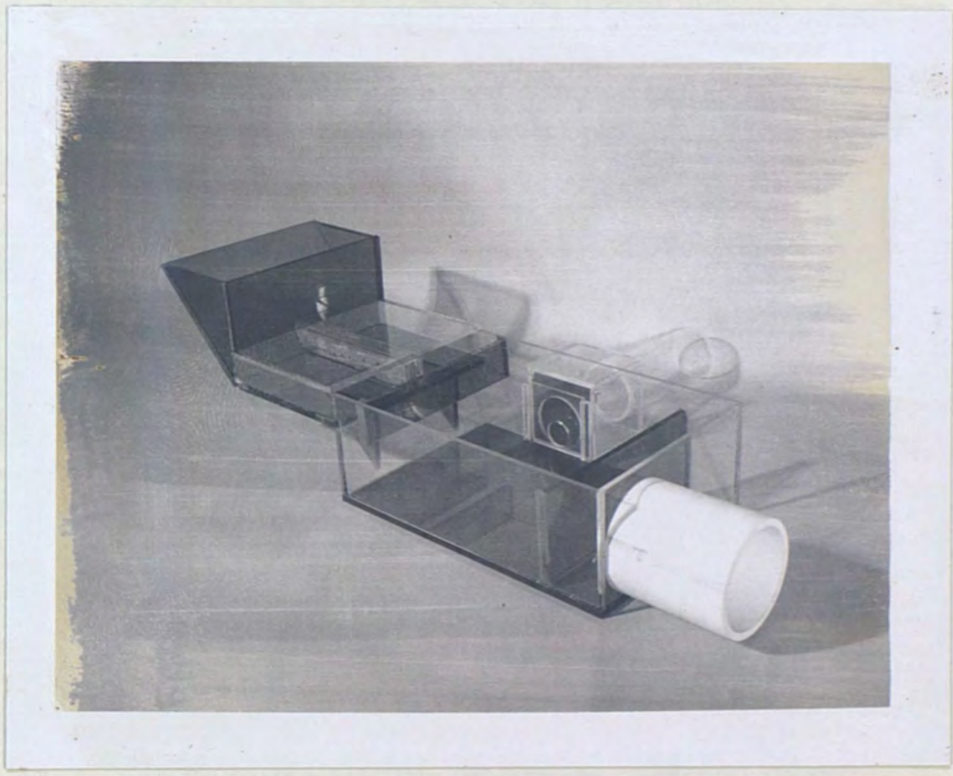
FIGURE II-6

model is pictured in Figure II-7 showing it in two views. View A brings out the side discharge orifice located as near to the bottom as possible and the flapper valve (shown open) to prevent reverse flow when the treatment pipe is full. The white pipe is the discharge pipe into the lake. The weir or baffle plate is adjustable and located just behind the white lake outlet pipe. Also, shown just forward of the inlet channels, forward of the hopper, is a deflecting baffle plate which was put in to lessen the turbulence in the orifice chamber, especially during high flow input tests. The second view shows the white lake discharge pipe clearly and also the method used for changing sizes of orifices. In this view, the flapper valve is closed which would be the position if no flow was passing through the orifice or when the treatment pipe is full.

A test was run on the model to determine the steady state flow conditions for various input flows (Q(in)) versus time. The results are shown in Figure II-8. The results were highly varied which indicates that the possibility of measurement errors were present. This is definitely a possibility since a stopwatch and an 1800 ml beaker were used as the flow measurement devices. It is noted that the .75 inch diameter exhibited the most regular data points whereas the larger orifice sizes seemed to have almost a vertical increase in flow versus time until they approached the specified 3.0 inch head height at which time they tended to flatten out. This phenomenon could be partially explained by the transient effect as the water raised within the surface area of the orifice. It would also be more distinct as the orifice diameter increased which was found to be true in the data collected. These transient



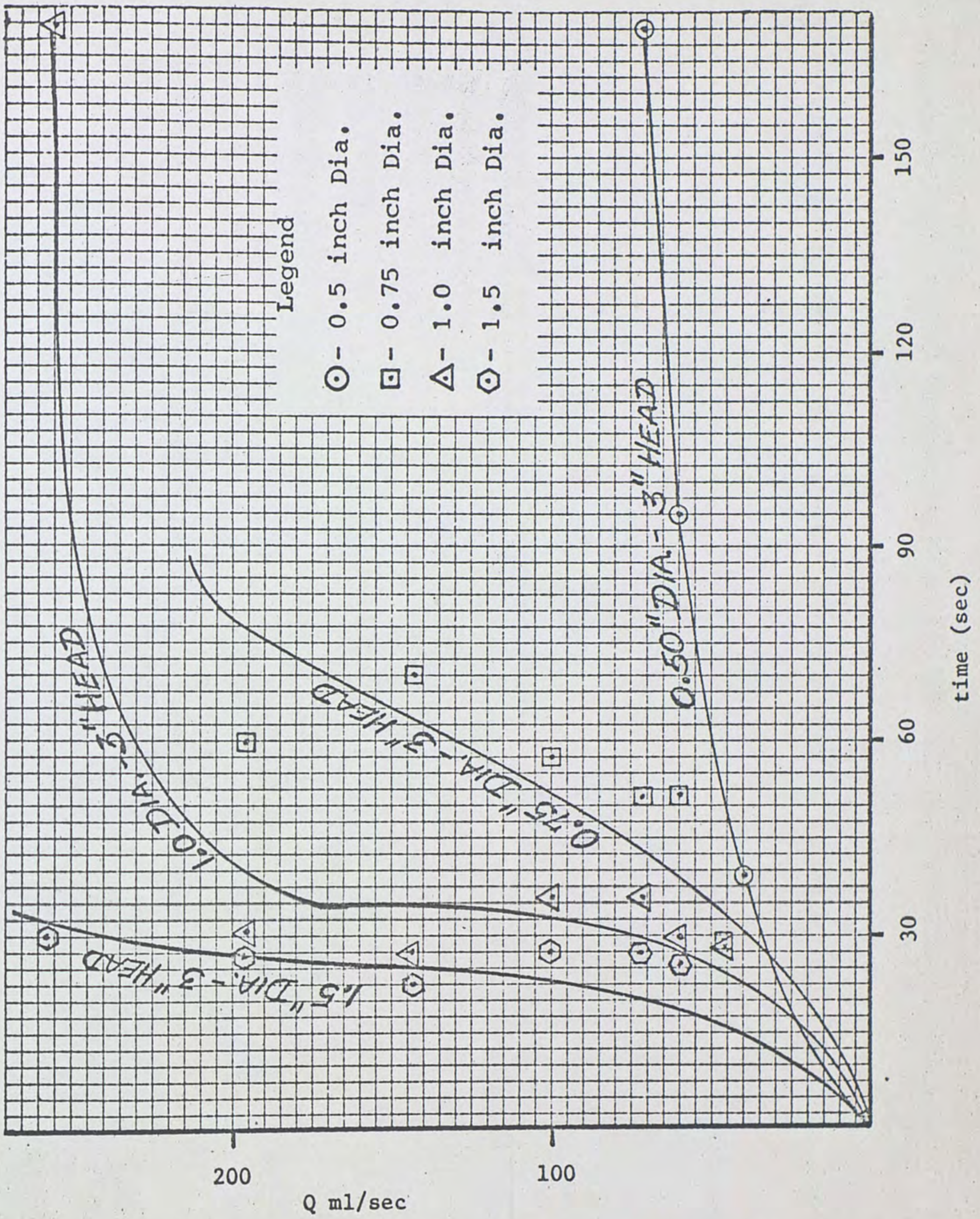
A. Orifice Side View With Flapper Valve Open



B. Lake Input Side View Showing Flapper Valve Closed

Stormwater Overflow Control Device

FIGURE II-7

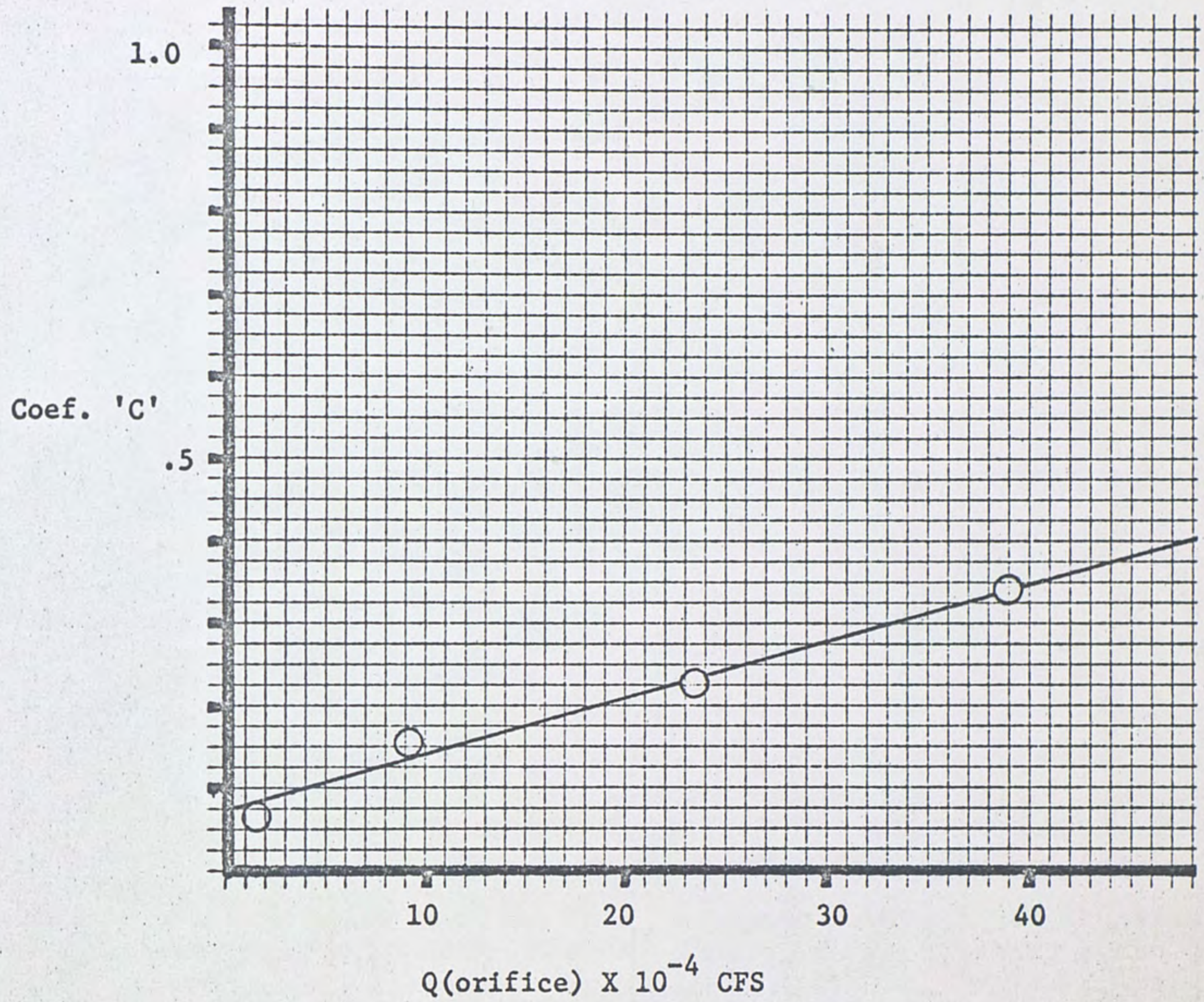


Steady State Flow Versus Time For Different Orifice Diameters With Fixed Volume Maintained

FIGURE II-8

flows could be very significant since they actually represent discharge through an orifice under low heads. (12) The formula Q = Ca 2gh is generally employed for all orifices. For those discharging under low heads, deviations from the formula is corrected for in the coefficient. This led to testing one of the orifices to determine the relationship of C in the general orifice equation for water at different heights in the orifice. This test indicated a relatively linear relationship, as shown in Figure II-9, considering that the precise measurement of flows was difficult. This verifies that there is a relationship between Q and C which could be accounted for in the selection of the discharge coefficient C.

In considering the model in comparison with the prototype, there exists a functional relationship among the several variables, such as, geometrical boundary dimensions, the characteristics of flow, and the fluid properties. All of which can be grouped together in a number of dimensionless parameters. If true similarity is to exist between flow in the prototype and flow in the model, all the dimensionless parameters referring to conditions in the model must have the same numerical magnitudes as the corresponding parameter referring to the prototype. This being true, then the model and the prototype are similar geometrically; the flow characteristics will be similar such that the Reynolds or Froude number are equal in both cases. With this condition existing, the flows can be varied in the model without destroying the similarity between prototype and model. It is a common error for one to think of the model scale simply as the ratio of corresponding geometrical dimensions between model and prototype. As explained by Rouse (15), one may think of the foot or



Discharge Coefficient Versus Flow Through 0.75 Inch Orifice

FIGURE II-9

the second in the model world as some fraction or multiple of that in our own world. In other words, one may treat all model dimensions as some fraction of those of the prototype, each measured according to the same standard dimensional units. It was also brought out by Rouse that true similarity is generally impossible when one and the same fluid is used, for it seldom happens that only one force property exists by itself. Even if the geometrical characteristics could be duplicated to the model scale, such as surface roughness, too great a reduction in scale would make viscous and capillary action of appreciable importance.

Since the same fluid (water) was used in the model and recognizing that true similarity did not exist between prototype and model, the aim was at a qualitative indication of prototype performance. Thus, if flow in the prototype is turbulent, that in the model must also be turbulent. In this way the model was used to successfully demonstrate the principle of operation.

In the actual performance of tests, it was observed that at high flows the water in the orifice chamber was very turbulent. This effect causes a mixing action in the chamber and possibly could keep heavier particles suspended in the chamber. By placing a baffle plate in front of the inlet channels it was observed that a drastic reduction in turbulence occurred and the water rose in the chamber in a much more even profile.

As to the capacity of the forward channel which would be filled to lake height when overflow occurred, would be dependent on the discharge pipe diameter into the lake, the head developed and the level of the lake at that time. However, it should be noted

that the water discharges into the lake by means of differences of head. Therefore, to reduce head losses by providing a large enough discharge pipe or even multiple discharge pipes would allow the flow to easier entry into the lake. Even at flows greater than 857.1 ml/sec in the model which is equivalent to flows of 52.3 CFS the weir was operating and the flow was discharging without accumulating in the forward chamber. However, the pipe diameter was three inches and was allowed to discharge freely. The discharge over a weir has not been defined exactly since not only does the flow pattern of one weir differ from another, but the flow pattern for a given weir varies with discharge. Also, the number of variables involved is so great that it makes a rigorous analytical approach extremely difficult. There have been many approximations (12) but they do not include the effects of viscosity, surface tension, the ratios of the dimensions of the weir to the dimensions of the approach channel, the weir crest, and the roughness of the side surface of the weir and approach channel. Since the stormwater overflow control device is designed for the most frequently occurring storm in the area the operation of the weir should be within its limitations most of the time.

III. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

Summary and Conclusions

Bodies of water enter a eutrophic state when the pollution applied to them becomes greater than their natural capacity for self-purification. One of the most obvious sources of lake eutrophication is the stormwater runoff. It has been previously concluded that the pollutional load of the initial flush of stormwater was much stronger than normal sewage. This pollution must be controlled and the stormwater overflow device presented is an attempt to control this situation.

This control device was designed as a modification to the existing stormwater collecting system. However, its application can be extended to any areas where stormwater runoff enters the lake directly. It is by no means the solution to the stormwater problem but for the situation applied it is a reasonably economical, easily maintained and effective means of controlling a certain range of rainfall intensities.

This system has the advantage over the existing system in that water doesn't stagnate in the system after a storm but is diverted to treatment facilities. The chamber is then dry making it easy to clean. Screening devices are still required to collect the larger type debris, such as cans, bottles, leaves, glass and paper. An anaerobic condition will not exist because the water will not remain in the overflow device for any great periods of time. A less desirable feature is also present in this device. There is no way to prevent surface slime and light particles from entering the lake. However, it is not the intent of the

system to remove all pollutants which are possibly present but to eliminate enough of them so that the natural biological balance of the lake can be maintained.

A graphical aid was developed to select the proper volume required for the stormwater overflow device by knowing only the designed input flow. Not only was the volume determined by the appropriate orifice size, but also the associated heads available to satisfy that volume requirement were determined.

A 1:12 scale model was constructed and tested to demonstrate the controllable variables associated with the control device. The variables were orifice diameter, head height, surface area and the input flow. It was demonstrated that the output flow from the orifice was head dependent which meant that as long as $Q_{(in)}$ minus $Q_{(out)}$ was equal (steady state condition) the surface area had no effect except on the time to reach steady state condition. From this observation it then became clear that to determine detention time required, an overflow condition had to exist ($Q_{(in)} > Q_{(out)}$). Then knowing the detention time the surface area required can be determined given the inlet flow ($Q_{(in)}$). Also, the model demonstrated the effect of the flapper valve. When flow in the orifice output drain was stopped the flapper valve, as expected, gradually closed as the pipe filled. The model was tested under the assumed conditions that the orifice output drain was not overloaded and the output to the lake was free discharge.

It is the characteristic of stormwater to gradually build up to a peak and then level off and finally taper off. This is an important feature to consider when determining detention times. In the above analysis, the flow rate was considered instantaneously at its peak flow.

Therefore, it can be easily seen that under normal operating conditions this time lag would definitely increase the expected detention time. This becomes important when $Q_{(in)}$ exceeds $Q_{(out)}$ by a significant amount.

This stormwater overflow device as described here is not the total solution but is a simple and effective system in comparison to the other elaborate and mechanical devices in use presently. The only requirement by the system is that a treatment facility be available.

Recommendations

As a result of this project research areas were determined that either require further study to determine effectiveness or would improve the overall operation of the stormwater overflow device.

From the test performed on the model, indications were shown that transient effects occurred as the fluid level rose in the orifice chamber which could affect the time it takes the water to obtain steady state. It is recommended that further study be performed to determine the transient effects of water rising within the diameter of a circular orifice at low heads. Knowing the effect of this phenomenon may allow for an accurate design analysis for determination of the proper volume of chamber versus detention time desired.

It was noted that the flapper valve provided a means of preventing reverse flow into the system. It could also be the control means of regulating the output flow. Since the proposed system is a static system, the orifice will always be open. This may not be necessary, since in the case of multiple storms in a day the runoff that occurred during the first storm may have been adequate to cleanse the area thus making diversion of further runoff unnecessary. Also, if the storm was

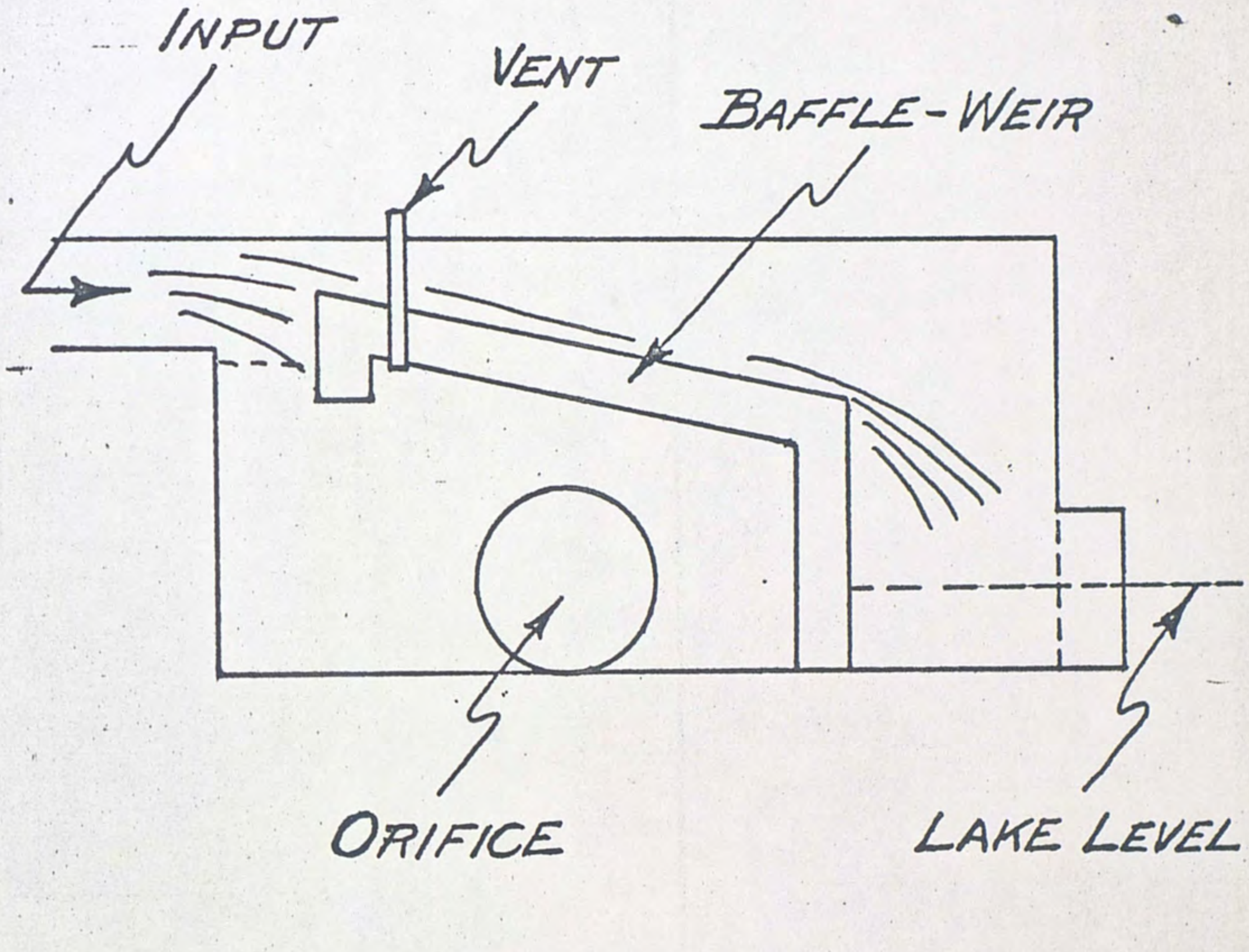
of an intensity and duration such that it removed the major initial pollutants early in the storm, then further diversion would be unnecessary and the additional water used to recharge the lake.

During periods of high input flow the orifice chamber was observed to be very turbulent with a resulting affect of possibly good mixing of the suspended solids intended to be removed which would discharge into the lake when the flow was great enough to overflow the weir. By properly locating a baffle in front of the input channels such that during high flows the water would be deflected off the baffle and downward, would lessen the turbulence considerably.

As an extension to the previous recommendation, a baffle weir effect could be provided. This would be constructed as shown in Figure III-1 and would operate in the same manner as before except when the flow filled the orifice chamber, the input flow would be diverted directly into the lake chamber elminating the mixing characteristics that could exist in the orifice chamber, due to turbulent conditions.

In the front or lake chamber it was noted that water remained in the chamber equivalent to the level of the lake. This water could become stale and stagnant during dry weather or droughts. This condition could be minimized by having only the area equivalent to the width of the output pipe allowed to have water. The remaining area should be raised to the average height of the lake and tapered for drainage for periods when the lake level raises.

Fluidic devices exist that are analogous to most all electronic devices. It is recommended that a thorough examination of these devices be performed to determine if an application can be found or designed to use the incoming water or a portion of it to control its own course,



Baffle-Weir Modification

FIGURE III-1

thus, eliminating the use of mechanical devices for the same function.

The operation of the weir is simple but is complicated by the number of variables involved. An equation for discharge over a weir has not been derived exactly since the flow pattern of one weir is different from the flow patter of another, along with the fact that the flow pattern for a given weir varies with the discharge. This effect was not examined to any great detail but should be examined to provide design criteria for sizing the forward overflow chamber.

For this proposed stormwater overflow control device a treatment facility is required of one kind or another. It is recommended that a small on-site treatment facility be given serious consideration. This treatment facility could be used not only to treat the storm runoff but in periods when it's not in use could be treating the lake water itself.

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APPENDIX A

CALCULATIONS FOR DETERMINING Q(IN) VERSUS STORAGE VOLUME VERSUS REQUIRED STEADY STATE VOLUME USING A SPECIFIC ORIFICE AND HEAD

A. Q(IN) VERSUS STORAGE VOLUME

Q(IN) (CFS)	Detention Time (Sec)	Steady State Vol. (Ft^3)
8	1800	14400
4	1800	7200
2	1800	3600
0	1800	0

B. VOLUME VERSUS ORIFICE AND HEAD

Orifice Dia (IN)	Head (Ft)	C (Dimensionless)	a (Ft^2)	$\sqrt{2gh}$ (Ft/Sec)	Q(orifice) (CFS)
6	4	.597	.196	16.05	1.88
6	3	.598	.196	13.90	1.63
6	2	.597	.196	11.35	1.33
9	4	.596	.442	16.05	4.23
9	3	.597	.442	13.90	3.67
9	2	.596	.442	11.35	2.99
10	3	.598	.545	13.90	4.52
12	4	.596	.785	16.05	7.51
12	3	.597	.785	13.90	6.51
12	2	.595	.785	11.35	5.30

C. DETERMINING VOLUMES REQUIRED GIVEN ORIFICE AND HEAD:

$$V = Qt$$

10 Inch Diameter, 3 Ft. Head

Q(IN) (CFS)	Q(orifice) (CFS)	Q(rem) (CFS)	D.t. (sec)	Vol (Ft^3)
4.6	4.5	.1	1800	180
4.8	4.5	.3	1800	540
5.0	4.5	.5	1800	900
5.2	4.5	.7	1800	1260
4.55	4.5	.05	1800	90

9 Inch Diameter, 3 Ft. Head

3.8	3.7	.1	1800	180
4.0	3.7	.3	1800	540
4.2	3.7	.5	1800	900
4.4	3.7	.7	1800	1260
3.75	3.7	.05	1800	90

12 Inch Diameter, 3 Ft. Head

7.6	7.50	.1	1800	180
7.8	7.50	.3	1800	540
8.0	7.50	.5	1800	900
8.2	7.50	.7	1800	1260

6 Inch Diameter, 3 Ft. Head

1.7	1.6	.1	1800	180
1.9	1.6	.3	1800	540
2.1	1.6	.5	1800	910
2.3	1.6	.7	1800	1260

6 Inch Diameter, 4 Ft. Head

Q(IN) (CFS)	Q(orifice) (CFS)	Q(rem) (CFS)	D.t. (sec)	Vol (Ft^3)
2.0	1.9	.1	1800	180
2.2	1.9	.3	1800	540
2.4	1.9	.5	1800	900
2.6	1.9	.7	1800	1260

APPENDIX B

TEST RESULTS FROM MODEL TESTS

Steady State Vol (Ft^3)	Orifice Dia (in)	Q (ml/sec)	t (sec)	Q (ml/sec)	Q (CFS)
9" X 7 3/16" X 3"	.5	1800/45.2	49.0	39.8	.0074
	.5	1800/39.0	33.0	46.2	.0016
	.75	1800/39.0	29.0	46.2	.0016
	1.00	1800/39.0	27.0	46.2	.0016
	1.5	1800/39.0	29.3	46.2	.0016
	.50	1800/30	100	60	.0021
	.75	1800/30	44	60	.0021
	1.00	1800/30	29	60	.0021
	1.50	1800/30	22	60	.0021
	2.00	1800/30	21	60	.0021
	.50	1800/25	227	72	.00254
	.75	1800/25	43	72	.00254
	1.00	1800/25	43	72	.00254
	1.50	1800/25	27	72	.00254
	2.00	1800/25	26	72	.00254
	.50	1800/18	overflow	100	.0035
	.75	1800/18	55	100	.0035
	1.00	1800/18	42	100	.0035
	1.50	1800/18	23	100	.0035
	2.00	1800/18	21	100	.0035
	.75	1800/12.6	81	143	.00504
	1.00	1800/12.6	24	143	.00504
	1.50	1800/12.6	14.5	143	.00504
	2.00	1800/12.6	16	143	.00504
	.75	1800/9.2	60	195.6	.0069
	1.00	1800/9.2	36	195.6	.0069
	1.50	1800/9.2	23	195.6	.0069
	2.00	1800/9.2	25	195.6	.0069
	.75	1800/7	overflow	257.1	.0091
	1.00	1800/7	184	257.1	.0091
	1.50	1800/7	29	257.1	.0091
	2.00	1800/7	16	257.1	.0091
	1.0	1800/3	overflow	600	.021
	1.5	1800/3	55	600	.021
	2.0	1800/3	14	600	.021

APPENDIX C

DETERMINE RELATIONSHIP OF DISCHARGE COEFFICIENT VERSUS FLOW THROUGH 0.75 INCH ORIFICE

From equation: $Q = Ca\sqrt{2gh}$

Diameter = 0.75 inch
Head = 3 inches
Area = $\pi Q^2/4 = \pi(.75)^2/4 = 0.44$ square inches

(A) Area Determination: From the New American Machinists Handbook (pp. 44) area of segments are given in percentage of area of a circle, according to percentage of h to D.

h(in)	D(in)	%	h/D	A(in^2)
.25	.75	.1875	.25	.086
.50	.75	.3750	.5	.220
.75	.75	.5625	.75	.354
1.00	.75		1.00	.440

(B) Flow (Q) Determination:

A	Q(ml/sec)	Q(CFS) X 10^{-4}
.086	4.7	1.66
.220	25.97	9.17
.354	66.67	23.54
.440	111.11	39.23

(C) Discharge Coefficient Determination:

$$C_1 = Q/a\ 2gh = \frac{4.7 \text{ ml/sec X } 144 \text{ in}^2/\text{ft}^2}{28.32 \text{ L/CFS X } 10^3 \text{ ml/L X } .086 \text{ X } 4.0135} = .0692$$

$$C_2 = \quad = \frac{25.97 \text{ X } 144 \text{ in}^2/\text{ft}^2}{28.32 \text{ X } 10^3 \text{ X } .222 \text{ X } (4.013)} = .148$$

$$C_3 = \qquad = \frac{66.67 \times 144}{28.32 \times 10^3 \times .354 \times 4.013} = .239$$

$$C_4 = \qquad = \frac{111.1 \times 144}{28.32 \times 10^3 \times .440 \times 4.013} = .320$$