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Swirl and Helical Bend Pollution Control Devices

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DESIGN MANUAL

SWIRL AND HELICAL BEND POLLUTION CONTROL DEVICES

by

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FOREWORD

The U.S. Environmental Protection Agency was created because of increasing public and governmental concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

This design manual consolidates and updates the work of many researchers over the past ten years. As the family of pollution control devices has evolved from the laboratory to prototype demonstration units, the value and need for refinements and modifications to design procedures became evident. Now that many of the units, particularly the combined sewer overflow regulators and swirl degritter, are ready for general use, this design manual is particularly appropriate. These secondary flow motion pollution control devices should play a key role in the nation's efforts to correct pollution problems from combined sewer overflows and stormwater discharges.

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ABSTRACT

Hydraulic and mathematical modeling have been used to develop several pollution control devices for specific applications, particularly for controlling and treating combined sewer overflows and stormwater discharges. Prototype testing of each unit has been accomplished by various researchers in the United States and other countries. This design manual brings together pertinent information concerning the design and operation of the units and thus, consolidates information from many reports. Inasmuch as the design has been evolutionary in nature, the design procedures contained in this manual replace that which has previously been published.

Two types of combined sewer overflow regulators are described: the swirl and the helical bend regulator/separator. Both units are static, that is, operate without moving parts and require no outside source of power. Both can remove up to 50 percent of the suspended solids. Both are also effective for treating separate stormwater discharges. Both serve a dual function - treatment and regulation of the flow.

The units treat waste flows by concentrating the solids in a small fraction of the total flow. This reduced volume becomes economical, or in effect, possible to treat in conventional wastewater treatment facilities.

The degritter unit is for use in removing from the underflow to treatment the solids concentrated by the combined sewer overflow regulators, or for use in conventional treatment facilities.

A primary treatment device and a sediment load polishing unit are also described. Both have special applications. In addition, several devices and applications which have been developed by others as a result of the basic information on the flow field characteristics and capabilities are described.

The design manual contains thorough descriptions of the design procedures, operating experience to date, and results obtained.

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SECTION I

INTRODUCTION

OBJECTIVE

From 1972 through 1979, the American Public Works Association in conjunction with the U.S. Environmental Protection Agency (USEPA) has prepared and published a series of research reports on the development and demonstration of secondary-flow-motion wastewater control/treatment devices. These reports have explained the hydraulic and mathematical testing for development and/or prototype evaluation of the following devices: combined sewer overflow regulators (1, 2, 3, 3a), a degritting device (4,5, 5a), primary treatment facility (5a, 6), a soil erosion product removal unit (7), and an analysis of the solids which can be removed from such flow fields.(8) These reports plus related articles and publications (8a, 8b, 8c) were evolutionary; that is, with additional testing and operating experience, designs have been modified to improve efficiency and apply to different waste streams.

This Design Manual has been developed to provide information for each type of device, provide design examples, estimate costs (January 1980), and give the results of prototype tests. Thus, this manual supercedes the design and cost information of the earlier reports. Nevertheless, these original reports remain valuable to designers as an indication of the development of flow principles and various "logical" modifications which were investigated and found unworkable.

From the experience gained to date, the following caution is offered: follow the design procedure given, understand the possible impact of design modification upon the flow field, and make certain that whoever performs the actual design comprehends the importance of working with a device that depends entirely on the hydraulic conditions developed.

THE COMBINED SEWER OVERFLOW AND STORMWATER PROBLEM

The initial secondary-flow-motion, wastewater control/treatment devices were developed in England on combined sewer overflows and reported by the Institution of Civil Engineers. (9, 10, 11) The work in the United States was initially directed at modifying the English designs to fit combined sewer overflow conditions in North America.

Overflow points are the built-in inefficiencies of combined sewers.

Untreated overflows from combined sewers constitute a serious and substantial water pollution source during both wet and dry periods. Nationwide, there are about 15,000 to 18,000 combined sewer overflow points. The 1977 Clean Water Act (PL95-217) requires development of plans and construction of facilities to control combined sewer overflow pollution. In October 1978, the USEPA reported to Congress that it would require an average of 8.3 to 14.3 years, assuming an annual appropriation of $5 billion, for the abatement of nationwide combined sewer overflow pollution. This indicates that Congress recognizes and is willing to support combined sewer overflow pollution control.

The North American practice of designing regulators exclusively for flowrate control or diversion of combined wastewaters to the treatment plant and overflow to receiving waters is being reconsidered. Sewer-system management that emphasizes the dual function of combined sewer overflow regulators for improving overflow quality by concentrating wastewater solids to the sanitary interceptor and diverting excess storm flow to the outfall has been recognized as a practical, efficient method to reduce receiving-water pollution.

It has been demonstrated (20, 21) that physical treatment systems are capable of handling high and variable influent concentrations and flowrates of combined sewer overflows while operating independently of other treatment facilities, except for treatment and disposal of sludge/solids residuals. Secondary-flow units emphasized in this manual are physical systems and have demonstrated good potential for treating the highly variable combined sewer overflow pollution loads.

Recently, considerable attention also has been given to the potential pollution from stormwater discharges. (12) Treatability studies indicate that stormwater may be even more amenable to treatment with secondary-flow devices than combined sewer overflows. Thus, the devices described in this manual should be considered for treating both combined sewer overflows and stormwater discharges.

HISTORICAL DEVELOPMENT OF SECONDARY FLOW MOTION POLLUTION CONTROL DEVICES

The concept of skimming the clearer top waters from the lower depth wastes flowing in combined sewers has long been used. A key principle is that wastewater treatment plants have been designed to treat only peak sanitary flows. Thus, when storm runoff occurs, much of the excess must be bypassed to receiving waters. For relief, skim-off points were inserted wherever natural waterways were convenient. Devices such as sidewall weirs, and weirs with orifices were commonly used to provide hydraulic control. During the 1950s, a new technique called a side outlet flow diverter was studied and used in Portland, Oregon. It utilized a free-surface vortex flow through a horizontal orifice plate to skim excess water from the combined storm and sanitary flows.

In 1970, an EPA state-of-the-art report on regulators (13) indicated that two British devices showed possibilities for application as combined sewer overflow regulators in the United States. These were the vortex-flow regulator, later called the swirl regulator/separator, and the helical bend regulator/separator. Both have been modified for adaptation to North American treatment practice.

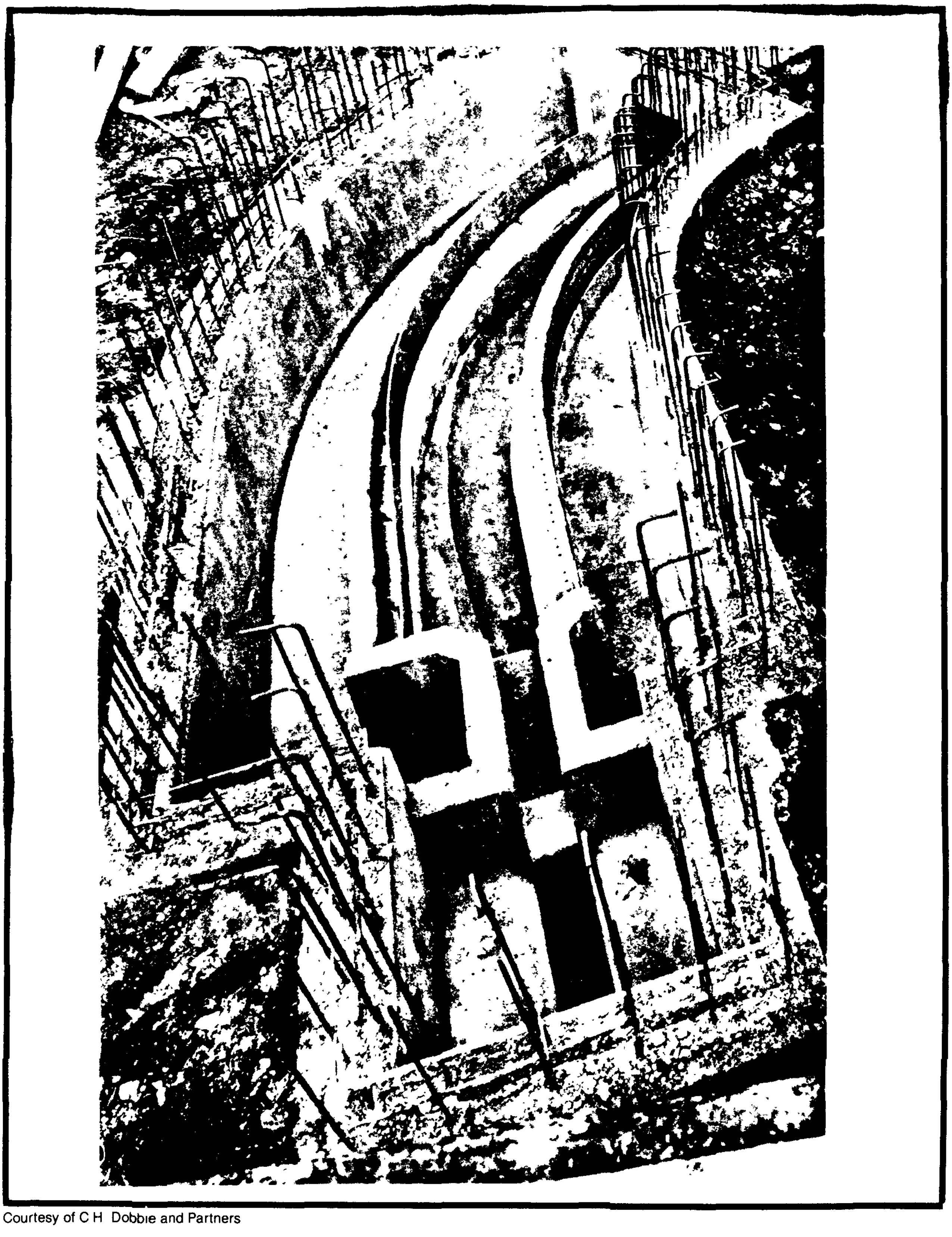
The basic hydraulic design of the swirl unit was created in a circular chamber concept by Bernard Emission for the City of Bristol, England, in order to obtain adequate weir length for overflows without the space requirement and expense of constructing a long lateral weir. As a bonus, it was found that this device could concentrate and divert as much as 70 percent of the combined wastewater settleable solids along with 30 percent of the flow volume to the treatment works.

The concept of solids removal by rotationally-induced forces causing inertial separation, rather than vertical gravity sedimentation, is behind the vortex principle utilized in this British device.

Much of the early work regarding curved channel flows was reported in the 1950s. Most of it was done in England and Russia. One of these early experimenters is T. M. Prus-Chacinski who wrote his doctoral dissertation on the subject. (14) Subsequent work led to an investigation of the use of helical motion for solids separation in combined sewers. In 1967, T. M. Prus-Chacinski reported the results of a study of a bend device which provided storm flow regulation (splitting) and solids separation. (10) In the early 1970s T. M. Prus-Chacinski and the firm with which he is associated, C. H. Dobbie and Partners, designed and constructed a helical bend combined sewer overflow regulator/separator for the City of Nantwich, England. Figure 1 shows the unit under construction.

The design criteria used for the English regulators differ from those used in North American practice, primarily in the ratio of wet-weather flow to dry-weather flow allowed to enter the interceptor for treatment. Thus, it was necessary to conduct hydraulic model studies to develop units for United States practice.

First, the swirl principle was applied for the development of dual purpose combined sewer overflow hydraulic regulators and concentrating the solids in the flow to a fraction of the total flow which could be conveyed to conventional wastewater-treatment plants. The clarified flow could be discharged with or without additional treatment. Eventually, it was applied for the development of a degritting device, as a primary separator, and for removal of soil and grit particles to reduce the effects of soil erosion at construction sites. Hydraulic models for development and testing studies took place at the LaSalle Hydraulics Laboratory near Montreal, Quebec. Mathematical modeling studies were conducted by the General Electric Company, Philadelphia, Pennsylvania. The testing program involved a series of variations of different swirl chamber elements which led to an integrated, optimized design. Of all the swirl applications tested, it was determined that the swirl combined sewer overflow regulator/solids separator is the most promising. This is based upon the unit's low construction and operation cost, ability to both hydraulically regulate and treat simultaneously, and minimum site requirements.



Courtesy of C H Dobbie and Partners

Figure 1 Helical Bend Regulator, Nantwich, England

FLOW PRINCIPLES

Swirl Combined Sewer Overflow Regulator/Separator - The fundamental cause of liquid-solid particle separation is the swirl action. Flow enters the cylindrical chamber tangentially and travels in a vortex path of decreasing radius. A foul flow outlet to the interceptor sewer removes concentrated wastes from the floor of the chamber, near the center. This is somewhat similar to the condition in a teacup in which tea leaves are present. If the cup is rotated and the tea leaves allowed to settle, they will be concentrated in the center, not along the outside edge.

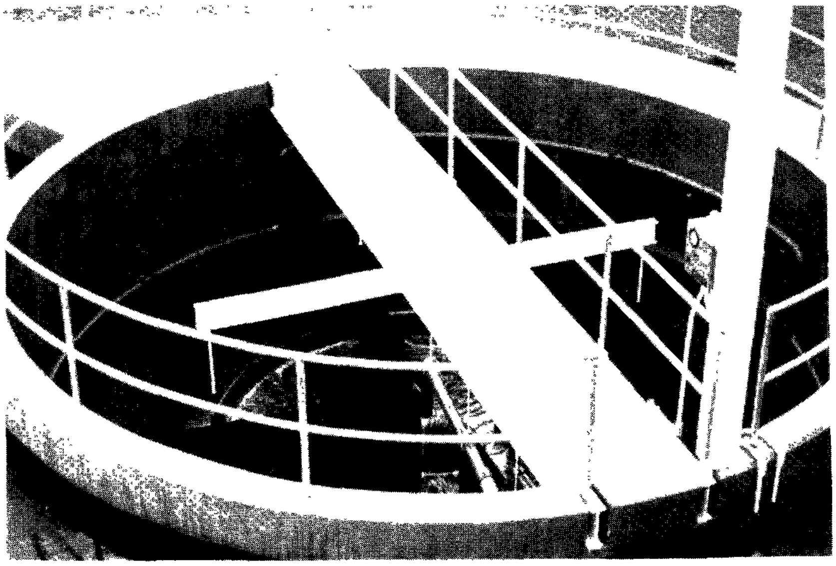
During dry weather, all flow entering the chamber is removed through the discharge pipe and transported to the treatment plant. During periods of excessive rainfall when the chamber is filled with combined wastewater, the clearer supernatant excess overflows the chamber through a central circular weir. The overflow may be conducted to storage chambers for later treatment, partially treated, and/or discharged to a stream. Floating solids are captured in the chamber and discharged at the end of the storm event to treatment. An attractive feature of the chamber is that it contains no moving parts. However, details of design must be carefully observed. Figure 2 shows photographs of some of these units and Figure 3 indicates the particle flowfield.

Helical Bend Combined Sewer Overflow Regulator/Separator - Deposits of sediment occur along the curved sides of rivers or streams, which suggested that a curved-path flow could be used to separate solid particles from the liquid.

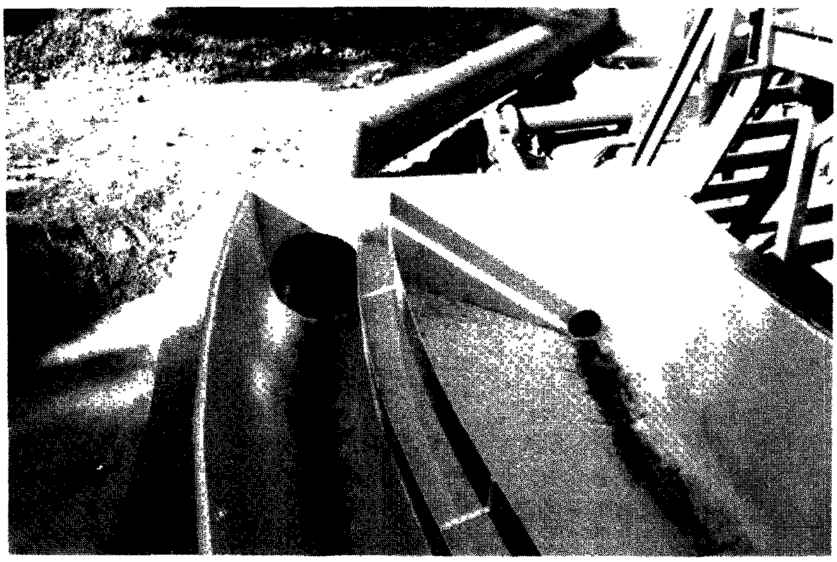
The operating principle is that flow moves into a curved path in a channel cross section with the deepest part at the inside of the curve. Solids are channeled into the trough by secondary currents and are removed into the sanitary interceptor sewer at the end of the bend through a foul-flow outlet. The clearer flow (supernatant) passes over a weir at the outside top edge of the bend. This flow may then be treated again or discharged directly to a waterway. The deepest part of the helical bend section was located at the inside of the curve rather than at the outside as it occurs in nature to develop stronger helical motion. The predominant secondary motions include the surface water moving toward the outside of the bend while at the bottom the flow along the channel slope is down the sideslope toward the deep trough. A smooth transition from the circular sewer pipe to the start of the bend is essential.

Figure 3 depicts the particle flowfield.

Other Swirl Devices - Other swirl devices such as the degritter, primary separator, and soil erosion unit use the basic swirl flow pattern with modifications to fit the application. For example, only solids and sludges are removed from the degritter and primary unit because they are, in effect, the final treatment units. For soil-erosion polishing, a rather large fraction of the total flow must be used to carry the fine particles. These requirements have led to the need for specific shapes for the bottoms



a. Swirl Regulator — overview, scum ring, weir, and floatables traps inside.



b. Helical Bend Combined Sewer Overflow Regulator/ Separator — downstream view of low-flow and overflow outlets.

Courtesy of Dr William A Pisano
Environmental Design & Planning, Inc

Figure 2 Swirl and Helical Bend Regulator/Separators

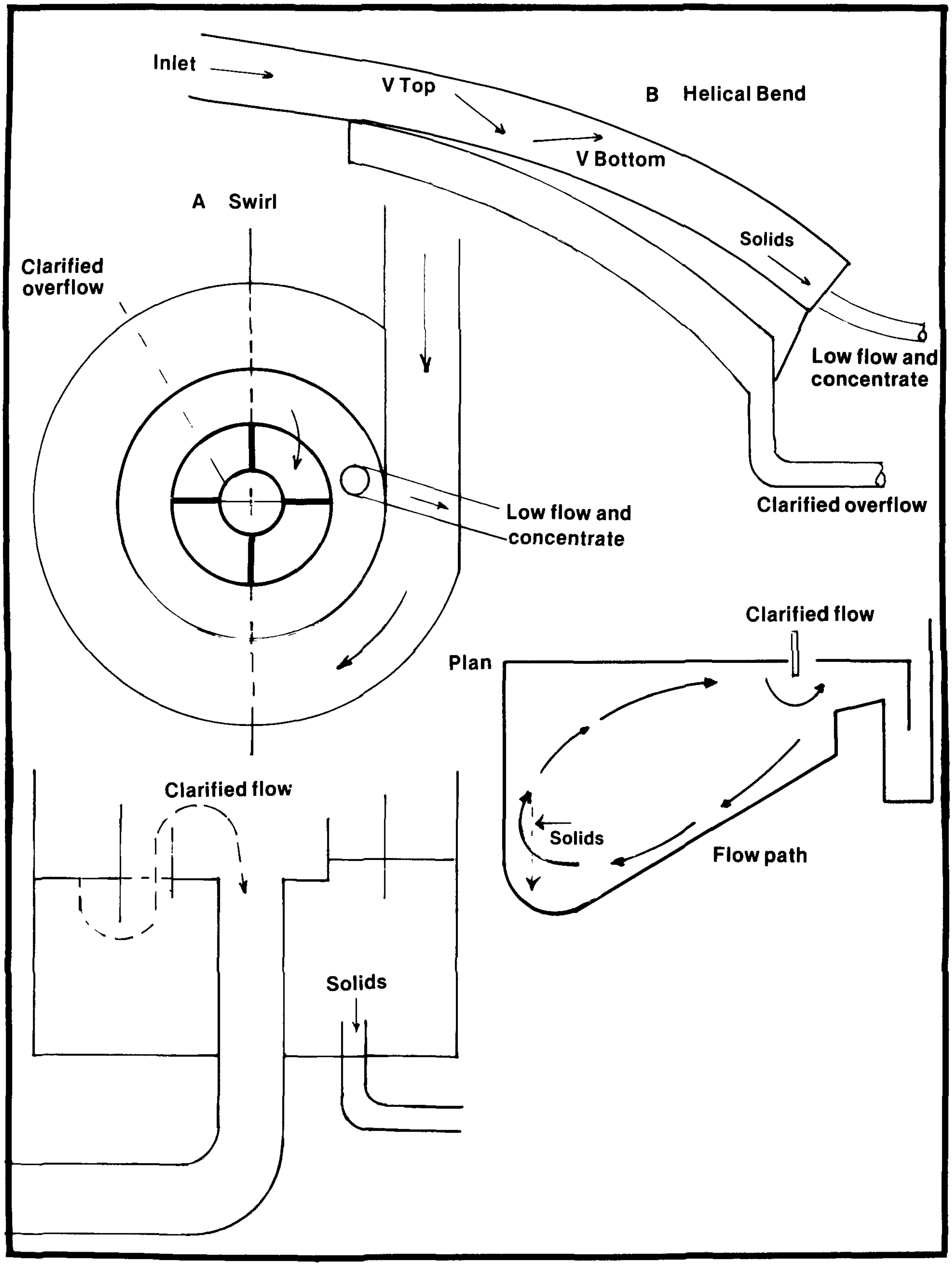


Figure 3 Flow Paths Secondary Flow Motion Control Devices

of the units for each application. Flow inlet and clarified overflow drawoff features are basically the same.

DESIGN FLOWS

A key consideration for the design of all of the devices described in this manual is the selection of the design flowrate and the amount of suspended solids to be removed. Both considerations directly influence the size of unit needed. In general, the larger the unit, the larger the flow that can be treated. Also, at less than design flow, larger units provide a higher degree of treatment.

Generally, criteria have not been firmly set regarding the allowable pollution load for combined sewer overflow. In the absence of such criteria, treatment capability will usually be set on the basis of the design flow to be treated and the settleability of the solids.

Conventional design of stormwater facilities has been determined on the basis of the frequency of occurrence of the storm event. In the design of conveyance facilities where property damage or other forms of inconvenience to the public can be expected, criteria have been based upon limiting the inconvenience to the occurrence of a storm which can be expected every so many years, often two to ten years.

With pollution control devices for such variable flows, the design task becomes one of calculating the smallest facility which will satisfy the pollution control needs of the site. The length or total volume of the storm event is not of major importance as the devices will provide treatment throughout the flow event.

From typical hydraulic analyses of storm events, the probability of storm intensities can be calculated. Stormwater monitoring or modeling allows an estimation of the solids load associated with the various intensities of flow. In general, if a pollution-control facility is designed for the maximum storm that occurs two or more times annually, the length of time that this rate will be exceeded on an annual basis is measured in minutes for the year. In addition, the concentration of solids loading during this peak period may be low. But because of the volume of flow, the total amount of solids can be significant. Thus, a careful analysis, rather than an arbitrary rule of thumb, will dictate the use of smaller, less costly units.

The full hydraulic capacity of the system must be calculated so that the total flow may be handled, either by bypassing or treating it partially. The secondary flow-motion devices have a wide range of efficiency; flows up to twice the design flow will receive a degree of treatment.

Information concerning the size and settling velocities of the solids which can be expected and which were used as a basis of determining design efficiency of solids removal appear in Sections II through VII. The levels are specific to the device and the basis is defined.

THE DEVICES AND THEIR USE

Two different types of combined sewer overflow devices were developed. Section II describes the swirl regulator/separator. Section III covers the helical bend regulator/separator. The units are compared in Section IV. In general, both can be used where a "coarse" level of treatment is needed for combined sewer overflows or stormwater discharges.

Although originally designed for treating the concentrate from the combined sewer overflow regulator, the swirl degritter, Section V, is suitable for conventional wastewater treatment plants or where grit problems occur in a collection system such as upstream of wet wells or syphons.

The swirl primary separator is described in Section VI. It was designed to give additional treatment to combined sewer overflows equivalent to full primary treatment. However, the device has limited application due to its height and low flow-handling capability. It may have application for some types of industrial wastes or where only 40 to 50 percent of the suspended solids must be removed.

The swirl concentrator for erosion runoff treatment, Section VII, is appropriate upstream of a stormwater-detention basin or on construction sites to polish stormwater runoff.

Other applications and variations of the basic devices described in this manual have been developed at universities or by the private sector. Applications for industrial waste treatment, agriculture wastes, and sewer bypass control are described in Section XI. Additional applications of the flow principles should continue to develop as researchers employ the basic principles for these devices.

It is important to note that the secondary flow motion devices treat the flow by concentrating settleable solids in a fraction of the incoming flow. This flow with the bulk of the solids must then receive additional treatment to remove the solids, except for the degritter. For combined sewer overflow devices, this can be accomplished by sending the concentrated flow to conventional wastewater treatment facilities. For the degritter, the grit is removed with a conventional grit elevator after washing. The primary separator must discharge solids to a digestor or other sludge treatment device. The erosion treatment unit requires concentrate to be directed to a solids basin where quiescent settling may complete the solids separation.

Where these devices are used, provision must be made to handle the increased solids load at the point of final removal from the system. For example, the solids from one or more combined sewer overflow treatment units may completely overtax conventional grit and solid handling facilities at a wastewater treatment plant. The total system must be considered in the design process.

Section II

SWIRL REGULATOR/SEPARATOR

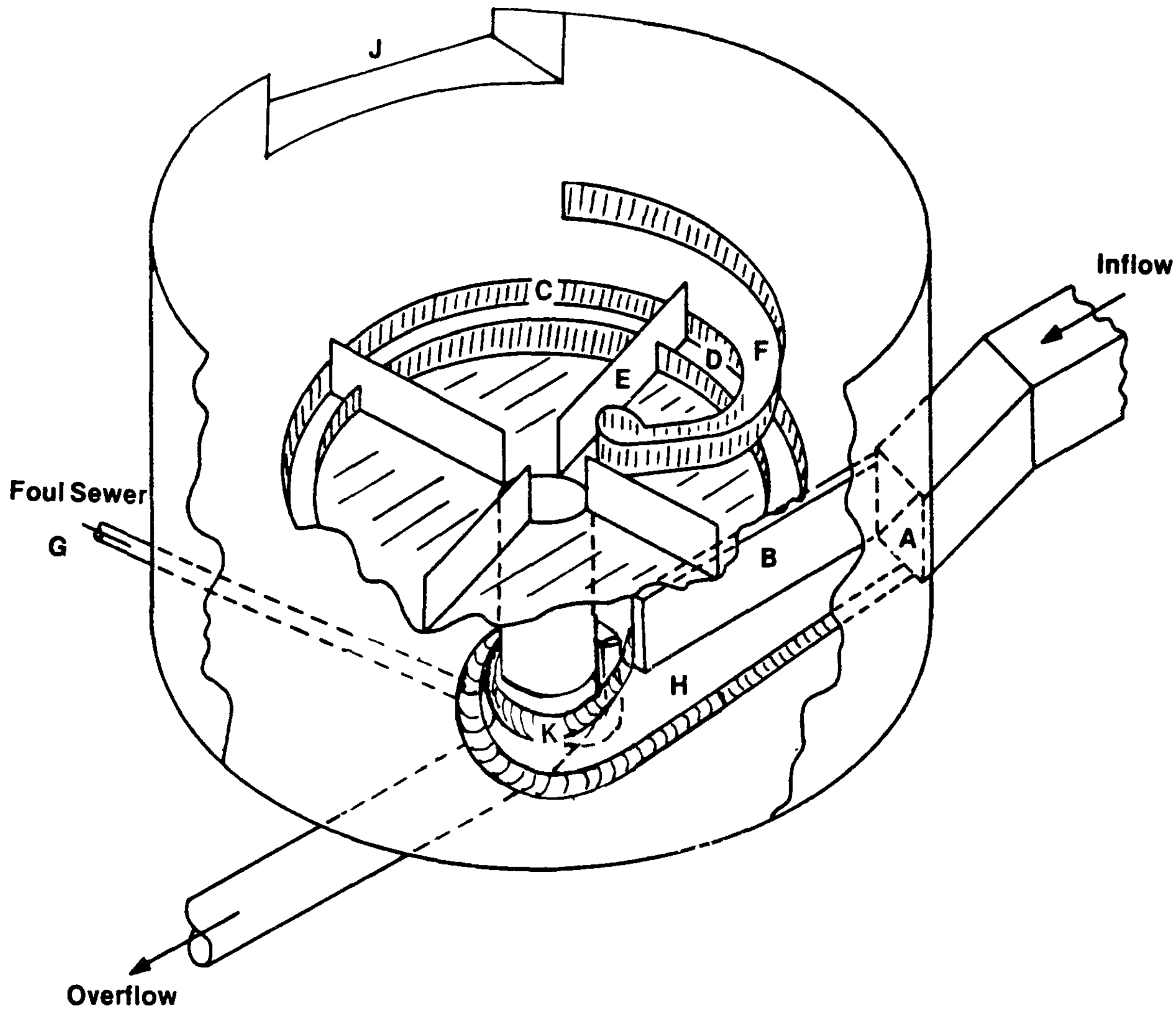
DESCRIPTION

The swirl combined sewer overflow regulator/separator is of simple annular-shaped construction and requires no moving parts to achieve a relatively high degree of separation of settleable and floatable solids from a waste stream. While accomplishing the separation of solids, it also regulates the flow to the interceptor sewer system. Wastes are concentrated into what should be a more economical to treat waste stream. Treatment of the concentrate could be at conventional wastewater treatment facilities or special combined sewer overflow treatment units.

The device consists of a circular channel in which rotary motion of the sewage is induced by the kinetic energy of the incoming sewage. Flow to the treatment plant is deflected and discharged through an orifice called the foul sewer outlet, located at the bottom and near the center of the chamber. Excess flow in storm periods discharges over a circular weir around the center of the tank and is conveyed to storage treatment devices as required or to receiving waters. The concept is that the rotary motion causes the sewage to follow a long spiral path through the circular chamber. A flow deflector prevents flow completing its first revolution in the chamber from merging with inlet flow. Some rotational movement remains, but in the form of a gentle swirl, so that water entering the chamber from the inlet pipe is slowed down and diffused with very little turbulence. The particles entering the basin spread over the full cross section of the channel and settle rapidly. Solids are entrained along the bottom, around the chamber, and are concentrated at the foul sewer outlet. Flow through the foul sewer outlet is limited to the hydraulic capacity of downstream facilities.

The device is essentially without moving parts and performs well under a variety of flow conditions. The primary features of the unit, as shown in Figure 4, include:

A. Inlet Ramp: The inlet ramp should be designed to introduce the incoming flow at the bottom of the chamber, while preventing problematical surcharges on the collector sewer immediately upstream. Introducing the inflow at the chamber floor will allow the solids to enter at a low position. It is essential that this ramp and its entry port introduce the flow tangentially so that the "long path" maximizing the solids separation in the chamber may be developed.



A Inlet ramp
B Flow deflector
C Scum ring
D Overflow weir and weir plate
E Spoilers
F Floatables trap
G Foul sewer outlet
H Floor gutters
I Downshaft
J Secondary overflow weir
K Secondary gutter

Figure 4 Isometric View of Swirl Combined Sewer Overflow Regulator/Separator

B. Flow Deflector: The flow deflector is a vertical wall which is a straight line extension of the interior wall of the entrance ramp. Its location is important because it directs flow which is completing its first revolution in the chamber to strike and be deflected inwards, forming an interior water mass which makes a second revolution in the chamber, thus creating the "long path".

C. Scum Ring: The purpose of the scum ring is to prevent floating solids from overflowing.

D. Overflow Weir and Weir Plate: The weir plate is a horizontal circular plate that connects the overflow weir to a central downshaft which carries the overflow liquid to discharge. Its underside acts as a storage cap for floating solids directed beneath the weir plate through the floatables trap. The vertical element of the weir skirt is extended below the weir plate to retain and store floatables.

E. Spoilers: Spoilers reduce rotational energy of the liquid above the weir plate and between the scum ring and weir, thus increasing the overflow capacity of the downshaft and improving the separation efficiency.

F. Floatables Trap: This trap is a surface flow deflector which extends across the outer rotating flow mass, directing floating materials into a channel crossing the weir plate to a vertical vortex cylinder located at the wall of the overflow downshaft. The floating material is then drawn down beneath the weir plate by the vortex and dispersed under the plate around the downshaft. Details are shown in Figure 5, photographs of an operating unit.

G. Foul Sewer Outlet: This outlet is the exit orifice designed to direct peak dry-weather flow and separated combined sewage solids in the form of a concentrate to the interceptor.

H. Floor Gutters: The primary floor gutter is the peak dry-weather flow channel connecting the inlet ramp to the foul sewer outlet, avoiding dry-weather solids deposition. The secondary gutter enhances capture of solids.

I. Downshaft: During higher-flow storm conditions, the low-volume concentrate is diverted to the interceptor via the foul sewer outlet, and the excess relatively clear, high-volume supernatant overflows the center circular weir into the downshaft for storage, treatment or discharge to the receiving stream.

J. Secondary Overflow Weir: Should the flow exceed approximately twice that for which the unit is designed, the excess flow is allowed to discharge over the secondary overflow weir as when such flows occur; the downshaft becomes hydraulically throttled and loses its efficiency. In addition, asymmetrical flow patterns develop reducing separation of solids from the flow.

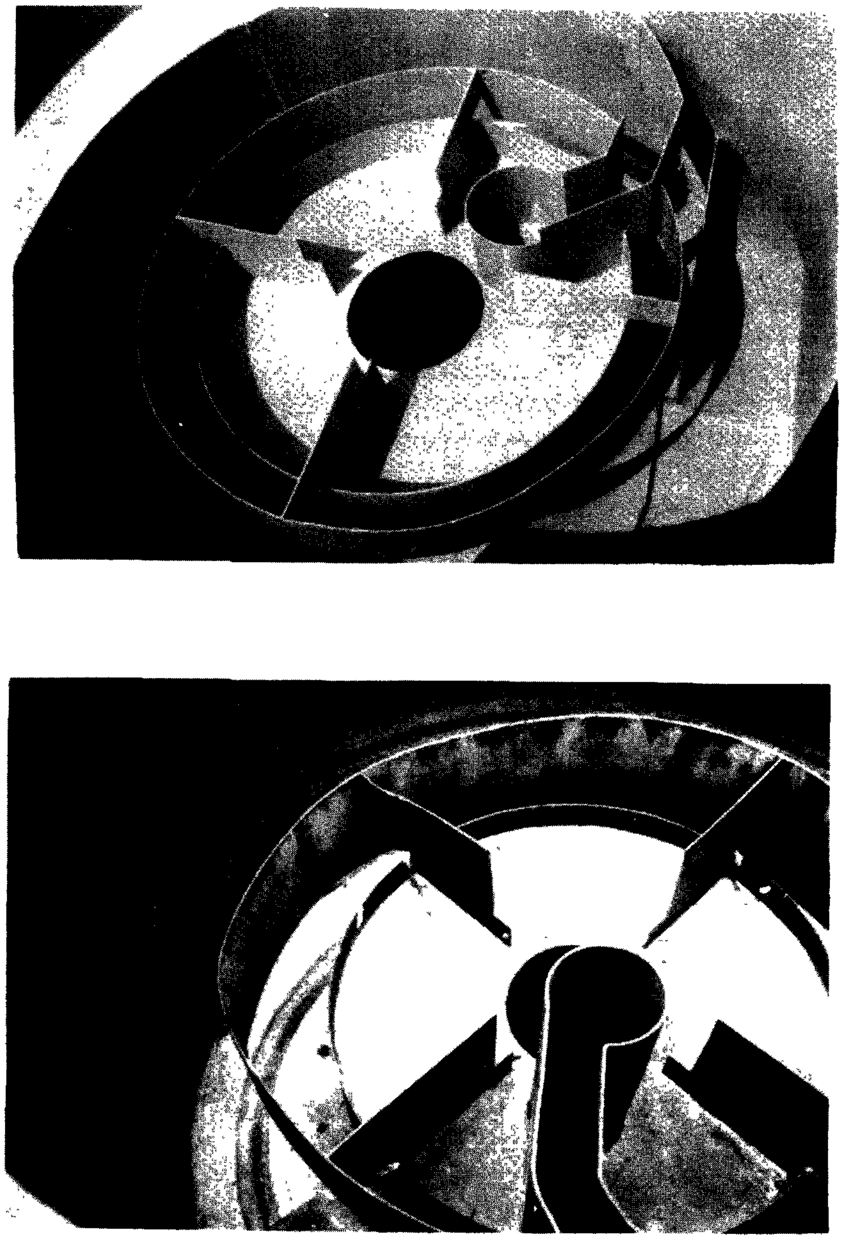


Figure 5 Photographs of Floatables Trap

The swirl device is capable of functioning efficiently up to twice the design flow and has the ability to separate larger settleable lightweight organic matter and floatable solids at a small fraction of the detention time required for primary separation--seconds to minutes as opposed to hours.

DESIGN GUIDELINES

The swirl unit was originally designed with a standard configuration to achieve 90 percent separation of settleable solids of a "typical" mixture of combined sewage. The design has been based upon the relationship of the diameter of the chamber, D_2, to the diameter of the inlet D_1. Subsequent work has allowed design flexibility by covering the ratio of D_2/D_1 from 4.5 to 12 and the efficiency of separation. Such flexibility has been desirable to allow the unit to be used where the loss of head through the unit must be considered, where the inlet diameter is fixed, to allow estimation of unit efficiency at greater than design flows, and to allow the use of more economically sized units if inlet diameter is fixed.

The design will be considered by the various factors which must be investigated and then an example will be given.

Hydraulics

Three flow quantities must be considered in the design: 1) the peak dry-weather flow; 2) the design flow, i.e., the flow for which the optimum treatment is desired; and 3) the maximum flow likely to occur through the chamber.

The peak dry-weather flow should be within the capacity of the gutter. The diameter of the foul outlet for the dry-weather flow should be a minimum of 20 cm (8 in.) and preferably be 25 to 30 cm (10 or 12 in.). At low flow rates, discharge through the outlet pipe may occur as gravity flow while at higher flows, discharge will occur as in a pressure pipe. It is difficult to size the pipe to act as a "throttle" pipe to pass a specific peak dry-weather flow. Therefore, it is recommended that a sluice gate or other flow control device be installed on the pipe in a manhole located outside the chamber. The use of a gate will permit adjustment of the opening and the discharge rate; further, it will allow the use of larger size pipe with less chance of clogging and, if clogging occurs at the gate, the gate can be opened to clear out the debris.

The use of a manually operated gate with a fixed opening (between adjustments) will result in considerable variation in the discharge rate through the outlet sewer due to variation in water level in the chamber.

Less variation in the discharge will occur if a tipping gate is used instead of a manual gate. However, this alternate would require the installation of two manholes to provide access to the upstream side as well as downstream side of the gate for maintenance purposes.

If it is necessary to limit the variation in flow of the foul sewage to a minimum, then a HydroBrake(R) as described in Section XI or a motor- or cylinder-operated gate should be used. Such gates could be controlled by either the downstream water level or the water level in the chamber. Electrical power would be required to operate the gate.

Tipping, motor-operated and cylinder-operated gates are described in the EPA Publication, Combined Sewer Regulation and Management, A Manual of Practice (15), and are not further considered in this report.

The maximum flow will determine the elevation of the chamber with respect to the inlet sewer. An important consideration is whether the inlet sewer can be surcharged and, if so, to what extent. Having determined the permissible water level at the inlet sewer, the circular weir must be set below this level so the weir discharge will equal the maximum design flow Equations are not available for determining the required head over the chamber weir; therefore, data obtained from the hydraulic model runs must be used. Stage discharge curves based on laboratory data are plotted on Figure 6 to indicate the discharge per unit length of weir.

Assuming the maximum flow 8.5 cu m/sec (300 cfs) and the circular weir length is 19.5 m (62.8 ft) discharge would be 435 l/s/m (4.3 cfs/ft). From Figure 6, this would indicate a head of

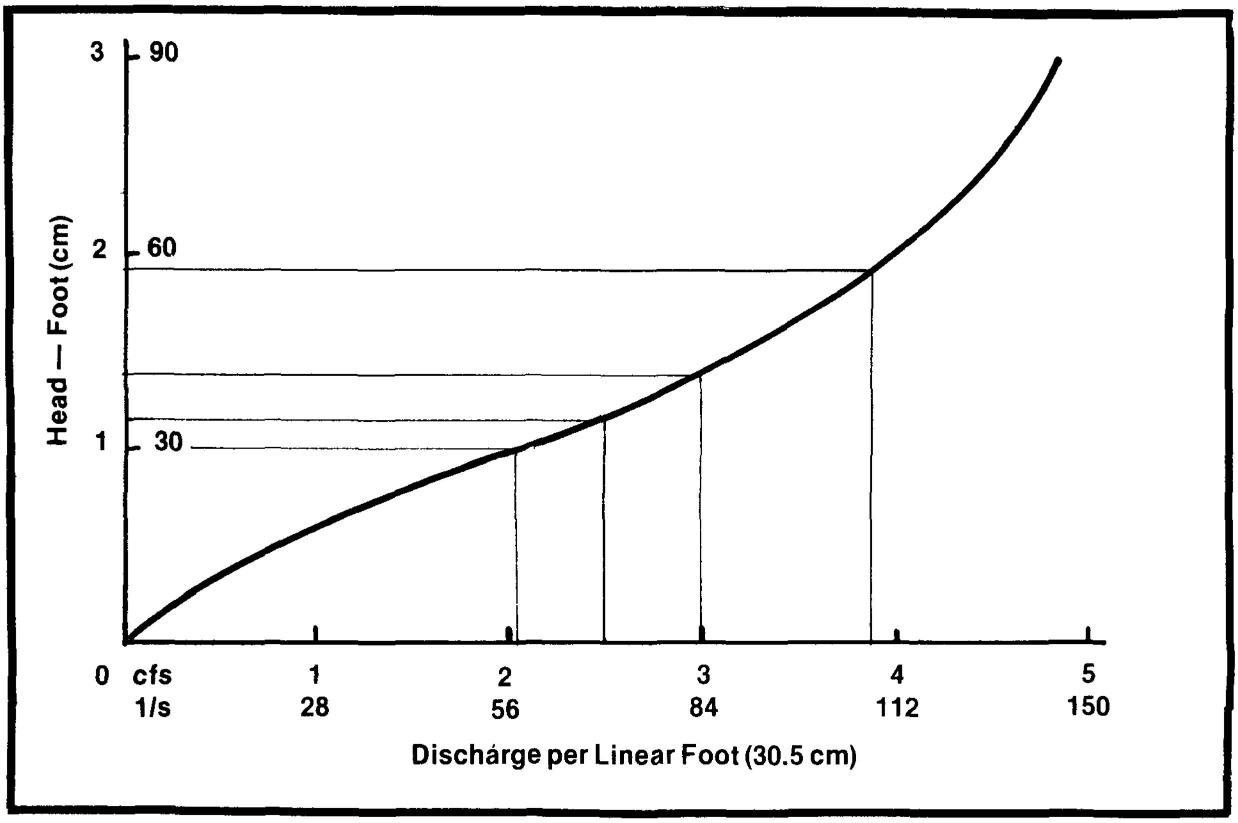


Figure 6 Head Discharge Curve for Circular Weir

91 cm (3 ft). Neglecting entrance losses, this would require that the weir crest be set 91 cm (3 ft) below the allowable hydraulic gradient of the inlet sewer.

In some cases it may be desirable to provide a side overflow weir on the periphery of the chamber to take part of the flow when the flow exceeds the design flow based on the minimum size necessary to achieve the desired removal of solids. As previously stated, the flow over the circular weir must not exceed twice the design flow.

It should be assumed that the discharge-head relations shown in Figure 6 are applicable to a side overflow weir. While this may not be correct, no better basis for estimating the flow is available.

General Details

Figure 7 presents the general relationship between the various parts of the units. In general, the larger the diameter of the unit, D_2, the larger the flow which can be treated. A low D_2/D_1 ratio of 4.5 to 6 indicates a minimum diameter deep unit, while a ratio approaching 12 indicates a wide, shallow unit. Note that all dimensions are in terms of D_2, D_1, or the ratio of D_2/D_1. The latter ratio is the key to the design.

Computation of Efficiency

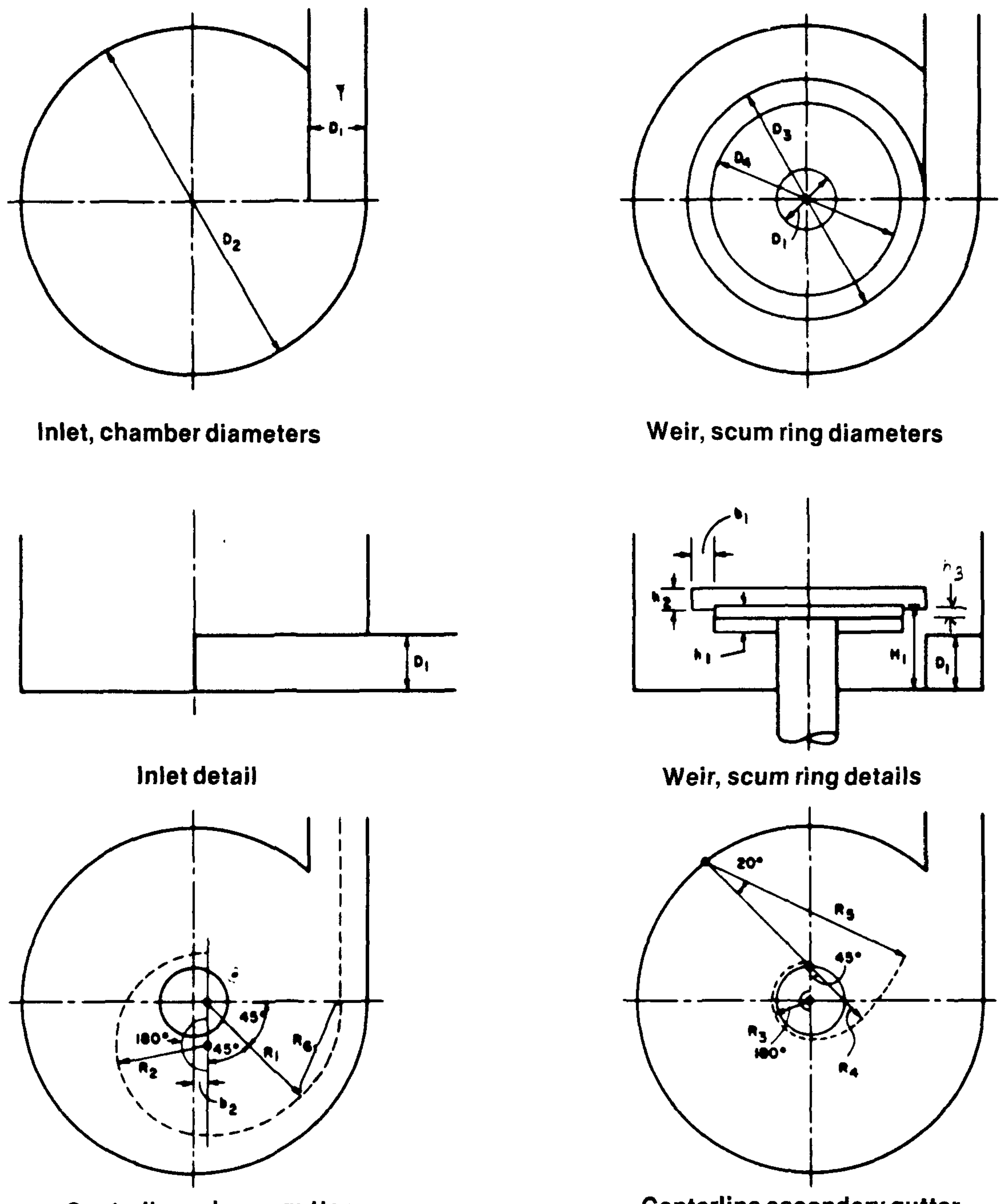
Efficiency of the swirl regulator/separator is based upon the volume of certain sized solids in the foul flow to the interceptor or treatment as compared to that in the clear overflow. Section VIII explains in detail the assumptions made upon which efficiency calculations are based. If reliable information concerning the specific gravity, grain size, and settling velocity of the solids in the combined sewage at a site is available, the standards of efficiency used in this report can be adjusted and a better estimate of efficiency made at a specific site.

Design Procedures

Field results are reported in a later portion of this section. The design procedure utilizes Figures 8 to 11 which allows rapid computation of the essential dimensions upon selection of desired efficiency and the quantity to be treated. The following procedure should be used:

1. Select Design Discharge: The design engineer must select the design discharge appropriate to each project, based on the design criteria for the project.

2. Select the Recovery Efficiency Desired: One of four performance efficiencies can be chosen--either 100, 90, 80 or 70 percent recovery of settleable solids. It is suggested that 90 percent settleable solids recovery be taken for design storm discharges. Only in cases



D_1 = from Figure 9, 10, 11 or 12
D_2 = from Figure 9, 10, 11, or 12
H_1 = $D_2/4$ or from Figure 12
D_3 = $2/3\ D_2$
b_2 = $D_1/6$
D_4 = $5/9\ D_2$
h_1 = $D_1/2$
h_2 = $D_1/3$
h_3 = $D_2/18$

R_1 = $7/18\ D_2$
R_2 = $D_2/4$ [min R_2 = 45cm (18 in)]
R_3 = $5/48\ D_2$
R_4 = $3/16\ D_2$
R_5 = $11/18\ D_2$
R_6 = Curve smoothed in to meet inlet centerline

Figure 7 General Design Details, Swirl Combined Sewer Overflow Regulator/Separator

where low probability peak flows are being considered would it be reasonable to design on the basis of 80 percent or 70 percent recovery.

3. Find the Inlet Dimension--D_1, and Chamber Diameter--D_2: Having selected the desired unit efficiency and the design flow, use Figures 8 to 11. With the discharge set, cross horizontally and intercept the curved line which represents the inlet diameter D_1. The ratio of D_2/D_1 can be immediately determined.

When head loss for the dry-weather flow is not a consideration, it may be desirable to select a low ratio, say 4.5 to 6. If head loss is of concern, a higher ratio should be considered. Inlet sizes to be used with the various D_2/D_1 ratios can be interpolated between the inlet diameters given. It may also be desirable to select a larger or smaller D_1 to coincide with the diameter of the inlet sewer. Figures 8 to 11 are based upon a standard ratio of the height of the unit from the overflow weir to the bottom (H_1) to the diameter of the unit (D_2) of 0.25.

Where it is desired to modify the chamber dimensions to minimize the weir height, Figure 12 may be used. Use of these curves presumes that the inlet dimension will be retained and that the weir height and chamber diameter will be modified.

Where the square inlet dimension cannot conveniently be made the same as the inlet sewer, a reducing or expanding adapter or conversion section would be necessary to ensure obtaining the efficiencies given in the curves. If the inlet sewer is concentrically aligned with the swirl chamber inlet, the transition section should have a length of at least three to six times D_1 (i.e. $3D_1$ to $6D_1$).

Another possibility would be to provide for the inlet sewer to discharge into an inspection manhole. From the manhole, a conduit with a square cross section would be provided to the swirl chamber. The distance from this conversion manhole to the square inlet discharge into the chamber should also be a minimum of three to six times D_1. This manhole arrangement could be used to provide for change in the alignment, elevation, or size between the inlet sewer and the square inlet into the swirl chamber. This chamber could also be used for a bar screen to prevent large floatables and objects from blocking the floatables trap and foul outlet.

4. Check Discharge Range Covered: The anticipated efficiency at various flow rates can be determined for different sized inlets and chambers by using Figures 13 to 15. If the selected D_1 curve is not shown in the figure, its recovery line can be interpolated and drawn between the given curves.

5. Recovery Rates: On Figures 13 to 15 for the values in excess of 90 percent recovery, two curves, one dashed and one full line, have been drawn, The dashed line is the extrapolated curve taken from the

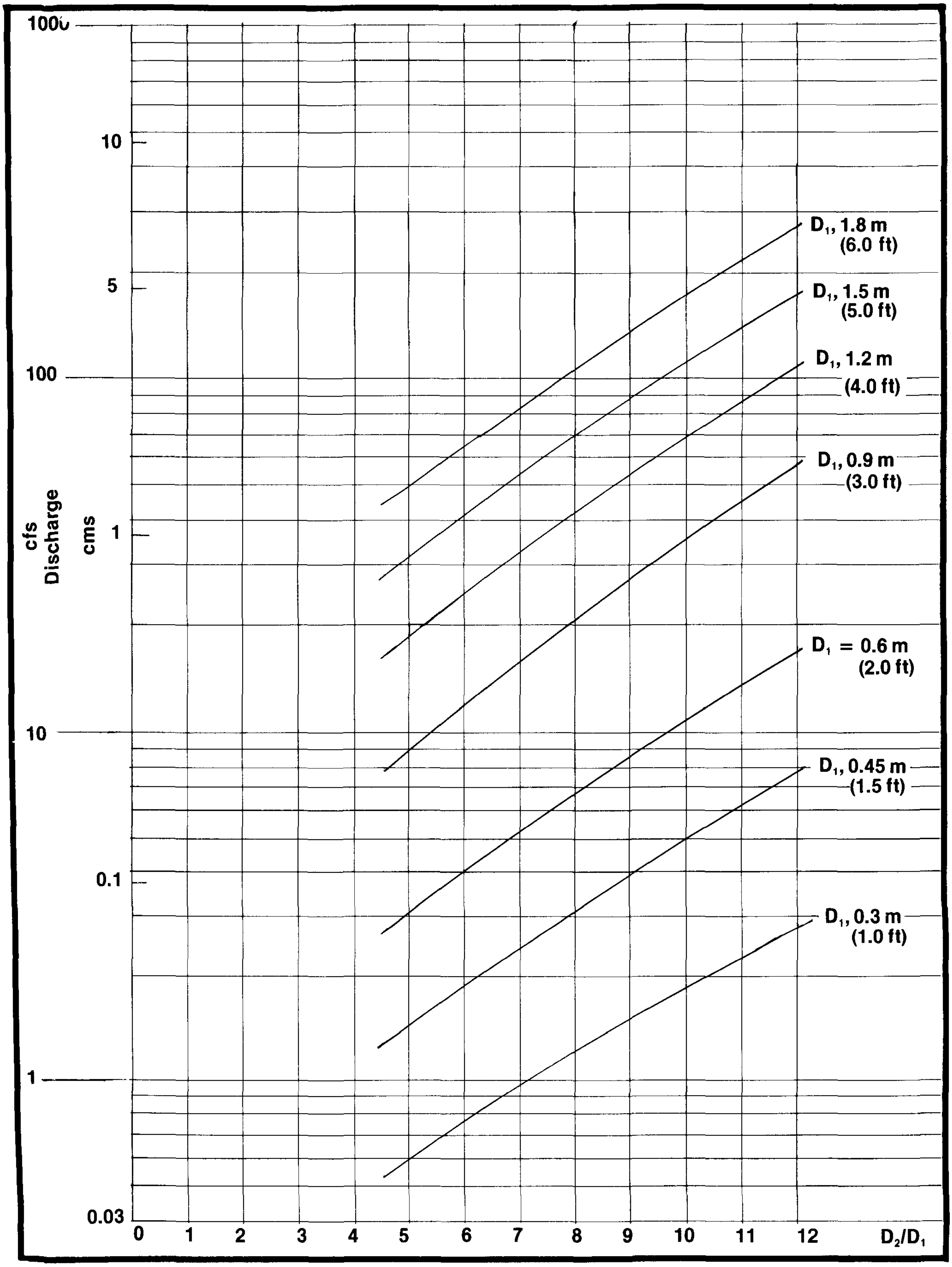


Figure 8 D_2/D_1 Discharge for 100% Recovery

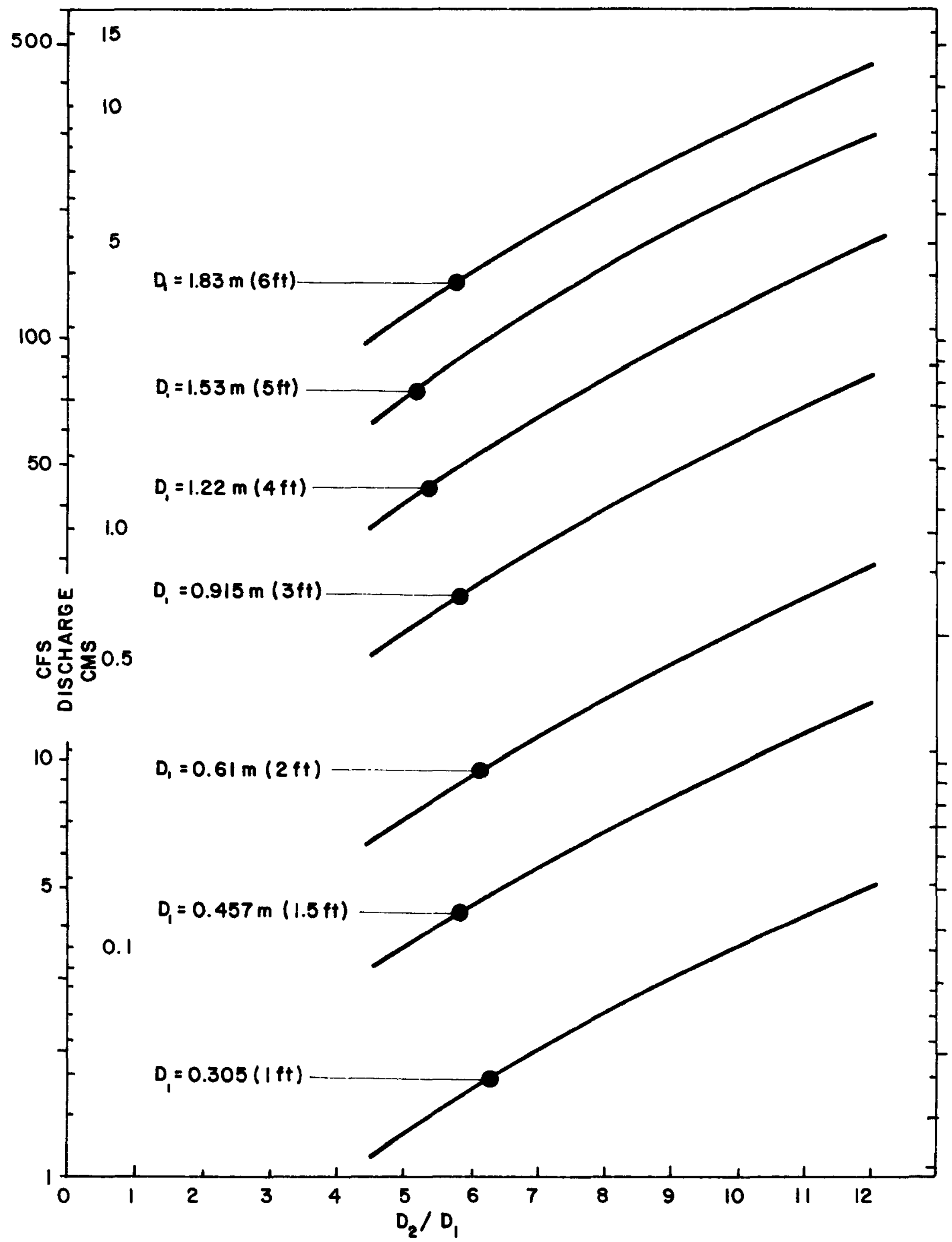


Figure 9 D_2/D_1 Discharge for 90% Recovery

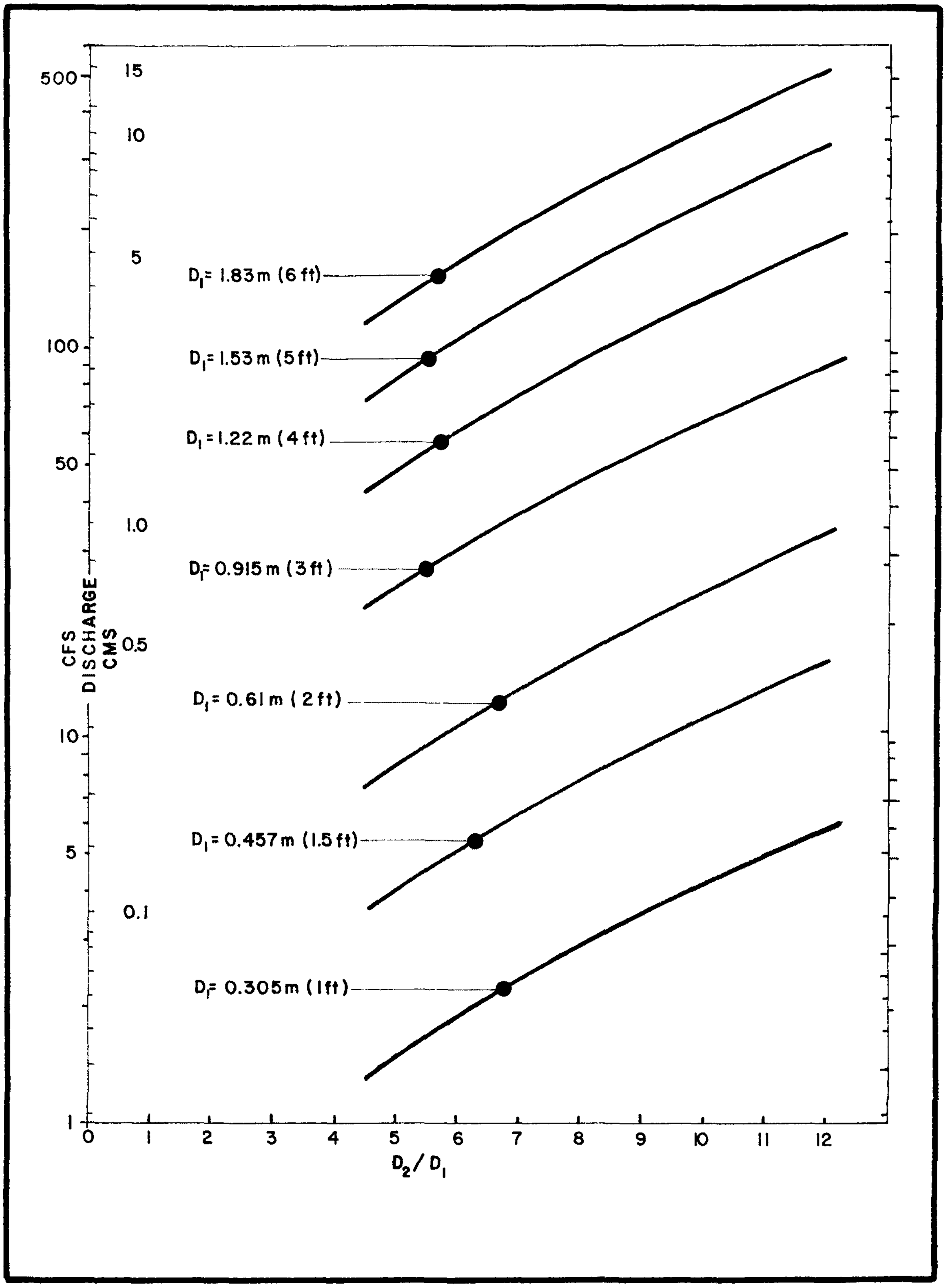


Figure 10 D_2/D_1 Discharge for 80% Recovery

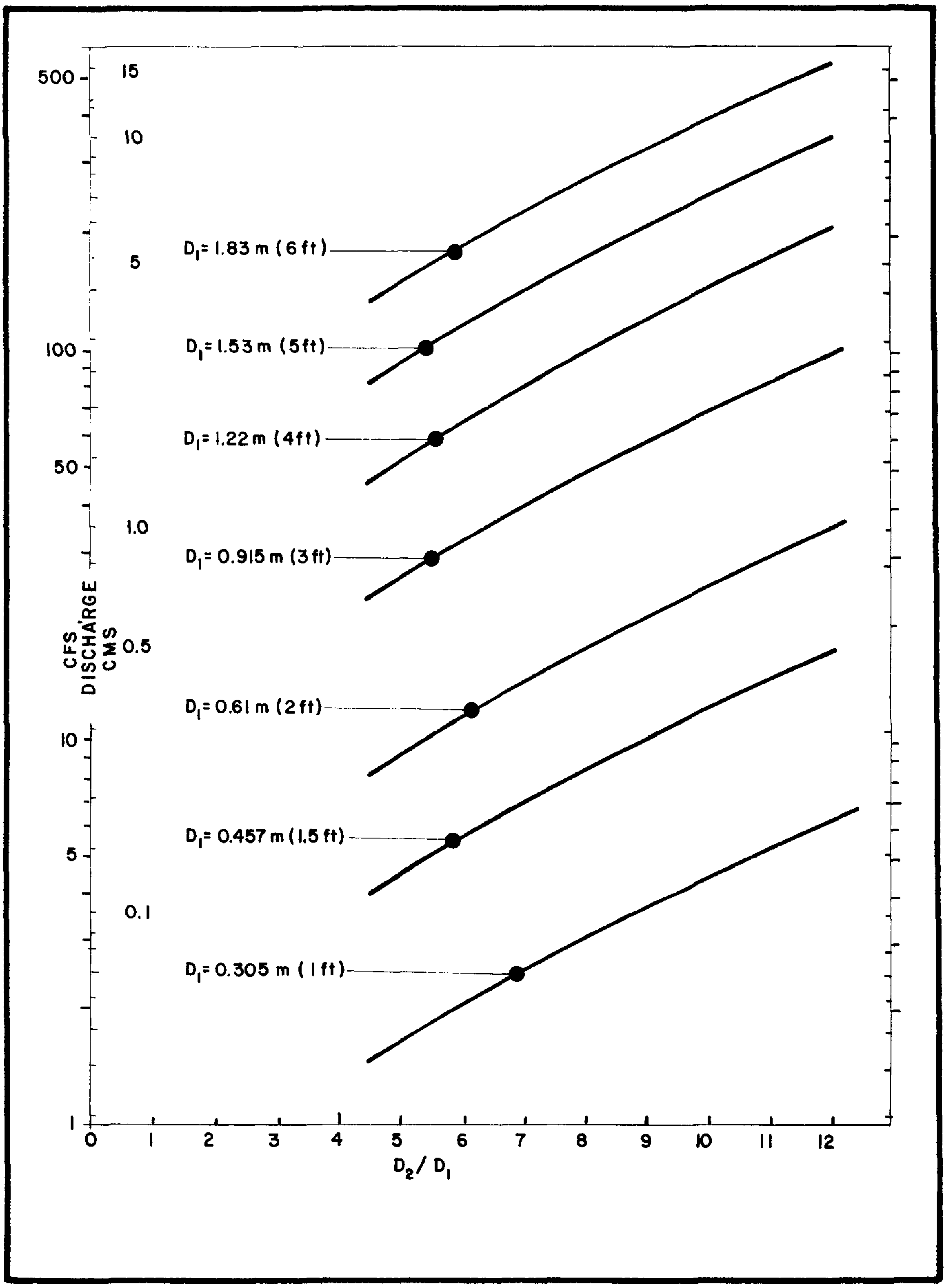


Figure 11 D_2/D_1 Discharge for 70% Recovery

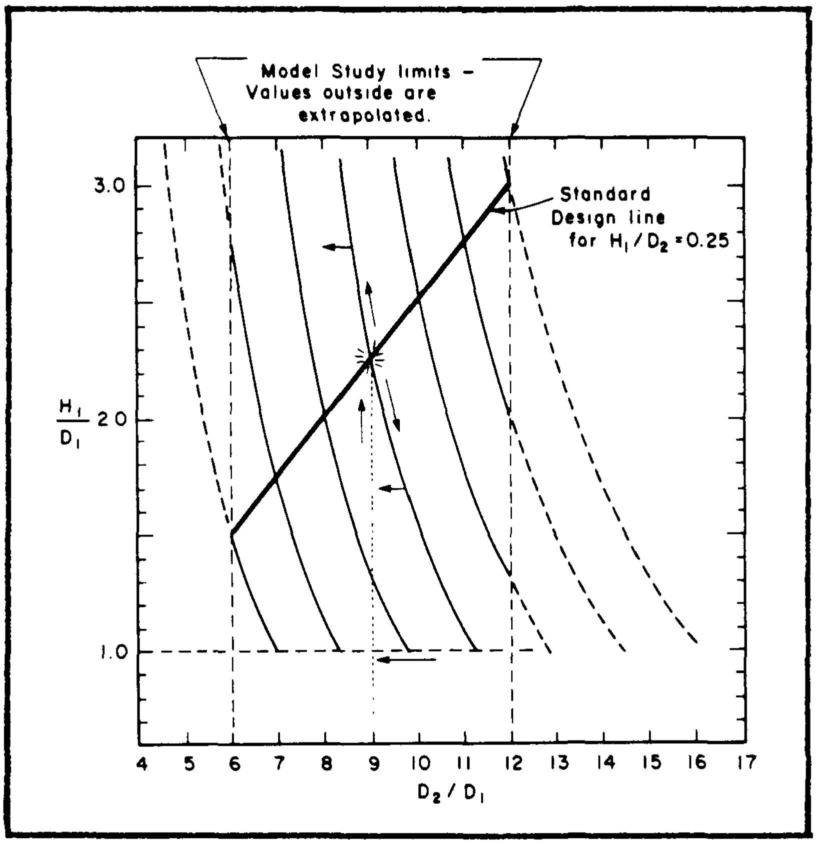


Figure 12 Geometric Modification Curve

laboratory data obtained for 70, 80, and 90 percent recovery results. The full line gives the measured 100 percent recovery results.

During hydraulic model tests it was found that to obtain recoveries in excess of 95 percent, a marked reduction in discharge was required. However, the maximum discharge to give 100 percent recovery was never actually determined as the swirl unit was not "forced" at this recovery rate. Therefore, the true discharge lies somewhere between the full and dashed lines.

An alternate set of curves, Figures 16 to 25, are given to assist in interpolating efficiencies for intermediate ratios of D_2/D_1.

Bearing in mind the lack of information at this recovery rate a conservative approach was taken for 100 percent recovery using the laboratorv results, i.e., the solid line values.

The recovery rates over the range of discharges represented by the sewer hydrograph should be checked, including the design discharge. The designer must determine at this stage that the discharge range and recovery rates are adequate, or carry out further adjustments in D_1 and D_2 dimensions through steps 2, 3 and 4 until they are adequate.

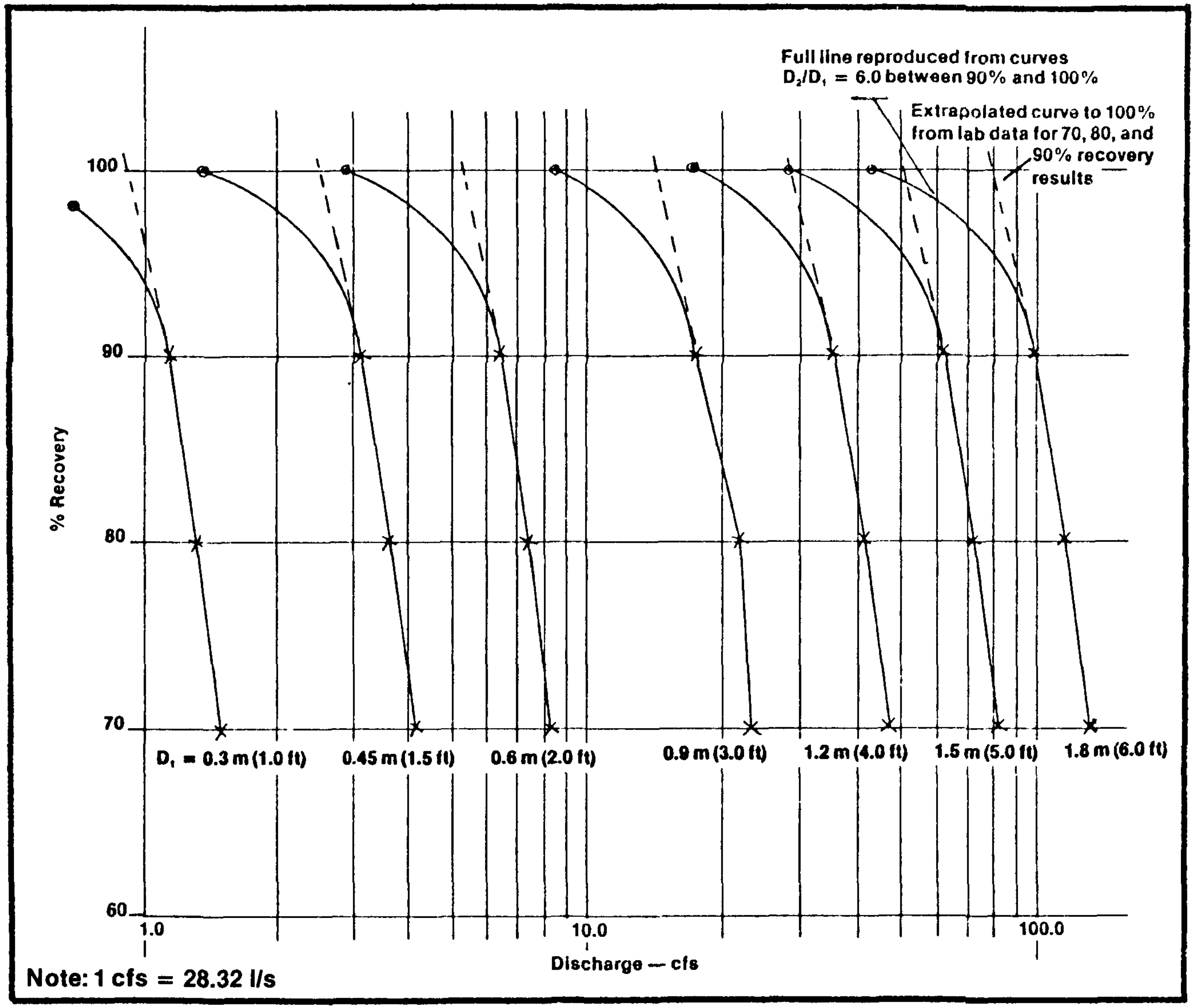


Figure 13 Settleable Solids Percent Recovery vs Discharge for D_2/D_1 = 4.5

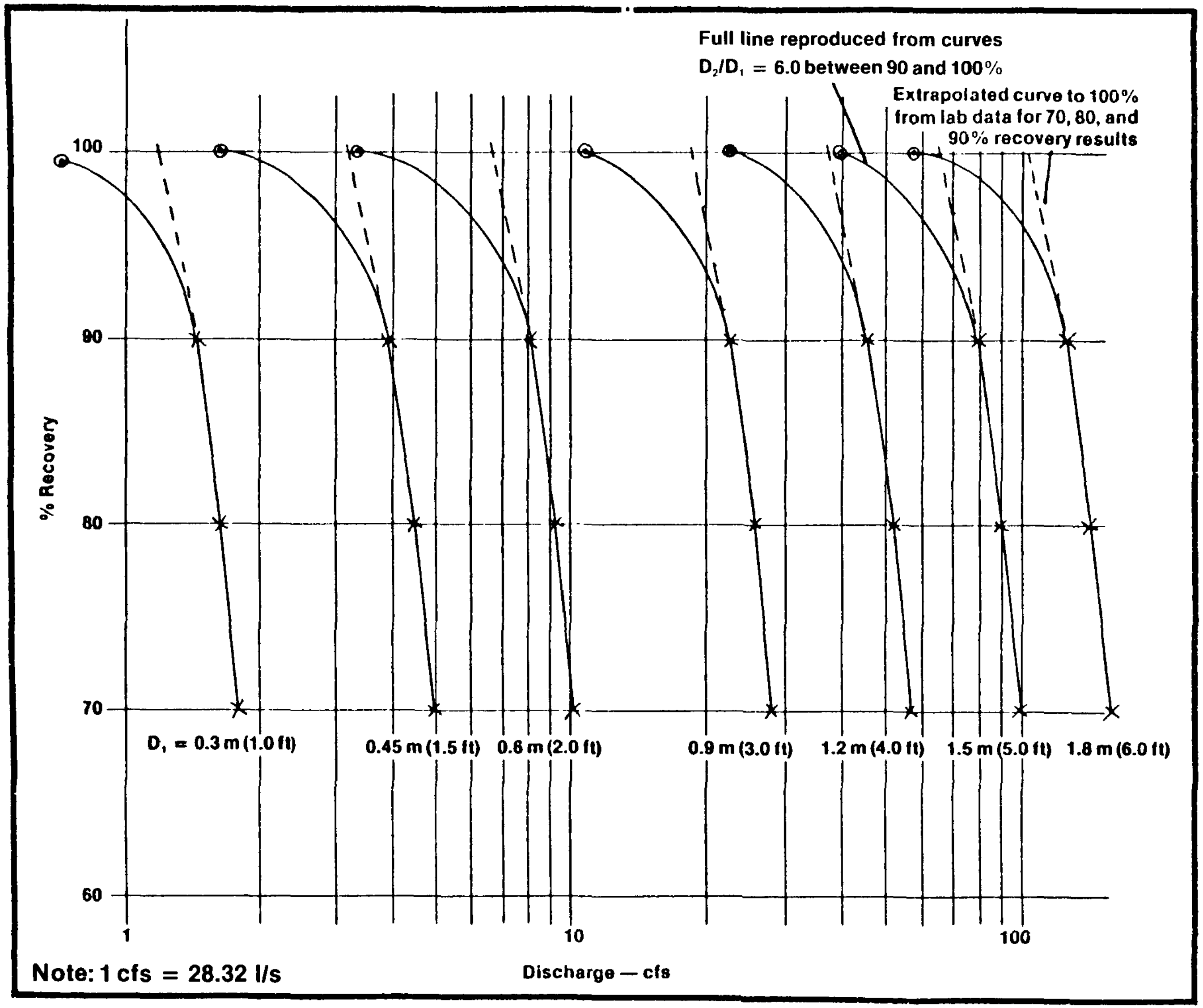


Figure 14 Settleable Solids Percent Recovery vs Discharge for D_2/D_1 = 5.25

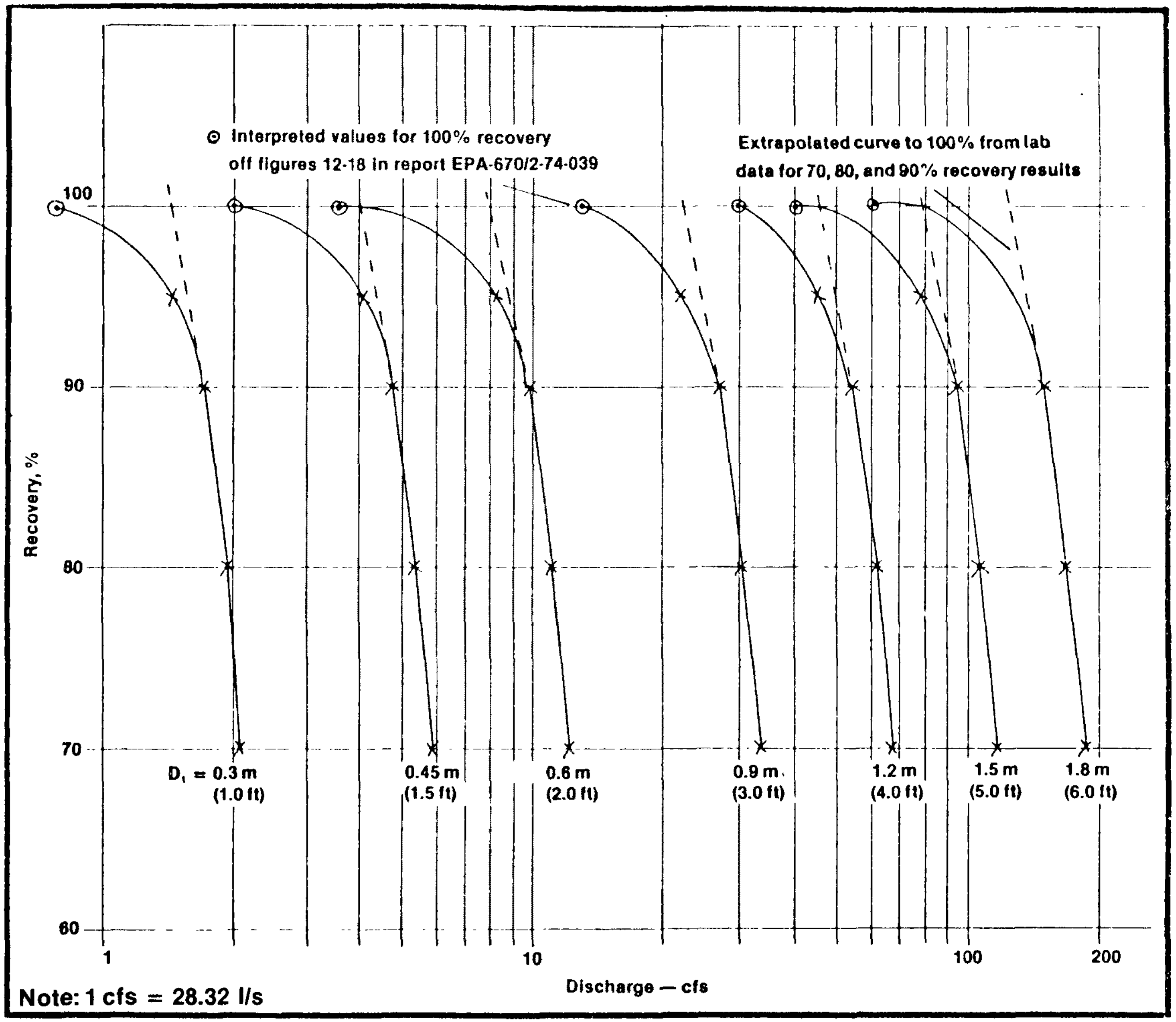


Figure 15 Settleable Solids Percent Recovery vs Discharge for D_2/D_1 = 6.0

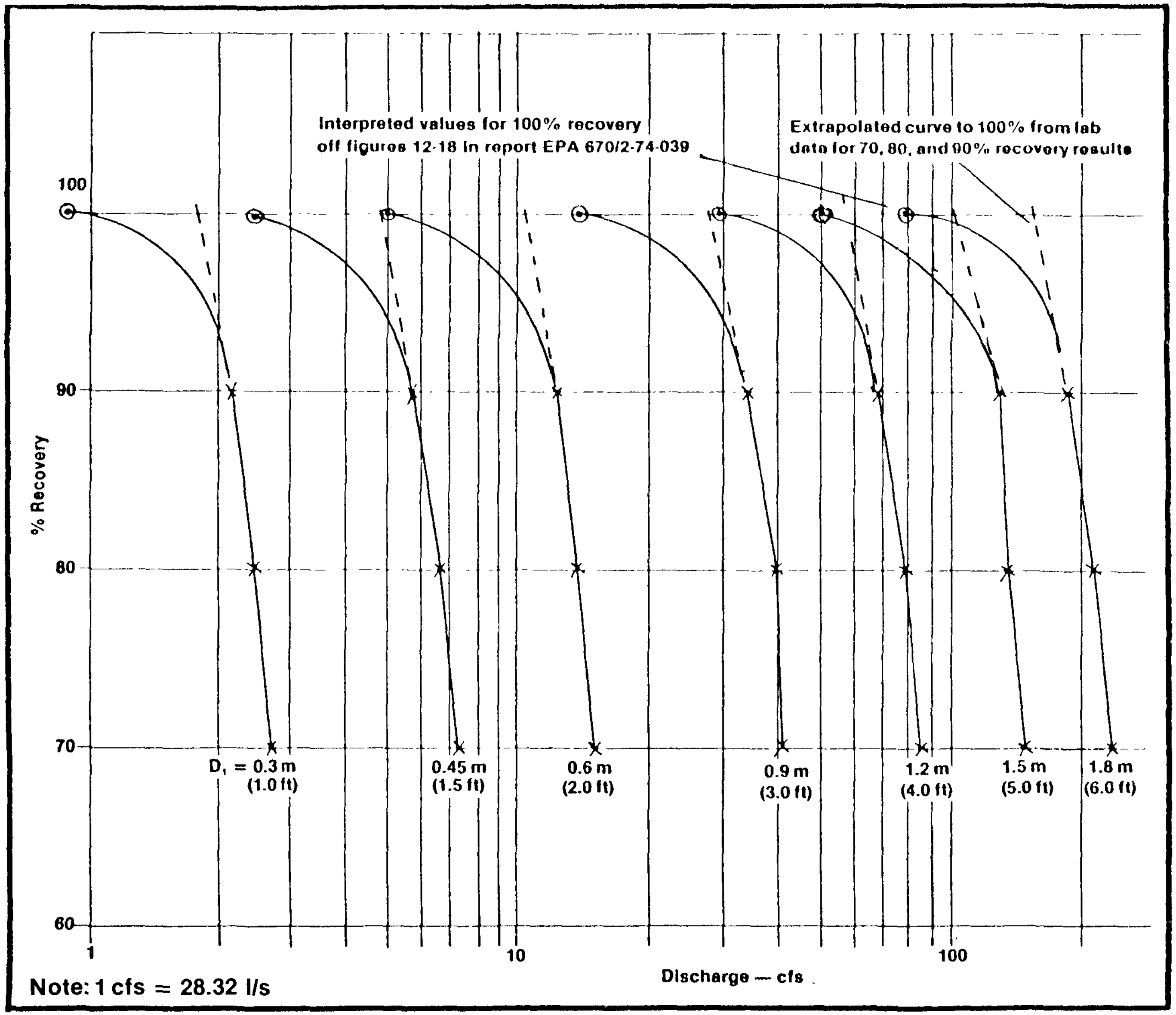


Figure 16 Settleable Solids Percent Recovery vs Discharge for D_2/D_1 = 7.2

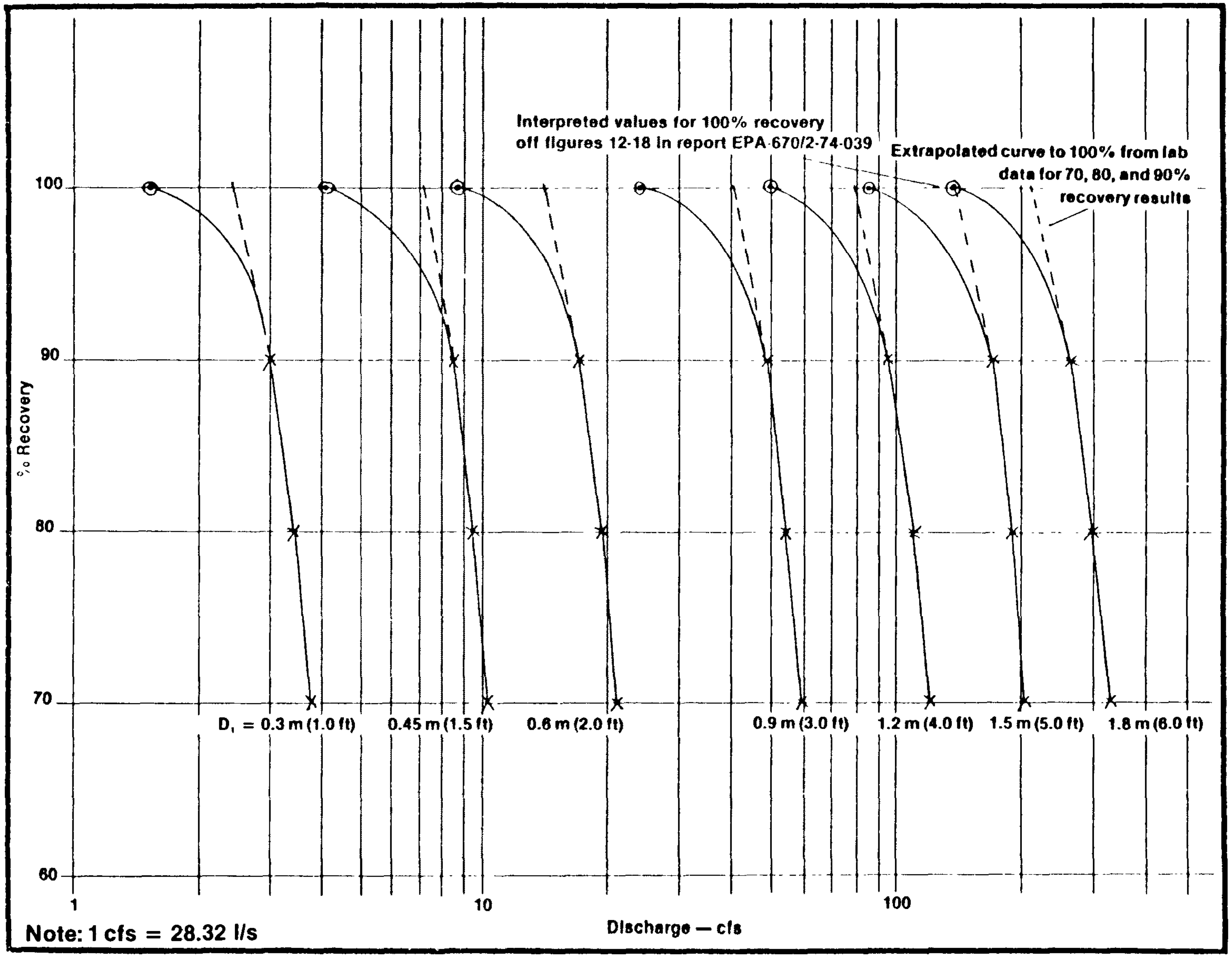


Figure 17 Settleable Solids Percent Recovery vs Discharge for D_2/D_1 = 9.0

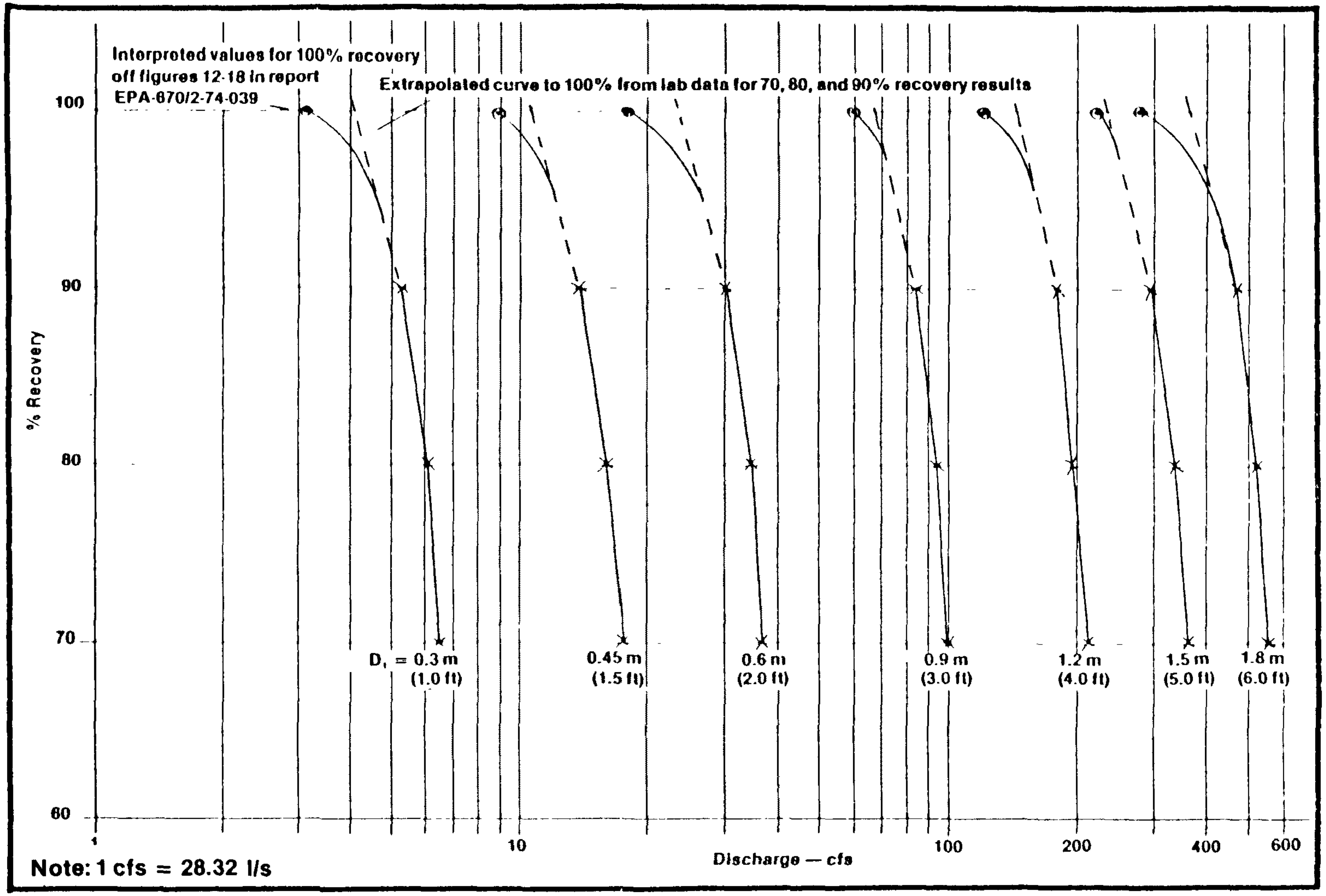


Figure 18 Settleable Solids Percent Recovery vs Discharge for D_2/D_1 = 12.0

Using the D_2/D_1 abscissa chosen for the standard design above, move vertically to the intersection with the bold standard design line to locate the working point. Constant operating conditions for the specific design then lie on the geometry modification curve passing through the working point. Moving down to the right corresponds to increasing the chamber diameter or width and lowering the weir height or chamber depth. Moving up to the left reduces the chamber diameter and increases the weir height.

Any choice of D_2D_1 or H_1/D_1 relationship can then be made, and the corresponding values found. It will then be necessary to redimension the other elements of the structure, based on the general design details in Figure 7.

6. Foul Discharge: The unit was designed to operate on the assumption that 3DWF would be handled through the foul outlet for discharge to the wastewater treatment facility. Greater Solids removal efficiency can be obtained with larger discharges as explained in 9 below.

7. Find Dimensions for the Whole Structure: Having made decisions on acceptable D_1 and D_2 values, these can be applied to Figure 7 to determine the necessary dimensions for all the features of the entire swirl chamber.

8. Geometry Modifications: The above steps have provided the geometric configurations to meet the design hydraulic conditions. However, at this stage other considerations such as available space, depth or head, or economic factors, might make it desirable to modify the general proportions of the chamber. The same operating conditions can be obtained if the geometry is modified according to Figures 8 to 11. This procedure assumes that the inlet dimension D_1 is retained from the above procedures, and that the chamber diameter and weir height would be modified.

9. Foul Discharge Modification: There may be a reason to allow a greater foul discharge than 3DWF. If the diameter of the foul outlet is increased from 30 cm (1 ft) to 91 cm (3 ft) there will be a marginal drop in efficiency, perhaps 3 percent. Figures 26 to 28 reflect the changes in this unit's efficiency as the percentage of flow to the foul sewer is increased to 50 percent of the total flow for D_2/D_1 ratios of 4.5 to 6.0.

These curves may be used to evaluate efficiency of separation at various stages of the unit's operation on either the rising or falling stage of the storm hydrograph. Laboratory data is not available for D_2/D_1 ratios above 6.

It must be realized that a major portion of the increased efficiency at higher foul sewer flows is due to the increased flow split, and not to the swirl concentration treatment received.

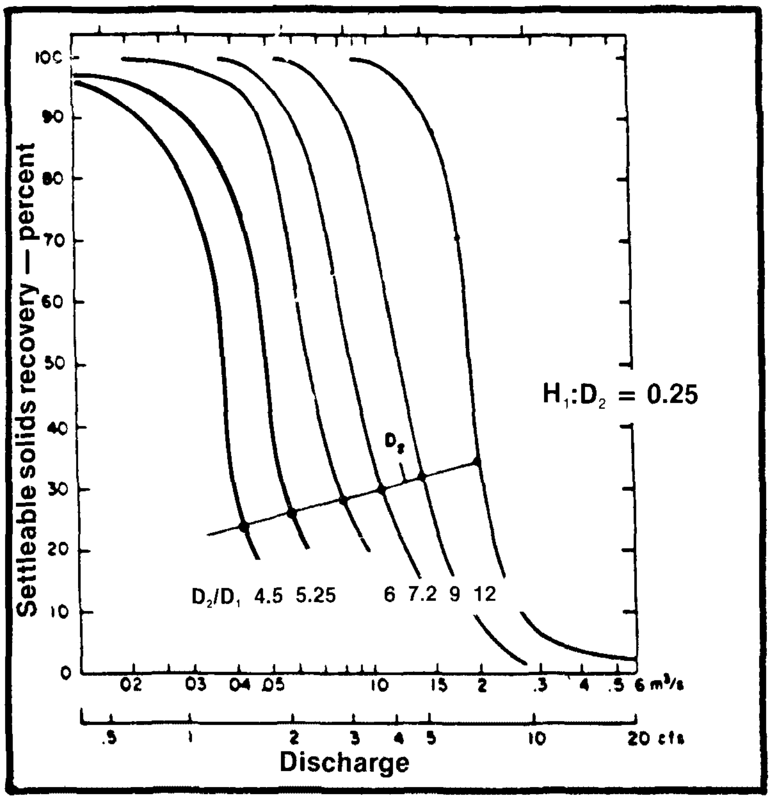


Figure 19 Settleable Solids Percent Recovery vs Discharge for Inlet Diameter of 30.5 cm (1 ft)

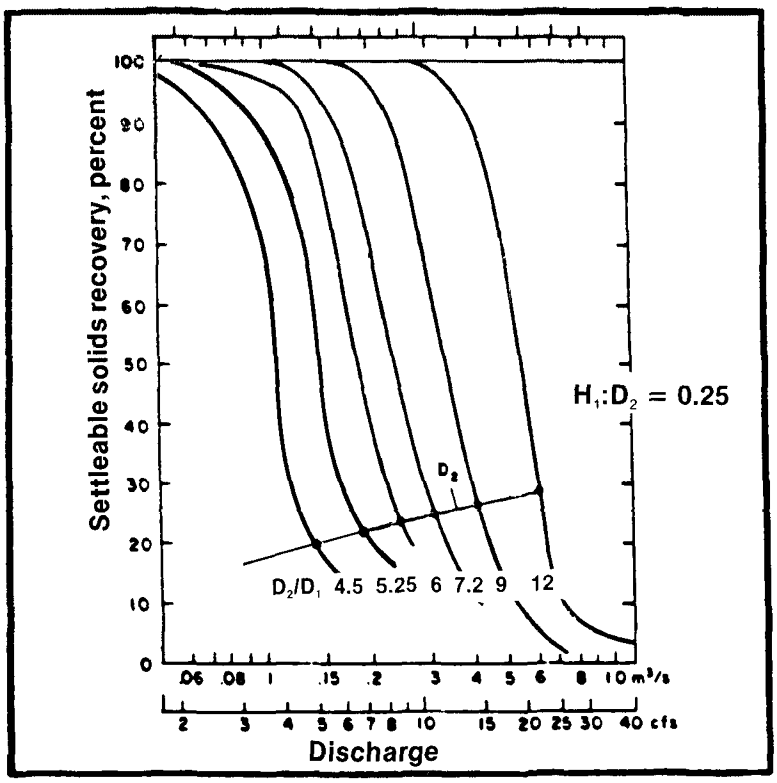


Figure 20 Settleable Solids Percent Recovery vs Discharge for Inlet Diameter of 45.8 cm (1.5 ft)

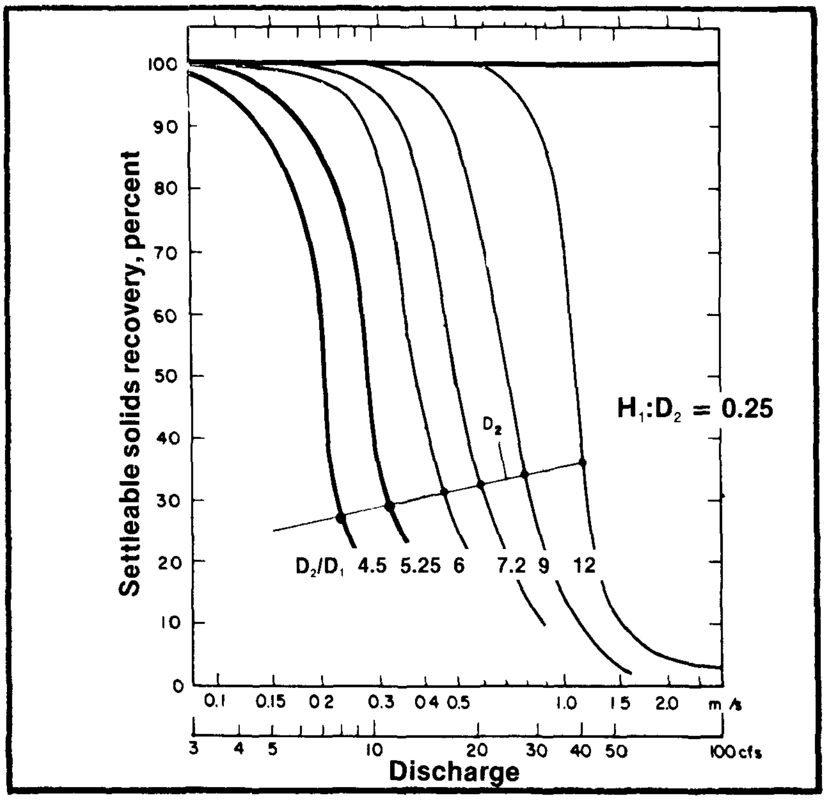


Figure 21 Settleable Solids Percent Recovery vs Discharge for Inlet Diameter of 61.5 cm (2 ft)

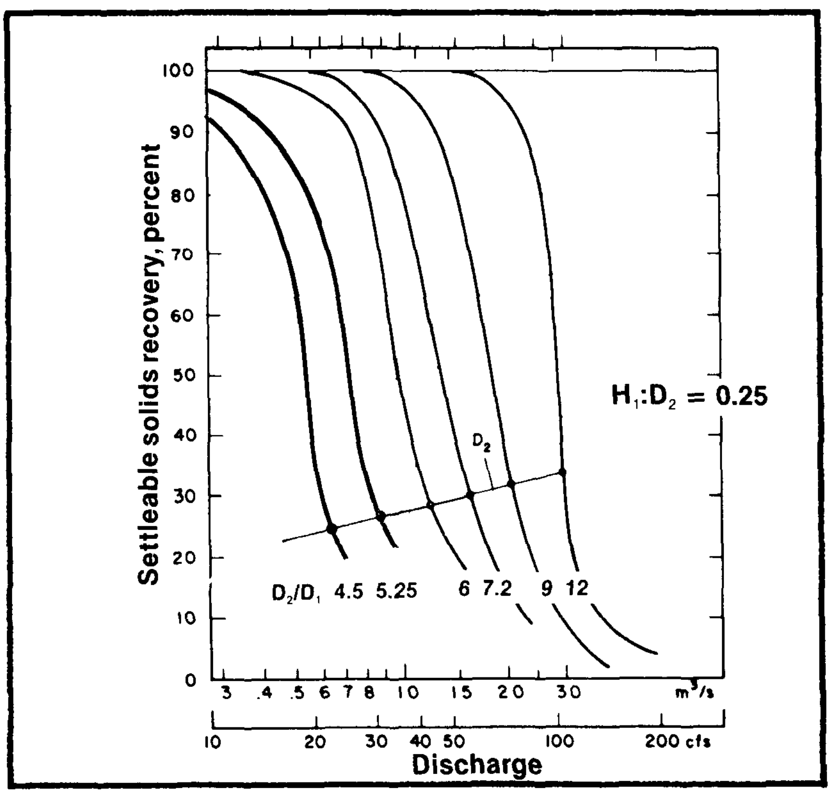


Figure 22 Settleable Solids Percent Recovery vs Discharge for Inlet Diameter of 91.5 cm (3 ft)

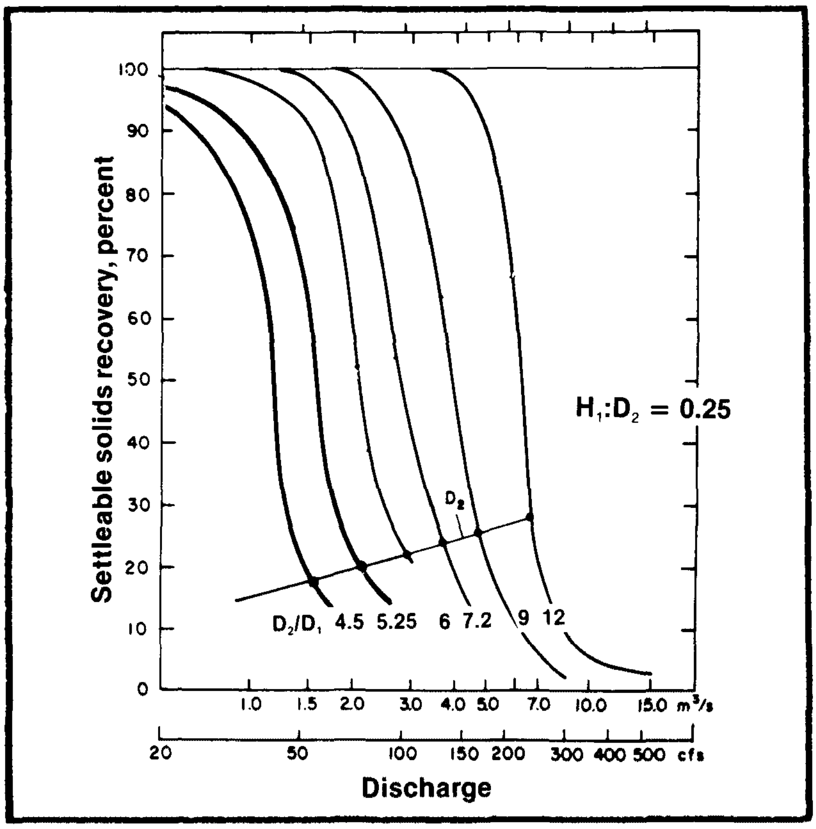


Figure 23 Settleable Solids Percent Recovery vs Discharge for Inlet Diameter of 122.0 cm (4 ft)

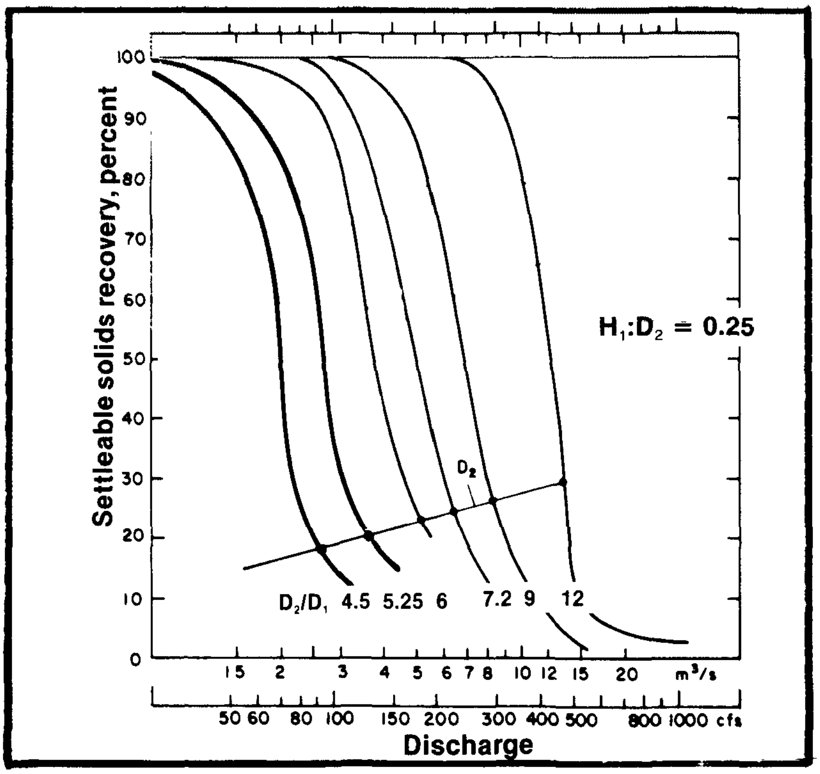


Figure 24 Settleable Solids Percent Recovery vs Discharge for Inlet Diameter of 152.5 cm (5 ft)

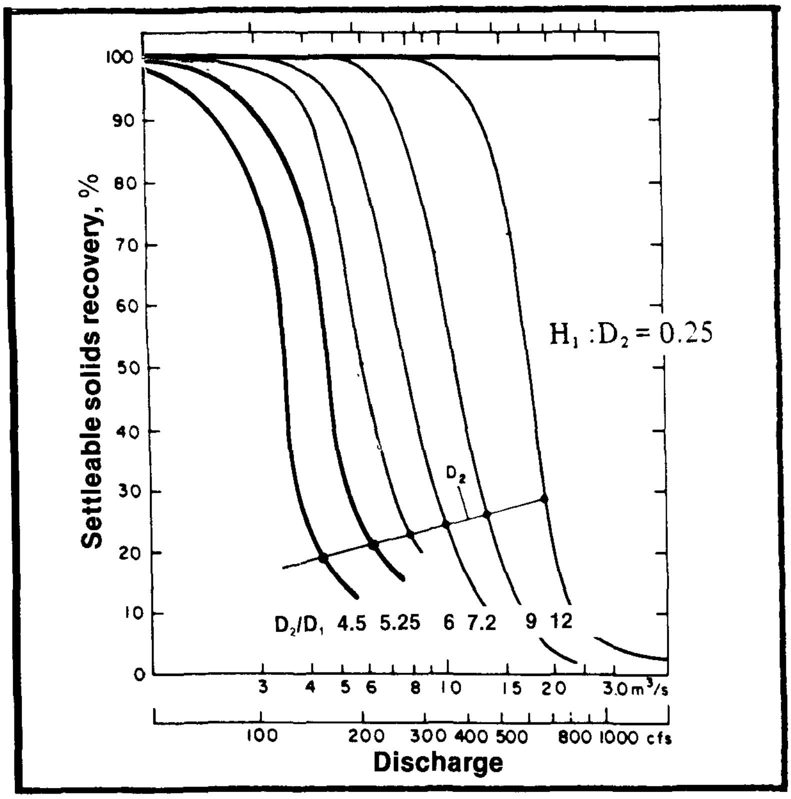
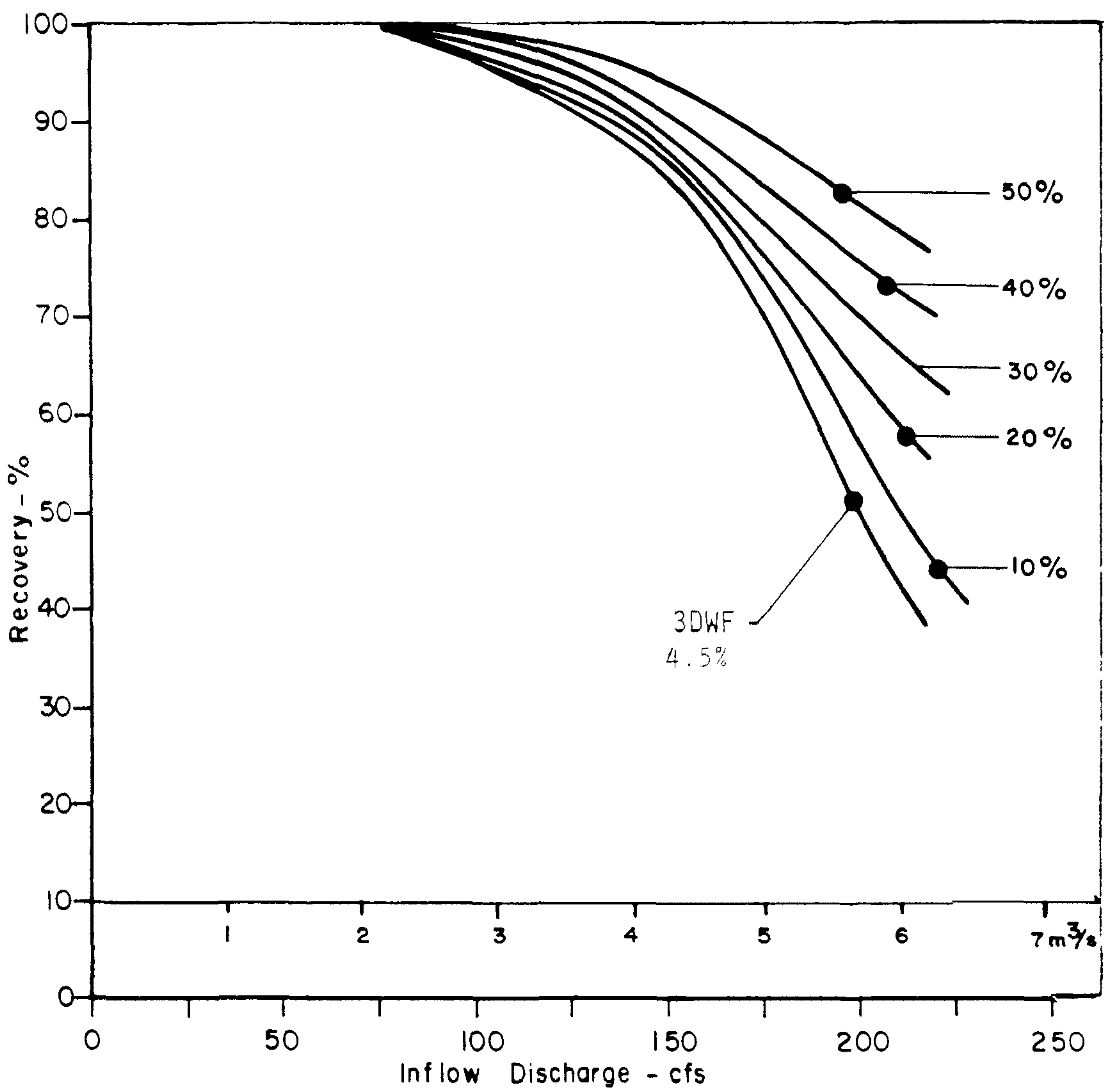


Figure 25 Settleable Solids Percent Recovery vs Discharge for Inlet Diameter of 183.0 cm (6 ft)

Design Example

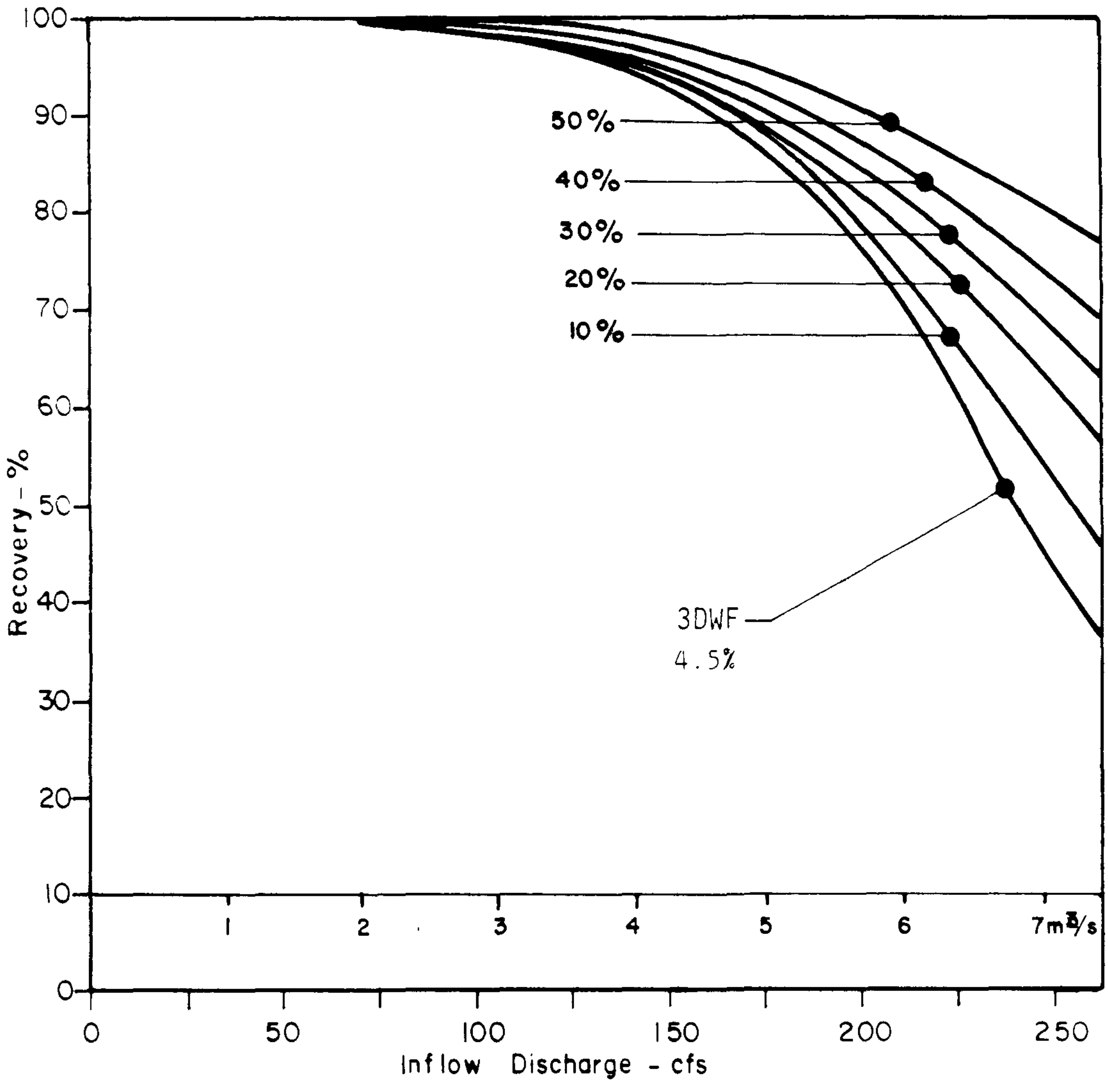
Table 1 illustrates the design procedure. Item 1 is the design discharge. Item 2 is the design settleable solids recovery efficiency. Item 3 is the possible inlet diameters selected from Figure 9. Item 4 is the ratio of D_2/D_1 from Figure 9. Item 5 is the computed chamber diameter D_2. Item 6 is the actual recovery efficiency obtained from Figures 23 to 25. If the recovery efficiency is below 90 percent a greater diameter (D_2) must be selected from Figures 13 to 18 or 19 to 23 to conform with 90 percent recovery. The revised chamber diameter is shown in Item 7. The final design chamber diameter is given in Item 8. The ratio of chamber height of weir (H_1) to width (D_2) in Item 9 is equal to 0.25.

The inlet velocity is shown in Item 10. It is evident that where there is a choice of inlet sizes, the largest inlet size will result in the lowest inlet velocity, and the smallest and most economical structure. Hence, the designer should select the largest inlet size shown on the design



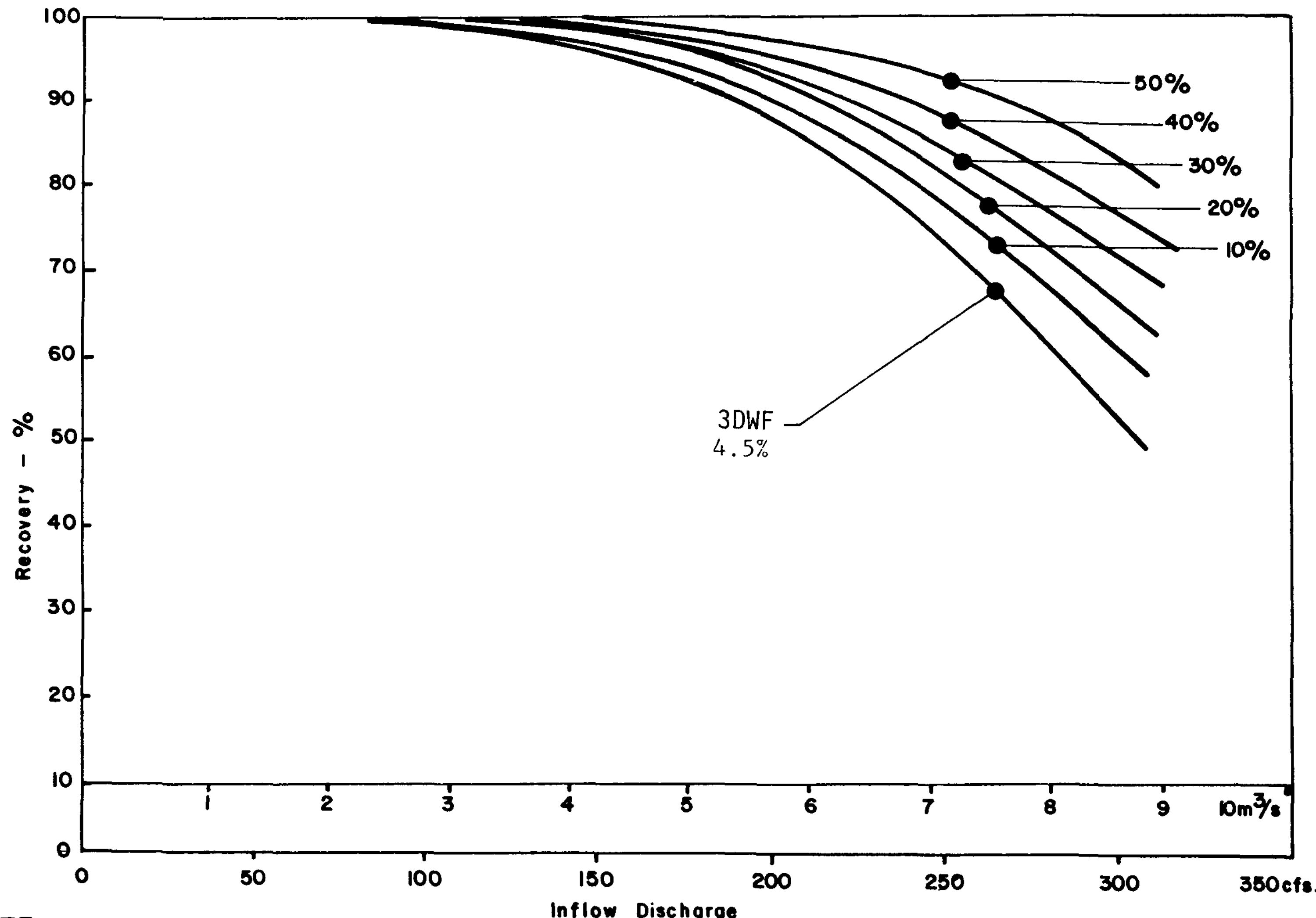
NOTE :
Percentages shown on curves denote portion of inflow drawn through foul outlet.

Figure 26 Efficiency of Separation of Higher Foul Sewer Discharges, $D_2/D_1 = 6$



NOTE : Percentages shown on curves denote portion of inflow drawn through foul outlet.

Figure 27 Efficiency of Separation of Higher Foul Sewer Discharges, D_2/D_1 = 5.25



NOTE : Percentages shown on curves denote portion of inflow drawn through foul outlet.

Figure 28 Efficiency of Separation of Higher Foul Sewer Discharges, $D_2/D_1 = 4.5$

Table 1
Design Procedures for Swirl Combined Sewer Overflow Regulator/Separator

		Design Discharge m^3/s (cfs)						
1. Design Discharge		1.42 (50)		2.83 (100)			4.67 (165)	
2. Operating efficiency	%	90	90	90	90	90	90	90
3. Inlet D_1 (Fig. 9)	m	1.2	1.8	1.5	1.2	1.8	1.5	1.2
	(ft)	(4)	(6)	(5)	(4)	(6)	(5)	(4)
4. D_2/D_1 (Fig. 9)	—	5.8	4.4	6.0	9.0	6.4	8.3	11.6
5. Diameter D_2	m	7.0	7.9	9.1	11.0	11.6	12.5	14.0
	(ft)	(23)	(26)	(30)	(36)	(38)	(41)	(46)
6. Recovery (Fig. 23)	%	90			88			78
(Fig. 24)	%			85		—	63	
(Fig. 25)	%		NA	—		90		
7. Revised D_2 (Fig. 23)	m			—	11.3			14.9
	(ft)			—	(37)			(49)
(Fig. 24)	m			9.8			14.6	—
	(ft)			(32)			(48)	—
8. Design diameter D_2	m	7.0		9.8	11.3	11.6	14.6	14.9
	(ft)	(23)		(32)	(37)	(38)	(48)	(49)
9. Depth H_1(0.25 D_2)	m	1.8		2.4	2.8	2.9	3.6	3.7
	(ft)	(5.8)		(6.4)	(9.2)	(9.5)	(12)	(12.2)
10. Inlet Velocity	cm/s	94		122	189	140	201	314
	(fps)	(3.1)		(4.0)	(6.2)	(4.6)	(6.6)	(10.3)
11. Final D_2/D_1	—	5.8		6.4	9.2	6.4	9.6	12.2
12. Design modification (H_1 = D_1)	m	1.2		1.5	1.2	1.8	1.5	1.4
	(ft)	(4)		(5)	(4)	(6)	(5)	(4)
Revised H_1/D_1 Fig. 12	—	1.0		1.0	1.0	1.0	1.0	1.0
Revised D_2/D_1 Fig. 12	—	7.7		7.6	11.5	7.6	1.2	NA
Revised D_2	m	8.2		11.6	14.0	13.7	18.3	
	(ft)	(27)		(38)	(46)	(45)	(60)	
13. Maximum discharge (2 × design discharge)	m^3/s	2.84		5.66	5.66	9.34	9.34	9.34
	(cfs)	(100)		(200)	(200)	(310)	(310)	(310)
14. D_4 (Fig. 7)	m	4.57		6.43	7.77	7.62	10.16	8.30
	(ft)	(15)		(21.1)	(25.5)	(25)	(33.3)	(27.2)
15. Length of Weir	m	14.36		20.21	24.48	23.93	31.88	26.1
	(ft)	(47.1)		(66.3)	(80.3)	(78.5)	(104.6)	(85.5)
16. Maximum discharge/length	l/s/m	19.78		28.01	23.12	39.03	29.30	35.79
	(cfs/ft)	(2.12)		(3.02)	(2.49)	(3.95)	(2.96)	(3.63)
17. Maximum head on weir (Fig. 6)	m	0.27		0.43	0.33	0.58	0.43	0.52
	(ft)	(0.9)		(1.4)	(1.1)	(1.9)	(1.4)	(1.7)

Note NA — not available

figures as being suitable for the design discharge with the hydraulic head available and the hydraulic constraint of the inlet sewer.

The foregoing design is based on a ratio of chamber diameter to depth of 4:1.

The ratio of D_2 to H_1 can be modified by use of the geometry modification curves in Figure 12.

Assume it is desirable to reduce the depth to its minimum value.
Determine the final ratio of D_2/D_1 as shown in Item 11 of Table 1.
Then, with the use of Figure 12, proceed as shown in Item 12.
Enter D_2/D_1 in Figure 12,
extend line vertically to standard design line, to working point.
Move down parallel to modification curves to horizontal line where ratio of H_1/D_1 is 1.0.
Then proceed down vertically to obtain revised ratio of D_2/D_1.
The resultant depth (H_1) is equal to the inlet dimension (D_1)
and
the chamber diameter (D_2) is larger than the diameter selected in Item 7 of the standard design.

Obviously other ratios of H_1/D_1 could be selected to obtain other size chambers for comparison purposes.

The maximum discharge, Item 13, without a peripheral side weir is twice the design flow. The diameter of the weir, D_4, is Item 14, with the corresponding length of weir, Item 15. The maximum discharge per unit length over the weir, Item 16, is computed, and from Figure 6 the maximum head over the weir, Item 17, is determined.

A D_2/D_1 ratio of 6 was developed (1) as the "standard design." Design flexibility has been extended to cover the range of 4.5 to 12 for D_2/D_1. However, in the absence of major constraints, the "standard design" is considered preferable.

Table 2 lists the areas and volumes for the structures shown in Table 1 for 2.832 cu m/sec (100 cfs) and 4.673 cu m/sec (165 cfs). For the standard design it is obvious that the largest inlet size results in the minimum area and volume. The areas and volumes of the modified design with minimum depth are compared with the areas and volumes of the smallest chamber in standard design. For the two sizes shown the design with minimum depth compared to the smallest standard design show an increase in area of 41 percent and a decrease in volume of 11 to 12 percent. This table indicates that for any given situation the designer has several choices and must weigh the advantages of each before reaching a final decision.

Table 2
Comparison of Variation in Area and Volume Between Standard Design and Design With Minimum Depth

		Design Discharge ³/s (cfs)			
1. Design discharge		2.832 (100)		4.673 (165)	
2. Inlet D	m	1.5	1.2	1.8	1.5
	(ft)	(5)	(4)	(6)	(5)
Standard design					
3. Area	m^2	74	97	105	166
	(sf)	(800)	(1,040)	(1,130)	(1,810)
4. Volume	m^3	181	280	305	598
	(cf)	(6,400)	(9,900)	(10,700)	(10,400)
5. Area change from smallest	%	0	+30	0	+60
6. Volume change from smallest	%	0	+55	0	+63
Modified design minimum H_1					
7. Area	m^2	105	154	147	262
	(sf)	(1,130)	(1,660)	(1,590)	(2,830)
8. Volume	m^3	158	185	265	393
	(cf)	(5,650)	(6,640)	(9,540)	(14,150)
9. Area change from smallest standard	%	+41	+101	+41	+150
10. Volume change from smallest standard	%	−12	+4	−11	+32

Note Area and volume are based on dimensions given in Table 1

Hydraulic Head Requirements

There must be sufficient hydraulic head available to allow dry-weather flows to pass through the facility and remain in the channel. The total head required for operation is shown in Figure 29.

Determination of the maximum elevation in the collector sewer that can be utilized for in-system storage and the differential between the maximum elevation in the collector and the elevation of the interceptor sewer is the total available head.

The head required will vary directly with flow and the outlet losses in the foul sewer.

If sufficient head is not available to operate the foul sewer discharge by gravity, an economic evaluation would be necessary to determine the value of either pumping the foul sewer outflow continuously, pumping the foul flow during storm conditions or bypassing the swirl concentrator during dry-weather conditions.

Site Requirements

The location of the swirl regulator/separator is dependent upon the elevation of the combined sewer and the location of the interceptor sewer. In some instances it may be feasible to construct the facility underground in the public right-of-way.

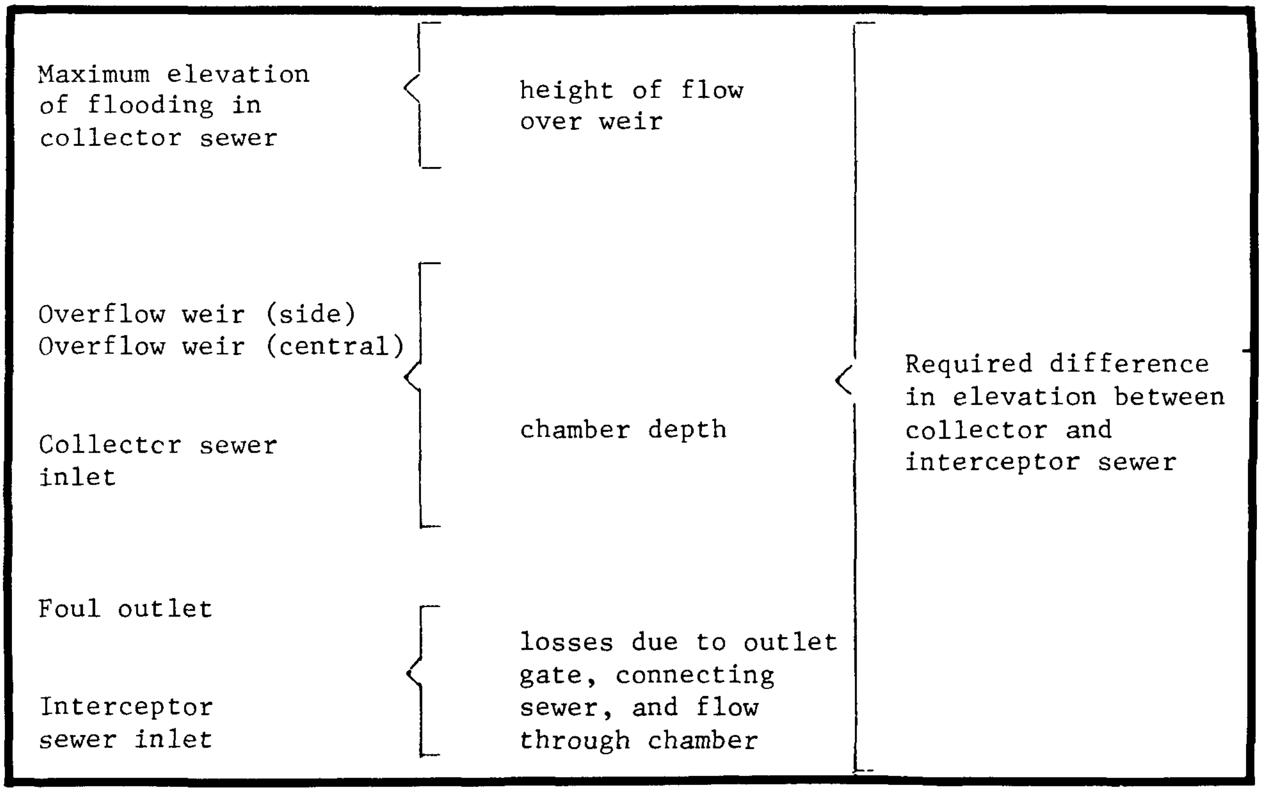


Figure 29 Head Requirements with Free Flowing Interceptor

The site should minimize construction of transition sewers from the collector and the clear overflow discharge to receiving waters. The foul discharge, due to its relatively small diameter, is not usually a critical cost consideration.

If the facility is to be housed in a building, only normal side yard restrictions of the adjacent property will be required. If the unit is to be left open, there should be about 7.5 m (25 ft) clearance. This distance, if the site is fenced, should eliminate most problems of vandals throwing items into the unit. Aesthetics dictate that proper screening of an open facility be provided.

Construction Considerations

The primary element is the circular chamber which normally would be constructed of reinforced concrete. However, it is not necessary to make the interior wall surface a perfect circle and the use of 61 cm (2 ft) wide prefabricated steel forms is considered permissible. The chamber could also be constructed with Gunite or steel. The interior features could be constructed of steel, plastic or fiberglass. However, the flow deflector should be constructed of steel due to abrasion from coarse solids.

It is suggested that the floor have a slope of one to fifty from the wall toward the center. Steeper slopes may reduce separation efficiency.

The layout of the gutter is extremely critical for elimination of deposits on the floor. The foul outlet should be located at the 320 degree position. The floor should have a circular depression around the outlet sewer with a diameter of about three times the diameter of the outlet sewer and a depth of the gutter. A semi-circular shape for the gutter is considered preferable for moving solids in low flow periods. The gutter should have sufficient capacity for the peak dry-weather flow.

The size of the outlet sewer will be governed to a large extent by the required size of the flow control device on the outlet pipe.

The inlet to the chamber must be aligned so as to introduce the storm flow or combined sewer overflow tangentially to the outer periphery of the chamber. An important element is the "flow deflector," a wall extending from the entrance of the inlet sewer to the zero degree position of the chamber. The top of this wall is the same level as the bottom of the weir skirt and is not connected thereto. Flow entering the chamber is directed toward the outside of the chamber by this deflector. Stormwater rotating in the chamber passes over the deflector wall and tends to cause the entering solids to be directed downward in the chamber.

It is important that the inlet sewer enter the chamber with its invert at the same elevation as the chamber bottom. A minimum grade into the unit is desirable, as long as self-cleansing low flow velocities are maintained. This criteria results in more rapid settling of solids to the bottom. If it is possible to surcharge the inlet sewer, then the chamber can be raised the amount of the surcharge and the drop in the inlet transition decreased accordingly.

It is suggested that the "clear water" downshaft and the weir be constructed of steel. Concern must be given to the support of the weir in order that it may be kept level. Temperature changes may tend to distort the elevation of the weir and the shape of the scum ring, if the weir and ring are supported from the top. The latter is not considered to be of major importance.

The downshaft supports a horizontal circular plate. The outer edge of the plate has a vertical plate welded to it which forms a weir above and a skirt below the plate. So-called "spoilers" are vertical radial plates located on the circular plate to prevent vortex action in the downshaft. At least four to eight evenly spaced spoilers should be used extending from the edge of the downshaft to the scum ring. To prevent floatables from flowing over the weir, a scum plate is set away from the weir with the lower edge of the scum plate 15 cm (6 in.) below the weir crest. This scum plate can be supported by the spoilers or by separate brackets.

Studies have indicated that there is less collection of debris on broad-crested weirs than on sharp-crested weirs. Therefore it is suggested the weir be semi-circular in shape .

The floatable deflector consists of a steel plate extending from the outer wall of the chamber to the scum ring and having the same height as the scum ring. From the scum ring two plates form a passage 30 cm (1 ft) wide through the weir. From the weir two plates resting on the horizontal weir plate form a passage to a location near the center. At this location a cylinder is provided through the horizontal weir plate. Vortex action at this point carries the floatables to the underside of the circular plate. The floatable deflector should be constructed as shown in Figure 30a.

The vortex cylinder through the circular horizontal weir plate should be located directly above the foul sewer outlet, Figure 30b.

Details of the gutter layout are shown in Figure 30c. The radii should be adjusted as required to provide a smooth transition in the gutter.

Structure Features

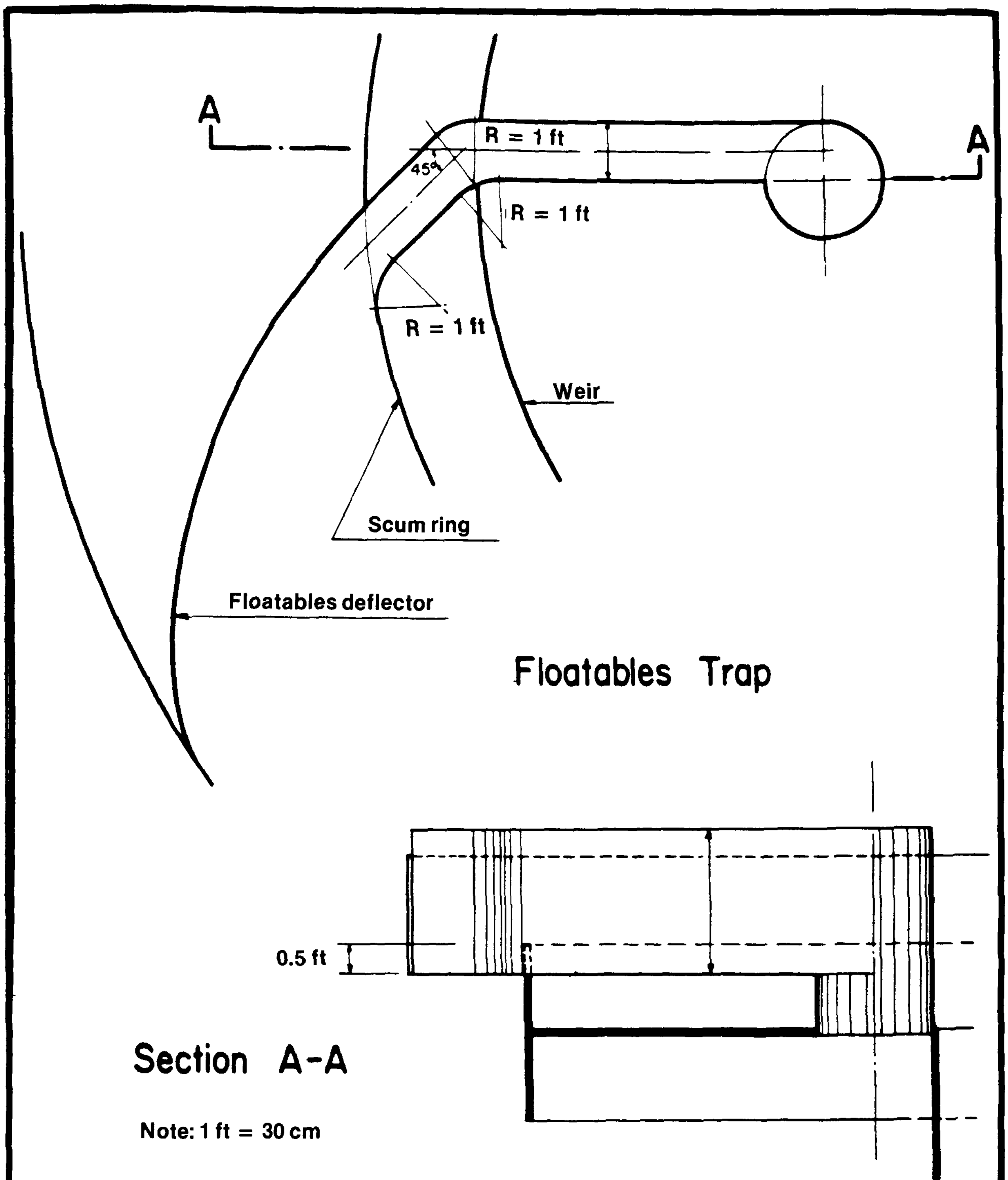
Plans and sections through a typical chamber are shown on Figure 31.

The provision of a roof for the chamber is not necessary for functional reasons but is considered desirable for safety and aesthetic considerations. Several openings are required in the roof. A manhole 60 to 75 cm (2 to 2.5 ft) diameter should be placed directly over the vortex cylinder for the floatables. This will permit rodding of the cylinder in case of clogging. Since the cylinder is located directly over the foul sewer outlet this manhole will also permit rodding of the outlet pipe. A large sidewalk door should be provided to permit removal of large floating objects. The size of the door should be related to some extent by the size of the inlet sewer and the possible size of floating objects. Although a poured roof is shown, a precast unit could also be used.

Three types of entrance stairs are shown in Figure 6.1.3 of the Combined Sewer Overflow Regulator Manual of Practice (15). The preferred access is the use of a 38 degree stairway with 20 cm (7.75 in.) risers and 25 cm (10 in.) treads surmounted with a superstructure with exterior dimensions of 4 m by 1.5 m by 2.4 m (13 x 5 x 8 ft) high. Minimum openings of 60 cm (2 ft) square should also be provided in the sluice gate manhole and the overflow manhole.

An inspection walk should be provided around the periphery of the chamber with a minimum width of 61 cm (2 ft). The walk should be located so that the weir and scum plate can be cleared of debris if required. A pipe handrail 76 cm (42 in.) high should be provided on the walk and stairs.

After each storm the chamber should be inspected. It may be necessary following storms to flush down the bottom of the chamber to prevent subsequent nuisance odors. Floatables collected under the horizontal plate may have a tendency to remain attached to the plate. Floatables may be subjected to heads of up to 1.4 m (5 ft) and this may cause them to adhere to the horizontal plate. Therefore, it may be desirable to remove the materials by flushing after each storm. In cities with many regulators, several days may elapse after a storm before each regulator can be inspected. Hence, automatic cleansing of the chamber bottom and horizontal plate is desirable.



DETAILS OF SPECIAL STRUCTURES — FLOATABLE TRAPS $D_2/D_1 = 6$

Figure 30a Details of Special Structures

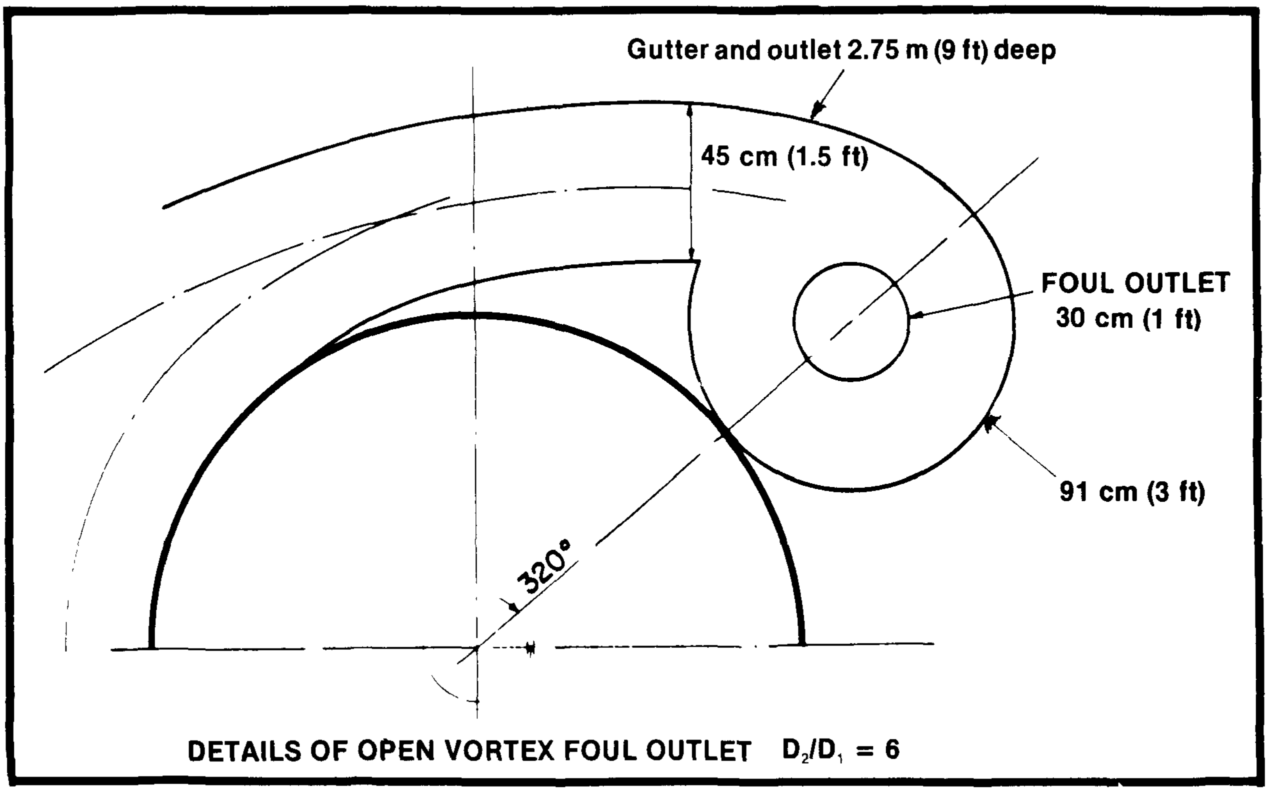


Figure 30b Details of Special Structures

If water used for this purpose comes from a potable supply there should be no physical connection between the supply and the flushing system. A more feasible source of flushing water may be either the nearby receiving waters to which the chamber discharges or the stormflow that passes through the chamber. The use of receiving water requires the construction of a sump and pumps. The use of stormflow requires the construction of a reservoir adjacent to the chamber to store the stormflow during the storm so that it can be used after the storm is ended.

One suggested method of using stormflow for flushing the chamber is shown in Figure 31a. This comprises a 1.2 m (4 ft) square chamber, 2.7 m (9 ft) deep, adjacent to the sluice gate chamber. The capacity is about 3,800 l (1,000 gal). Stormflow enters the manhole through a 30 cm (1 ft)-square opening in the chamber wall set with top of opening level with the circular weir crest. The opening is covered with 1.2 cm (0.5 in.) mesh to prevent solids from entering. The velocity parallel to the chamber wall should keep the screen from clogging. A shear gate is installed in the common wall between the two chambers so that the stormflow chamber can be emptied into the sluice gate chamber after each storm. A vertical wet-pit non-clog pump is used to pump the stormflow into the flushing lines.

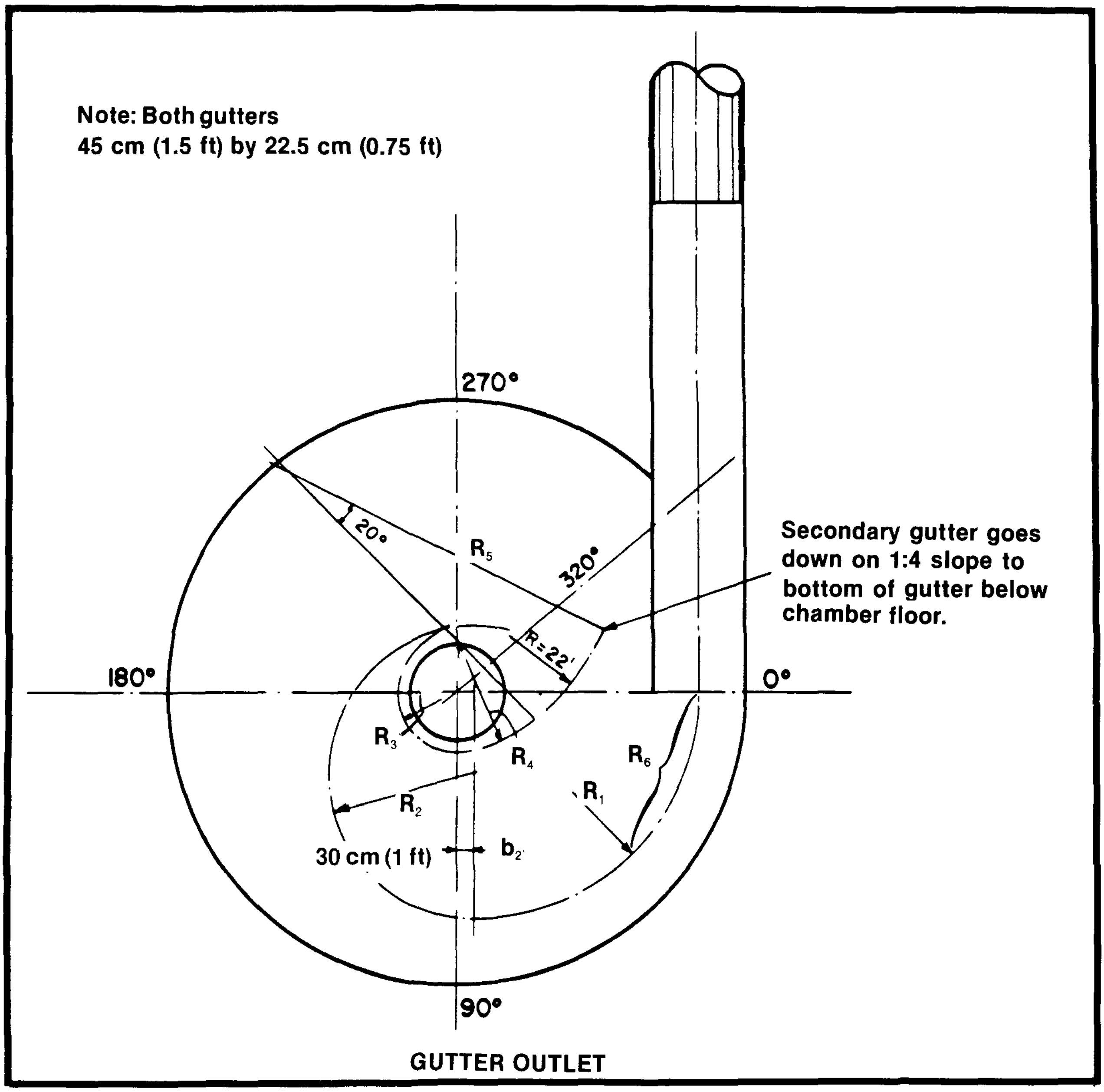


Figure 30c Details of Special Structures

A 10 cm (4 in.)-diameter pipe is installed on the underside of the horizontal plate adjacent to the skirt. This pipe has eight 1.3 cm (0.75 in)-nozzles aimed upward at the bottom of the plate. When the water level in the chamber has fallen to some point below the plate, the pump will operate for 5 minutes, discharging 300 l/min (80 gpm) at 40 psi.

For flushing the bottom of the chamber another 10 cm (4 in.)-diameter pipe is attached to the chamber wall above maximum flow level with sixteen 13 cm (0.75 in.)-nozzles pointed straight downward. When the water level in the chamber has fallen to below the chamber bottom the pump will again operate for about 5 minutes.

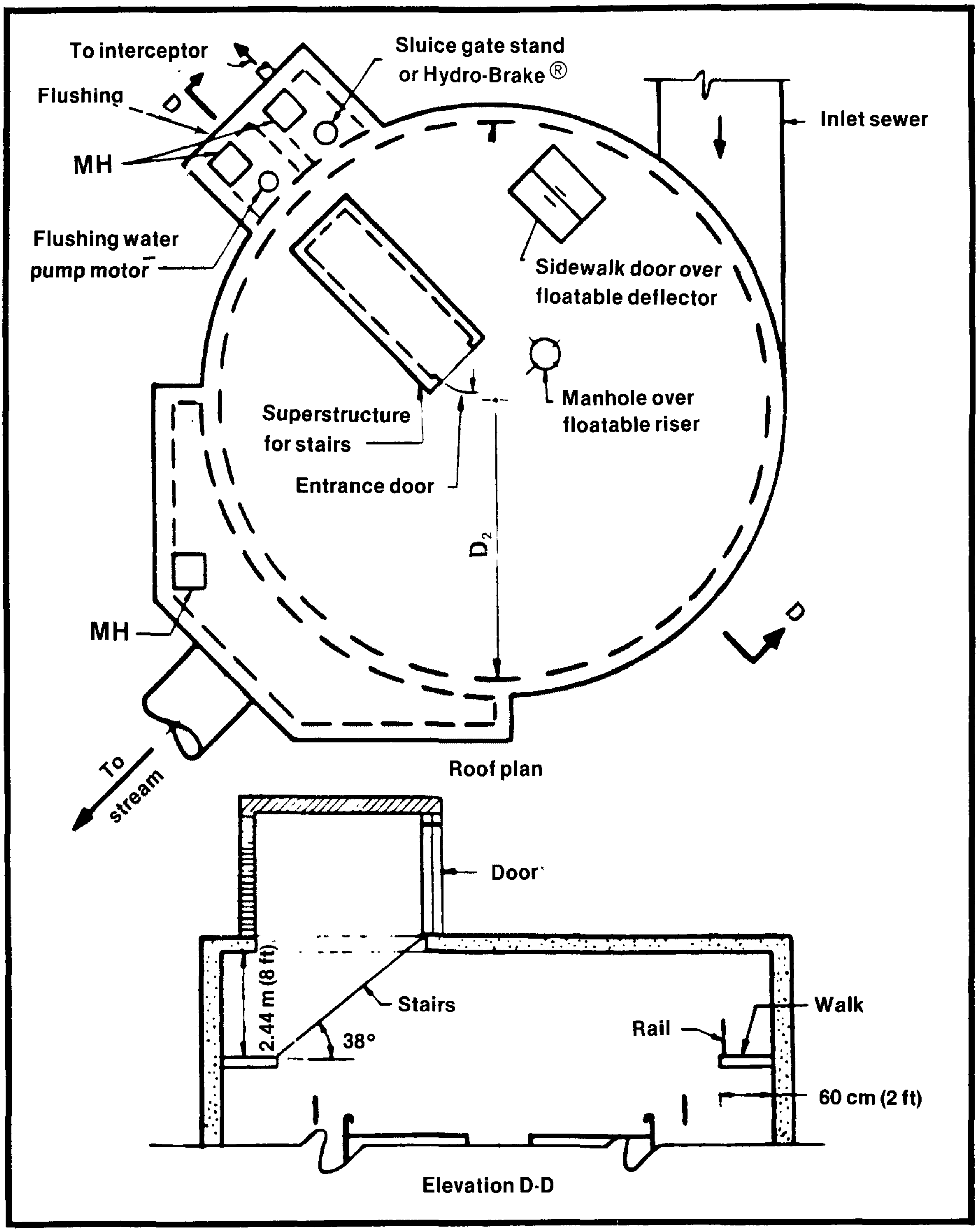


Figure 31a Plan and Elevation

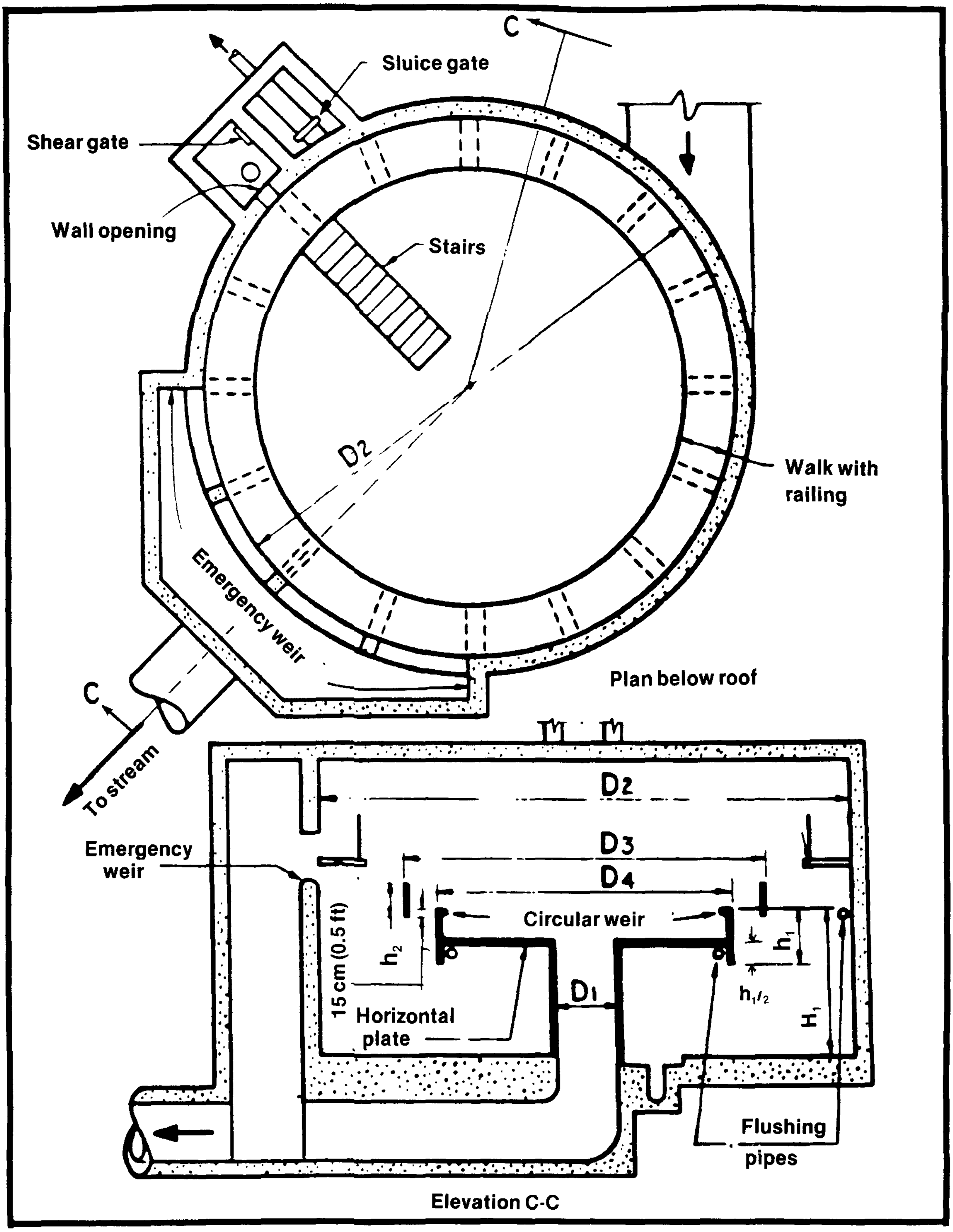


Figure 31b Plan and Elevation

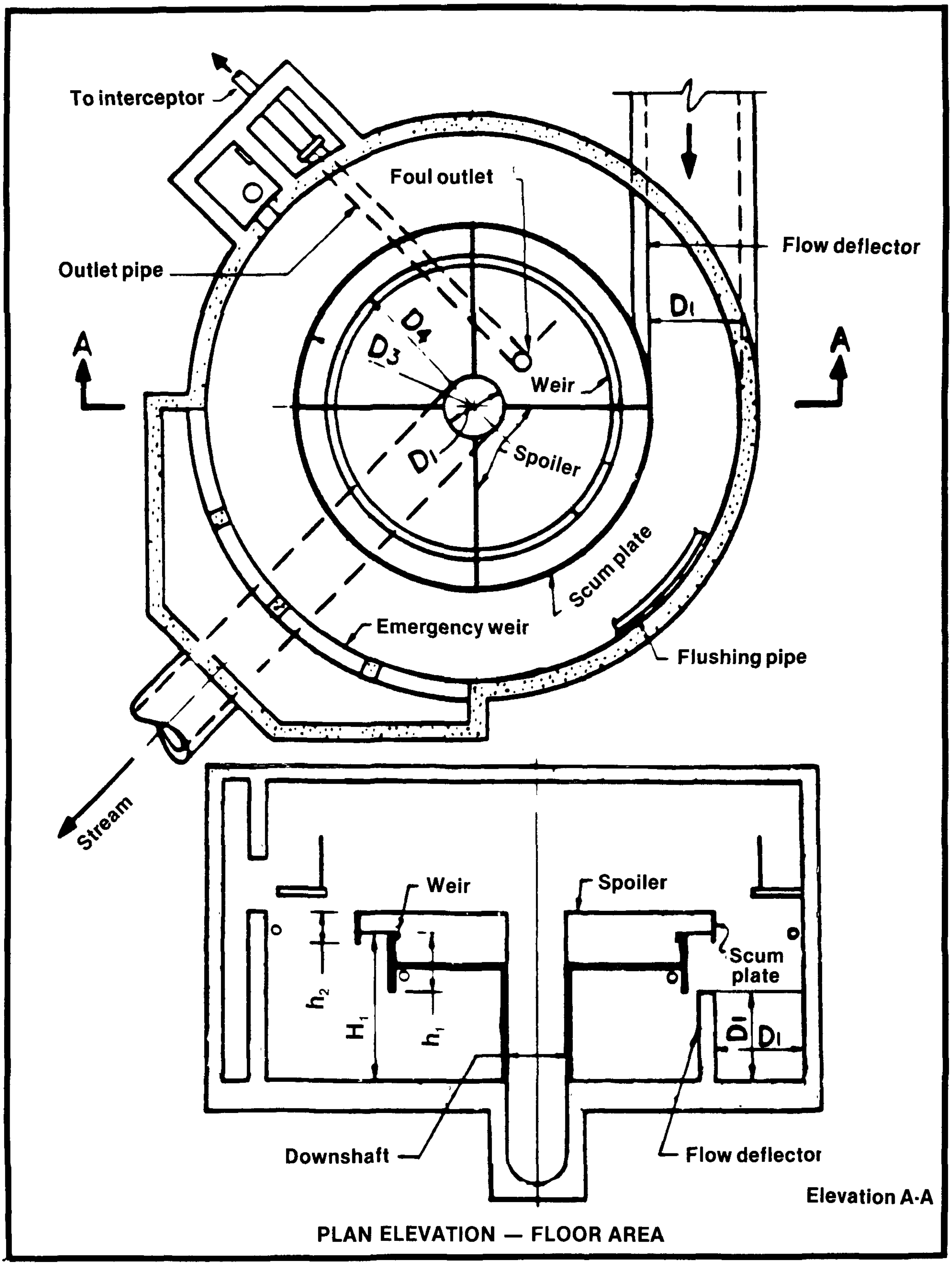


Figure 31c Plan and Elevation

The use of an epoxy covering of internal surfaces is being tested. Preliminary results indicate that with this coating, the unit is essentially self-cleansing.

Hydraulic Compilation

Most combined sewer overflow regulators are designed for use in connection with existing combined sewers and either existing or proposed intercepting sewers. The vertical distance between the hydraulic grade lines in the combined sewer and interceptor must be great enough to permit installation of the regulator. It may be necessary to run through the hydraulic computations at any specific location in order to determine if the swirl concentrator can be used. Table 3 indicates the nature of the computations required to illustrate the factors that should be considered.

In the following computation the "foul sewer" is the outlet pipe from the chamber to the sluice gate manhole and the "branch interceptor" is the sewer from the sluice gate manhole to the interceptor.

As stated previously, some type of control should be provided on the foul sewer where it leaves the chamber. In the following computations the control is assumed to be a manually operated sluice gate. This type of control will result in the greatest variation in flow to the interceptor between dry-and wet-weather periods. One way to decrease the amount of the variation is to design the branch interceptor to flow full under peak dry-weather conditions. Increasing the length of the branch interceptor will also help to decrease the variation. Under these conditions when wet-weather flows occur, the flows will surcharge the sewer and the hydraulic grade line will rise and limit the discharge capacity.

If the variation in flow is too great, then a tipping gate or motor- or cylinder-operated gate should be used instead of the manually operated gate.

The hydraulic gradient and energy lines for peak dry-weather flow should be computed starting at the interceptor and proceeding upstream through the sluice gate manhole to the chamber. The quantity diverted to the interceptor during storm periods is determined in a similar manner by trial and error method assuming various discharges.

In the initial computation, the hydraulic computations should start at the water surface in the interceptor at peak dry-weather flow. In subsequent trials it may be necessary to raise the branch interceptor at its junction with the main interceptor which will result in flow at critical depth at the end of the branch interceptor. In this case it may be necessary to compute the backwater curve for the flow in the branch interceptor to determine the depth of flow at the upstream end. Figures 32 and 33 present the results of the design computations for flow in the foul outlet.

Table 3
Design Example, Swirl Combined Sewer Overflow Regulator/Separator

Design Example
Sample Computations

L	=	Length
A	=	Cross-sectional area
D	=	Diameter
V	=	Velocity
d	=	Depth of flow
Q	=	Discharge
b	=	Width of opening
g	=	Acceleration of gravity
C	=	Coefficient
W.S.	=	Water Surface
H.G.L.	=	Hydraulic grade line
E.L.	=	Energy line
n	=	0.013 (Manning)
S	=	Slope
d_1	=	Depth of swirl concentrator
Interceptor		
D	=	3 ft; invert el. = 10 ft; W.S. = 12.4 ft
Combined Sewer		
D	=	6 ft; invert el. = 19.14 ft; S = 0.0005

Peak dry weather flow = 3 cfs
Design flow = 165 cfs
Maximum flow = 300 cfs

Note Conversion factors — U S customary to metric
1 ft = 0 305 meters — 1 cuft/sec = 28 32 l/sec

			Invert	H.G.L.	E.L.
Interceptor					
Assume			10.00	12.40	
Branch Interceptor					
L	=	100 ft, Q = 3 cfs			
D	=	1.0 ft, S = 0.007			
V (full)	=	3.8 fps			
d/D	=	0.8			
V (0.8 full) =		(1.14) (3.8) = 4.3			
$V^2/2g$ =		0.28 ft			
Set downstream end so flow line is same as interceptor				12.40	
Invert 12.40 — 0.8				11.60	
Exit loss	=	0.28; 12.4 + 0.28			12.68
Upstream end					
Rise	=	(100) (0.007) = 0.70			
		11.60 + 0.70	12.30		
		12.40 + 0.70		13.10	
		12.68 + 0.70			13.38

Table 3 (continued)

	Invert	H.G.L.	E.L.
Sluice Gate Manhole	12.30		
Entrance loss (0.5) $V^2/2g$ = 0.14			
13.38 + 0.14			13.52
Assume loss of velocity head in manhole		13.52	
Sluice gate			
Use 12 inch by 12 inch gate			
Assume opening 0.67 ft high			
V = 3/0.67 = 4.5 fps			
$V^2/2g$ = 0.31 ft			
Exit loss = 0.31			13.83
13.52 + 0.31			
Contraction loss at gate			
$0.3V^2/2g$ = 0.09			
13.83 + 0.09			13.92
Set gate invert at manhole invert	12.30		
Use 1.0 ft square conduit			
Top conduit 13.30			
V = 3/1 = 3 fps			
$V^2/2g$ = 0.14 ft			
13.92 − 0.14		13.78	
Outlet Pipe			
D = 1.0 L = 20 A = 0.785	12.30		
Start pipe 1 ft upstream of gate			
V = 3/0.785 = 3.8 fps			
$V^2/2g$ = 0.22 ft			
Enlargement loss = (0.25) (0.22)			
= 0.06			
E.L = 13.92 + 0.06			13.98
H.G.L. = 13.98 − 0.28		13.70	
L = 20 ft S = 0.007			
Rise = (20) (0.007) = 0.14			
Upper end 12.30 + 0.14	12.44		
13.70 + 0.14		13.84	
13.98 + 0.14			14.12
Use 90° C.I. bend			
Length invert to bell 1.85 ft			
Top of bell 12.44 + 1.85 = 14.29			
Bend loss $0.25V^2/2g$ = 0.06			14.18
E.L. = 14.12 + 0.06			
H.G.L. = E.L.		14.18	
H.G.L. is below top of bell at 14.29			
Chamber Bottom			
Gutter invert	14.29		
Make gutter 0.75 ft deep			
Chamber invert at center			
14.29 + 0.75	15.04		
Use transverse slope of 1/4 in. per ft			
Rise = (15) (1/4) = 3 3/4 in.			
= 0.31 ft			

Table 3 (continued)

	Invert	H.G.L.	E.L.
Chamber invert at wall			
15.04 + 0.31	15.35		
Gutter			
Try one-half 18-in. pipe			
Length from end of ramp to foul outlet = 64 ft (from Fig. 7)			
Total fall = (12) (1/4) = 3 in. = 0.25 ft			
S = 0.25/64 = 0.004			
Q = 6.5 cfs (full pipe)			
V = 3.7 fps (full pipe)			
One-half pipe			
Q = (0.5) (6.5) = 3.2 cfs > 3.0			
V = (1.0) (3.7) = 3.7 fps OK			
Chamber			
For design flow of 165 cfs	Weir		
d_1 = 9.0 (Table 1)			
Weir crest 15.35 + 9.00	24.35		
Weir diameter = 20 ft			
Weir length = 62.8 ft			
Weir discharge per ft 165/62.8 = 2.6			
Weir head = 1.2 (Fig. 6)			
H.G.L. for 165 cfs 24.35 + 1.2		25.55	
Set emergency weir 28 ft long at elevation 25.55			
Determine W.S. for maximum flow of 300 cfs			
By trial and error			

	H	Q
Circular weir	2.0	248
Emergency weir	0.8	45
Foul outlet		3 ±
		296

	Invert	H.G.L.	E.L.
Water surface 24.35 + 2.0		26.35	
This is at 180° position			
Assume same at 0° position			
At 0° position area between deflector and wall equals			
(6) (9 + 2.0) = 66 sq ft			
V = 300/66 = 4.6 fps			
$V^2/2g$ = 0.33 ft	24.35	26.35	26.68
At 0° position			

Table 3 (continued)

	Invert	H.G.L.	E.L.
Inlet Pipe			
D = 6 ft A = 28.3 sq ft			
V = 10.6			
$V^2/2g$ = 1.74	Invert		
Enlargement Loss			
(0.25) (1.74 − 0.33) = 0.35			
Required E.L.			27.03
Required H.G.L.		25.29	
Required invert so pipe is not surcharged 25.29 − 6.0	19.29		
Required vertical distance from W.S. in interceptor to invert of inlet sewer 19.29 − 12.40 = 6.89 ft			
Determine flow to interceptor when maximum flow is 300 cfs and W.S. in chamber is 26.35			
Assume 8.6 cfs			
Interceptor			
Assume W.S. as before		12.40	
Branch Interceptor			
D = 1.0; V = 11.0; $V^2/2g$ = 1.88			
S = 0.06			
Exit loss 1.88			14.28
Rise = (100) (0.06) = 6.00			20.28
Manhole			
Entrance loss 0.5 $V^2/2g$ = 0.94			21.22
Sluice gate (from before)			
A = 0.67; V = 12.9; $V^2/2g$ = 2.58			
Exit loss 2.58			23.80
Contraction loss (0.3) (2.53) = 0.77			24.57
Outlet Pipe			
L = 20 S = 0.06			
Rise = (20) (0.06) = 1.20			25.77
Bend loss (0.25) (1.88) = 0.47			26.24
H.G.L. for 8.6 cfs		26.24	
Actual H.G.L.		26.35	

Therefore discharge through foul outlet will be about 8.6 cfs when maximum flow of 300 cfs occurs.

COSTS

Typical dimensions for three sizes of the swirl regulator/separator are given in Table 4. Basic dimensions are taken from Table 1. Cost estimates are based on these dimensions and the construction details shown in Figure 31.

Table 4
Swirl Regulator/Separator Dimensions

Design discharge	m^3/s	1.42	2.83	4.67
	(cfs)	(50)	(100)	(165)
Diameter of chamber D_2	m	7.0	9.8	11.6
	(ft)	(23)	(32)	(38)
Diameter of inlet D_1	m	1.2	1.5	1.8
	(ft)	(4)	(5)	(6)
Height-invert to roof floor to weir H_1	m	1.8	2.4	2.9
	(ft)	(5.8)	(8.0)	(9.5)
Head on weir Fig. 6	m	0.27	0.40	0.55
for 150% design flow	(ft)	(0.9)	(1.3)	(1.8)
Clearance to walk	m	0.30	0.30	0.30
	(ft)	(1.0)	(1.0)	(1.0)
Headroom	m	2.44	2.44	2.44
	(ft)	(8.0)	(8.0)	(8.0)
Total height from invert of	m	4.8	5.5	6.2
chamber to underside of roof	(ft)	(15.7)	(18.3)	(20.3)
$D_3 = 5/9\ D_2$	m	3.9	5.4	6.4
	(ft)	(12.8)	(17.8)	(21.1)
Weir length	m	12.2	17.0	20.1
	(ft)	(40)	(53)	(66)
Discharge per unit length	m^3/s	0.18	0.25	0.34
at 150% design flow	(cfs)	(1.8)	(2.8)	(3.7)

Note: obtain other dimensions from Fig. 7

Assumptions for Estimating

The height of the structure from the floor of the chamber to the underside of the roof is based on the following criteria:

Assumption 1: The clearance between the top of the walk and the water surface is 0.31 m (1.0 ft) when the discharge is 150 percent of design discharge and the foul outlet is not functioning. The head on the weir is determined from Figure 6.

Assumption 2: The headroom above the walk is 2.44 m (8.0 ft).

If the underside of the roof is assumed to be at ground level and the inlet sewer approaching the chamber is assumed to have 2.44 m (8 ft) cover, then the crown of the sewer would be at the level of the walk or 0.3 m (1 ft) above high water level. Hence, the inlet sewer would not be subject to surcharge. The estimates are based on these assumptions. If a "modified" design is used with smaller H_1 and a larger D_2, the resultant costs may be more or less than those shown.

Additional assumptions for estimation purposes are as follows:

A. The walls are 0.30 m (1.0 ft) thick.

B. The roof is of poured concrete, about 0.25 m (0.83 ft) thick, with two beams 0.92 m (3.0 ft) by 0.46 m (1.5 ft).

C. The bottom concrete slab is 0.61 m (2.0 ft) thick.

D. The concrete walk is 1.22 m (4.0 ft) wide and supported on concrete beams.

E. The superstructure is 3.96 m (13.0 ft) long by 1.52 m (5.0 ft) wide by 2.44 m (8.0 ft) high.

The miscellaneous cost is taken as 25 percent and is intended to include stairs, handrails, scum baffle, circular weir, flushing water system and pipes, a manual sluice gate and manhole, electrical work, ventilating work, and doors.

Contingent and engineering costs are taken as 25 percent of the foregoing.

Construction Costs

The costs of the three selected sizes of swirl regulators, designed for 90 percent removal of grit, are shown in Table 5. These costs are for a unit requiring sheet piling, poured concrete walls, and a roof slab.

Alternate Approach

A 1976 report for USEPA (23) approached the cost from the standpoint of only the facility without site improvements and interconnecting piping. Rather, only unit material costs were developed. Based upon a H_1/D_2 ratio of 0.25, chamber diameters of 4.4 m (12 ft) to 17.6 m (48 ft) were developed. Costs were then related to the surface area. Table 6 presents the results in tabular form.

The cost of the swirl regulator/separator includes the basic chamber, which does not include roof, pumping stations, flow measurement or basin dewatering facilities. These items, if applicable, must be added to derive a total estimated project cost. The chamber dewatering facility is normally incorporated in the sludge (or concentrate) pumping station or concentrate discharge facilities. Costs for raw wastewater pumping stations, sludge (or concentrate) pumping stations, and flow measurement facilities are presented later in this section.

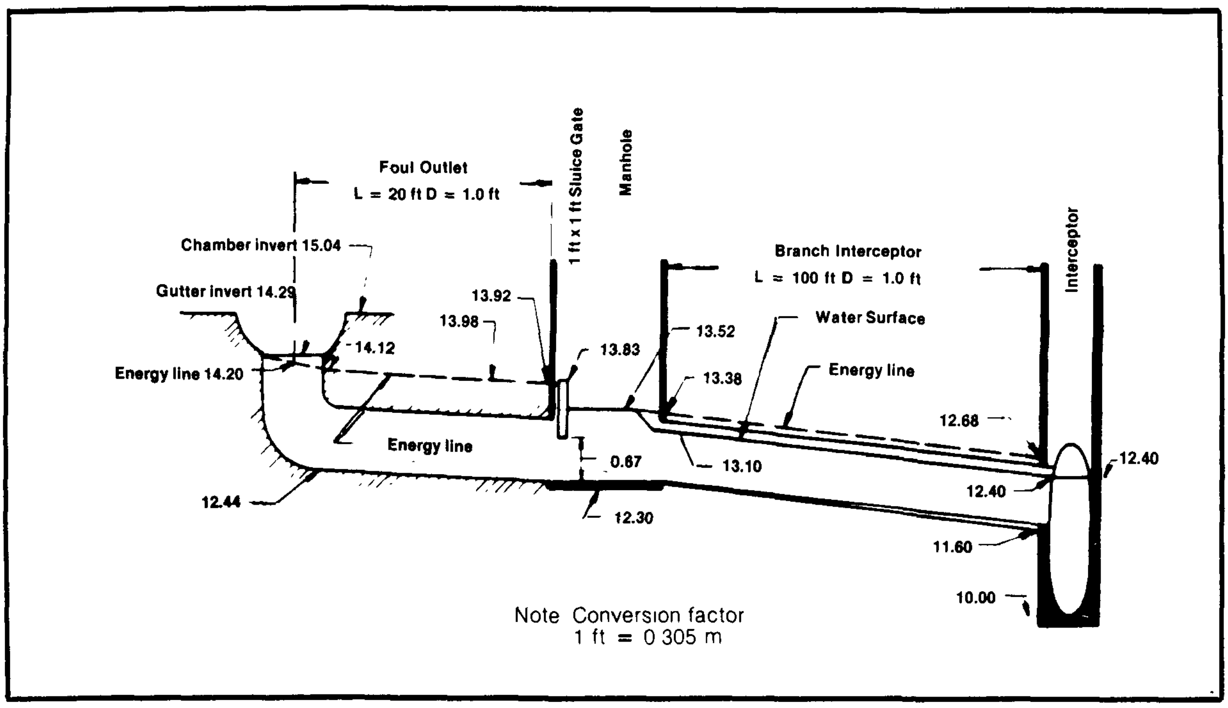


Figure 32 Hydraulic Profile 85 l/sec (3 cfs)

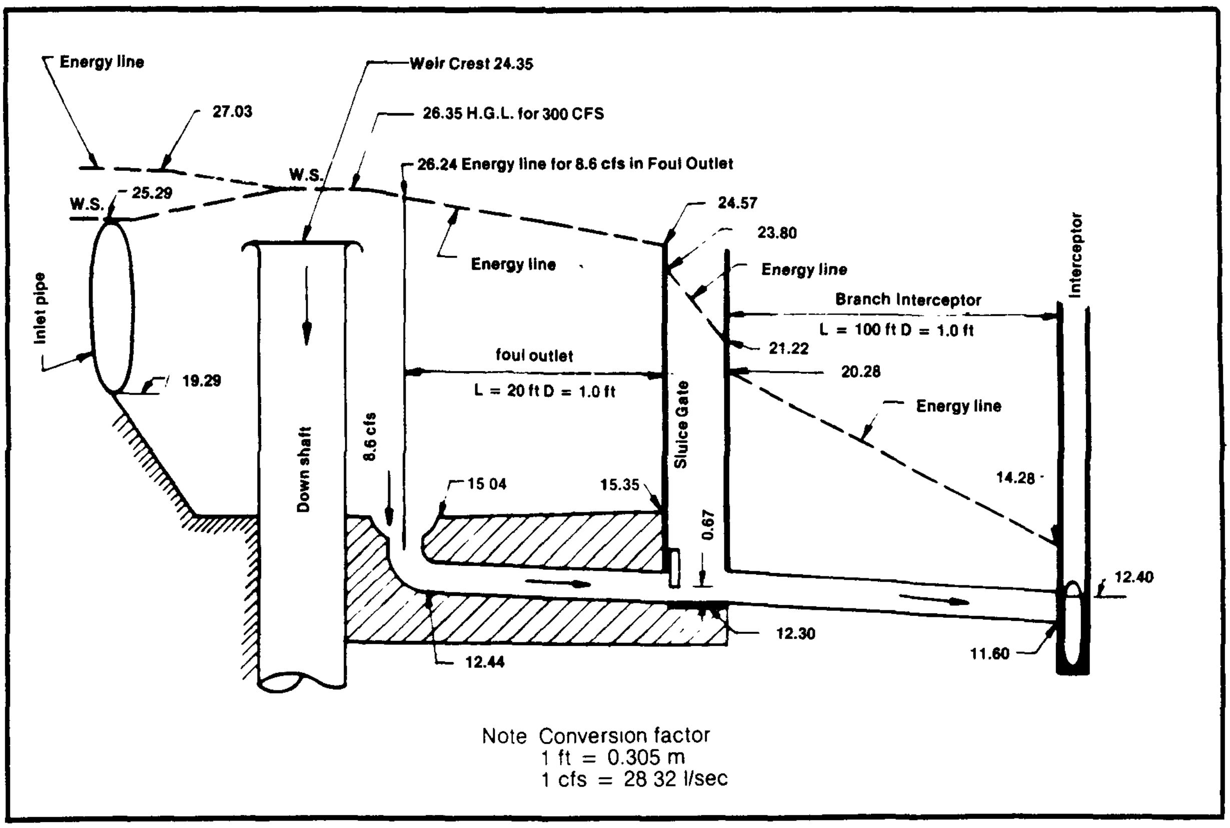


Figure 33 Hydraulic Profile 245 l/sec (8.6 cfs)

Table 5
Construction Cost — Swirl Regulator/Separator

Capacity 1.42 m^3/s (50 cfs)

Item	Quantity	Amount
Sheet piling	200 m^2 (2,160 sf)	$ 25,920
Excavation	460 m^3 (600 cy)	10,800
Reinforced concrete	98 m^3 (128 cy)	48,000
Concrete block walls	27 m^2 (290 sf)	3,480
Roof	6 m^2 (65 sf)	910
Outlet pipes		1,940
Downshaft and plate		3,000
Subtotal		94,030
Miscellaneous costs	25%	23,500
Bypass sewer		15,000
Subtotal		132,530
Contingent and engineering costs 35%		46,400
Total		$178,930

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Capacity 2.83 m^3/s (100cfs)

Item	Quantity	Amount
Sheet piling	290 m^2 (3,120 sf)	$ 37,440
Excavation	900 m^3 (1,180 cy)	16,200
Reinforced concrete	156 m^3 (204 cy)	76,500
Concrete block walls	27 m^2 (290 sf)	3,480
Roof	6 m^2 (65 sf)	910
Outlet pipes		4,500
Downshaft and plate		4,500
Subtotal		135,430
Miscellaneous costs	25%	33,860
Bypass sewer		29,400
Subtotal		198,690
Contingent and engineering costs 35%		69,540
Total		$268,230

Table 5 (continued)

Capacity 4.67 m^3/s (165 cfs)

Item	Quantity	Amount
Street piling	375 m^2 (4,030 sf)	$ 48,360
Excavation	1,360 m^3 (1,780 cy)	32,040
Reinforced concrete	216 m^3 (282 cy)	105,750
Concrete block walls	27 m^2 (290 sf)	3,480
Roof	6 m^2 (65 sf)	910
Outlet pipes		6,000
Downshaft and plate		7,500
Subtotal		204,040
Miscellaneous costs	25%	51,010
Bypass sewer		42,000
Subtotal		297,050
Contingent and engineering costs 35%		103,950
Total		$401,000

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Table 6
Cost Estimate Unit Prices, Swirl Combined Sewer Overflow Regulator/Separator D_2/D_1 = 0.25
90% efficiency

Cost Component	Surface Area (square feet)						
Sq. meters	10.5	23.6	42.0	65.7	94.6	128.7	168.2
Sq. feet	113	254	452	707	1018	1385	1810
Manufactured equipment	10,500	16,500	22,500	28,500	34,500	40,500	48,000
Concrete	600	1,340	2,220	3,500	5,120	8,470	12,970
Steel	2,480	4,885	8,305	13,250	20,580	30,425	42,530
Labor	4,075	9,765	17,770	29,650	45,355	68,560	95,100
Metal pipe and valves	—	—	—	—	—	—	—
Concrete pipe	—	—	—	—	—	—	—
Housing	—	—	—	—	—	—	—
Electrical and instrumentation	3,000	3,200	3,400	3,600	3,800	4,000	4,200
Miscellaneous items	3,100	5,350	8,130	11,780	16,400	22,790	30,420
Contingency	3,560	6,160	9,350	13,540	18,860	26,210	34,980
Total estimated cost	27,315	47,200	71,675	103,820	144,615	200,955	268,200

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OPERATION AND MAINTENANCE

The operation and maintenance requirements for swirl regulators/separators has been assumed to be constant for all sizes of units.

Cleaning of the unit may be done with automatic washdown facilities. Many utilities perform routine visits to unattended stations to assure the facility is operable when needed and for a check to assure the facility has not been vandalized. It is assumed that the unit will be visited every other week (26 times per year) and the inspection visit will require two hours, including travel time. Such costs are common to all types of facilities and are not unique to the swirl.

With automatic washdown facilities only one or two special site visits to remove large objects can be anticipated. If manual hosing is required after each storm event, four to eight hours of labor per event for cleanup should suffice, particularly if the interior of the facility is covered with epoxy to minimize adherence of solids to the walls. Absence of square corners and the presence of the gutters greatly facilitates washdown.

If a bar screen is used, frequency of cleaning will be dependent upon the normal amount of large floatables in the combined sewer. As four to five centimeter (1.75 to 2 in.) openings are sufficient, it is likely that the screen will need to be cleaned only after each storm event. Such cleaning and handling of the solids should take less than one hour per storm event.

PROTOTYPE INSTALLATIONS

Results from three demonstration units are available which have tended to validate the laboratory testing. Problems of sampling the combined sewer flow and the clear and foul sewer flow have proved to be very difficult. Merely characterizing the settling velocity of the typical solids in order to determine the theoretical solids removal efficiency as set by the assumed solids concentration used in the laboratory has proved very difficult. Applicable data from three installations will be categorized under construction cost, operating experience, and pollution removal efficiency.

The units which have been constructed include those by Onondaga County (Syracuse), New York; Lancaster, Pennsylvania; and Boston, Massachusetts. The unit in Syracuse is unique in that it was constructed even though there was not sufficient hydraulic head available to allow gravity flow during dry-weather flow without maintaining the swirl unit almost full. A pump was placed on the foul line. Costs and problems associated with the pumping have not been included in this text as they are, in effect, independent of the unit's operation. The Boston facility is a part of a large demonstration project to test the swirl and helical bend regulator/separator. Comparison tests are to be run on combined sewer overflows and stormwater. However, only generalized operating experience is available at this time.

The Lancaster unit is a full-scale unit which has been in operation for a year. Sampling problems to date do not allow definitive conclusions to be drawn as to the actual solids removal efficiency.

A small pilot facility has also been tested at San Francisco, California. The swirl was tested on dry-weather sanitary flow. The unit, as predicted, was not suitable as the size and concentration of solids was not in the operating range of the unit.

Table 7 gives information on the three prototype units.

Table 7
Comparison of Prototype Units

Item	Syracuse	Lancaster	Boston
D_2/D_1	0.25	0.25	0.25
Design efficiency	90%	90%	90%
D_2	3.7 m (12.3 ft)	8.8 m (24 ft)	3.2 m (10.5 ft)
Design flow	23.4 cu m/min (8.9 mgd)	68 cu m/min (26 mgd)	10.2 cu m/min (3.9 mgd)
Dry-weather flow	1.3-2 cu m/min (0.5-0.75 mgd)	4.7 cu m/min (1.8 mgd)	Not applicable

Construction Cost

Available data from Syracuse and Lancaster are presented in Table 8 and 9. The Boston unit was prefabricated from an available storage tank and labor and fabrication costs are not comparable to a regular construction contract.

Table 8
Construction Cost — Syracuse. NY

	Reported Costs	Current Equivalent Cost*	Portion Attributable To Swirl Unit
Site work	$ 18,700	$ 30,600	—
Piping	19,700	32,300	—
Swirl chamber	19,700	32,300	$32,300
Electrical	4,100	6,720	6,720
Miscellaneous	3,500	5,740	2,870
Totals	$ 65,700	$107,660	$41,890

*ENR 3140

Table 9
Construction Cost — Lancaster, PA

Concrete	$ 69,000
Excavation	14,000
Miscellaneous metal	28,000
Roof	5,000
Valves and gates	8,000
Paint	4,000
Ventilation	3,000
Total estimated cost	$131,000
Estimated cost attributable to swirl unit	$ 88,000

ENR 3140

Operating Experience

The experience of all three agencies with the operation of the swirl unit has been very good. Generally, operational problems which have become apparent have been due to design deficiencies. Structural members in the flow field induce motions which impede concentration of solids. Among the other problems which have been found, two demand careful attention.

Floatables - Where large branches or other debris can be found in the flow, the possibility exists of blocking the floatable inlet. This has been reported to have happened at least twice in Syracuse. Lancaster has placed a bar screen ahead of the swirl at a diversion chamber. If the floatables trap is blocked, floatables escape to the receiving waters. Thus, for full protection, the bar screens are desirable, even though they add an element of maintenance of up to one hour per storm event.

Shoaling of Solids - The transition in the inlet structure must be designed with care to prevent shoaling of solids. At Lancaster, the flow path through the inlet diversion-screening chamber is offset. Shoaled material has built up in the backwater. This could be prevented by having the flow channel continue in a straight line.

Lancaster reports that the unit is self-cleansing with the automatic washdown facilities and that no additional cleaning has been necessary in 16 months of operation. Hangup of floatable debris can be almost completely eliminated by the application of an epoxy coating as was done in Boston.

The use of the HydroBrake(R) as discussed in Section XI appears to be satisfactory in Lancaster and Boston as a means of controlling the foul sewer flow.

Treatment Effectiveness

Monitoring for treatment evaluation has been performed at the Syracuse facility. Efficiency has been calculated on the basis of actual performance without comparison to theoretical values based upon the laboratory work.

Suspended Solids - Relatively good SS removal efficiencies were determined over the entire storm flow range at the Syracuse prototype as shown in Table 10. Total mass loading and concentration removal efficiencies ranged from 33 to 82 percent and 18 to 55 percent, respectively, with flowrates from 0.54 cu m/min (0.2 mgd) to 20.5 cu m/min (7.6 mgd). Figures 34 and 35 illustrate the total SS mass removals with respect to time and storm flowrate. The shaded areas between curves indicate a trend of higher removals at storm onset when concentrations are generally higher, and again near the end of the storm when flowrates drop.

Table 10
Suspended Solids Removal, Syracuse, NY

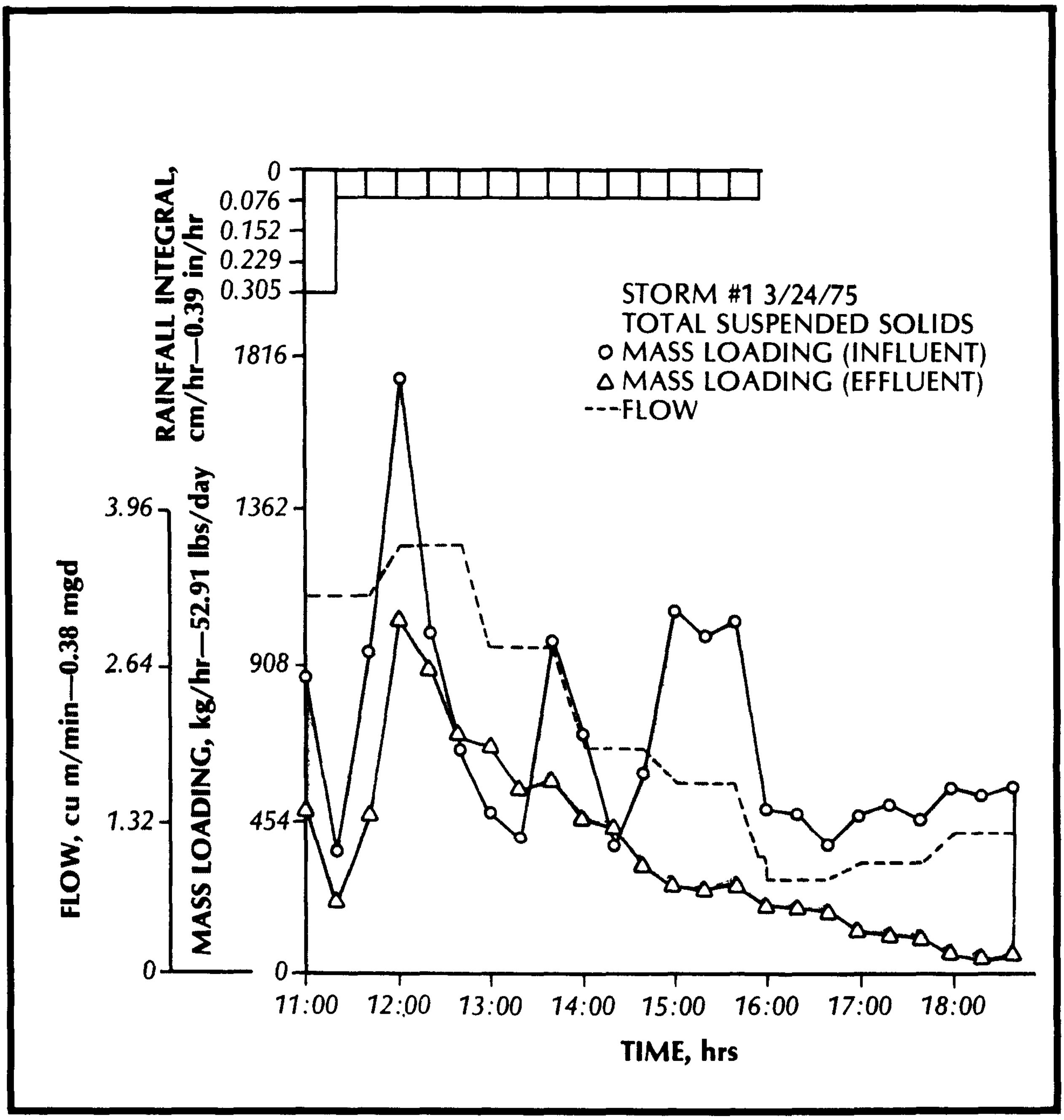
	Swirl Concentrator						Conventional Regulator		
	Average SS per storm, mg/l			Mass Loading kg			Mass Loading kg		
Storm #	Inf.	Eff.	Removal [b] (%)	Inf.	Eff.	Removal [b] (%)	Inf.	Under-flow	Removal[a] (%)
2-1974	535	345	36	374	179	52	374	101	27
3-1974	182	141	23	69	34	51	69	33	48
7-1974	110	90	18	93	61	34	93	20	22
10-1974	230	164	29	256	134	48	256	49	19
14-1974	159	123	23	99	57	42	99	26	26
1-1975	374	167	55	103	24	77	103	66	64
2-1975	342	202	41	463	167	64	463	170	37
61975	342	259	24	112	62	45	112	31	28
12-1975	291	232	20	250	168	33	250	48	19
14-1975	121	81	33	83	48	42	83	14	17
15-1975	115	55	52	117	21	82	117	72	62

a — For the conventional regulator removal calculation it is assumed that the SS concentration of the foul underflow equals the SS concentration of the inflow.

b — Data reflecting negative SS removals at tail end of storms not included.

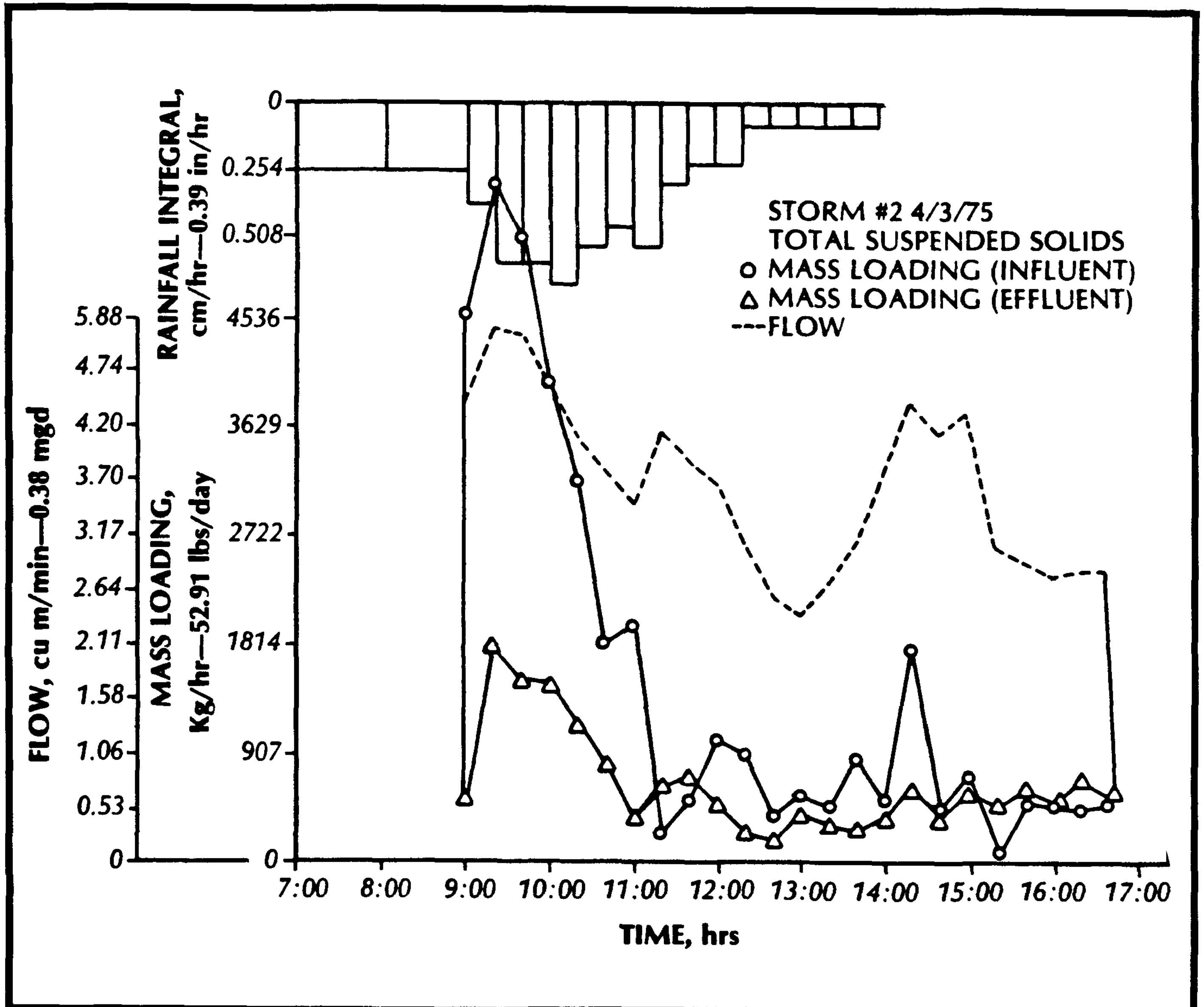
Source EPA Technology Transfer Capsule Report (EPA-625/2-77-012)

Figure 36 further reveals the trend of greater SS mass loading reduction as the SS influent concentrations increase. Suspended solids influent concentrations greater than 250 mg/l generally resulted in removals of better than 50 percent of the total mass loading to the swirl.



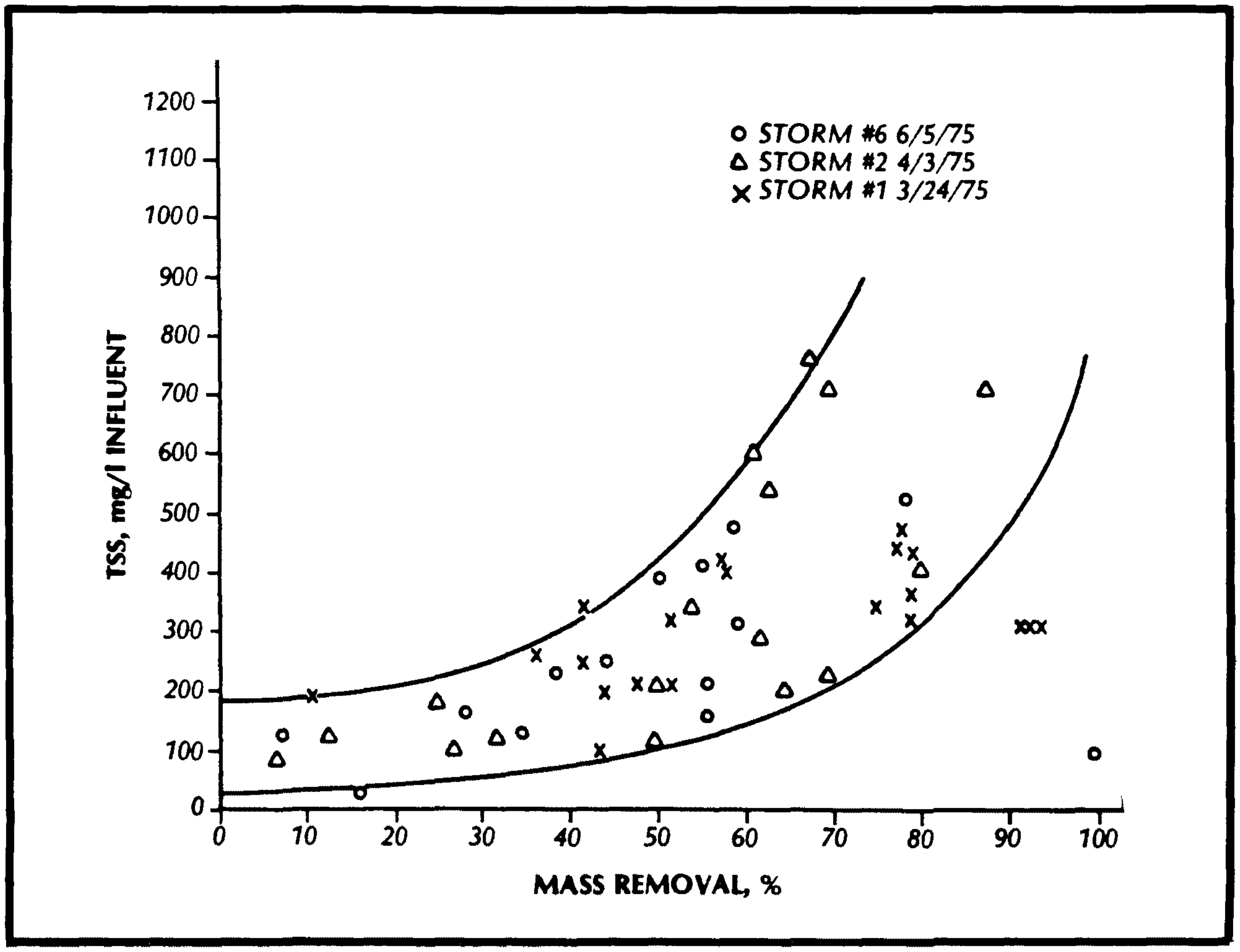
Source EPA Technology Transfer Capsule Report (EPA-625/2-77-012)

Figure 34 Suspended Solids Removal, Syracuse, NY, Storm #1



Source EPA Technology Transfer Capsule Report (EPA-625/2-77-012)

Figure 35 Suspended Solids Removal, Syracuse, NY, Storm #2



Source EPA Technology Transfer Capsule Report (EPA-625/2-77-012)

Figure 36 Suspended Solids Influent Concentration vs Percent Mass Loading Removal, Syracuse, NY

Care must be taken in evaluating swirl solids treatability since under dry-weather flow conditions, all regulators are designed to divert the entire flow volume and associated solids to the intercepting sewer until a predetermined overflow rate is reached. This diversion to the interceptor continues at a maximum throughout the storm. However, the swirl has the added advantage of concentrating solids as well as conventionally diverting flow during overflow events. This concentrating effect is evidenced by removal efficiencies in terms of SS concentrations varying from 18 to 55 percent (Table 10) as previously stated whereas conventional regulators are assumed not to concentrate solids at all (zero percent removal based upon concentration) (Table 10, footnote a).

If the swirl regulator was replaced by a conventional flow regulator, the net mass loading reductions (attributable to the SS conventionally going to the intercepted underflow) would have ranged from 17 to 64 percent (Table 10) as compared to a more effective range of 33 to 82 percent (Table 10) for the swirl. This may be a better way to compare the effectiveness of the swirl to conventional combined sewer overflow regulators since conventional devices will remove the solids associated with the flow diverted for treatment.

For low-flow storms approaching the maximum dry-weather capacity of the interceptor, the advantages of swirl concentration are reduced as would be expected based on the physical principle of mass balance involved. In other words, as the ratios of "inflow to foul outlet underflow" or "weir overflow to foul outlet underflow" decrease, the SS removal advantage from swirl concentrating also decreases. This is because the intercepted hydraulic loading to underflow becomes more significant in the net mass loading calculation of the hypothetical conventional regulator. This phenomenon is exemplified by the SS total (of the swirl) compared to SS net (of the conventional regulator) mass loading removals of Storm No. 1-1975 (Table 10), where the hydraulic loadings to the swirl were low, approaching dry-weather conditions.

Many outfalls are designed to pass 20, 100 and even 1,000 times average dry-weather flow as opposed to the Syracuse facility which, at best, passes only 10 times average dry-weather flow. For these cases, the swirl concentrating effect will exhibit distinct advantages over conventional regulators for SS removal.

BOD_5 Removal

Prototype analyses indicated BOD_5 removals of 50 to 82 percent for mass loading, and 29 to 79 percent in terms of concentration (Table 11). Figures 37 and 38 indicate the trend for BOD_5 total mass loadings removal for the swirl prototype. Figure 39 indicates higher removals at higher BOD_5 influent concentrations.

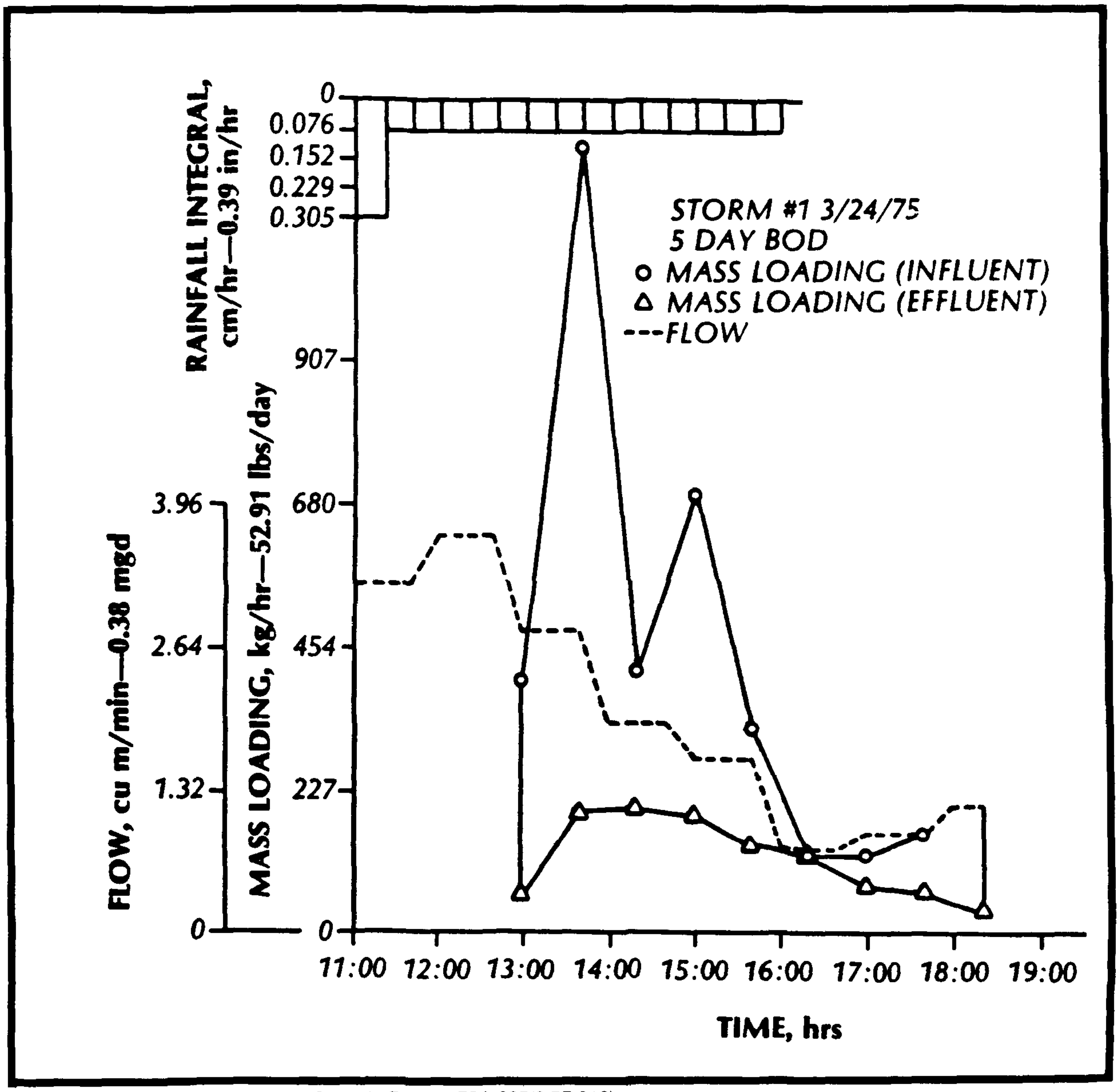
Removal of BOD_5 could not be modeled in the laboratory. Association of the BOD_5 load with the various sizes of settleable solids will dictate removal loads. Such relationships are also important for other pollutants such as phophorus, ammonia, and heavy metals.

Table 11
BOD_5 Removal, Syracuse, NY

	Mass Loading, kg			Average BOD_5 per storm, mg/l		
Storm -	Influent	Effluent	Rem. (%)	Influent	Effluent	Rem. (%)
7-1974	26,545	4,644	82	314	65	79
1-1975	3,565	1,040	71	165	112	32
2-1975	12,329	6,164	50	99	70	29

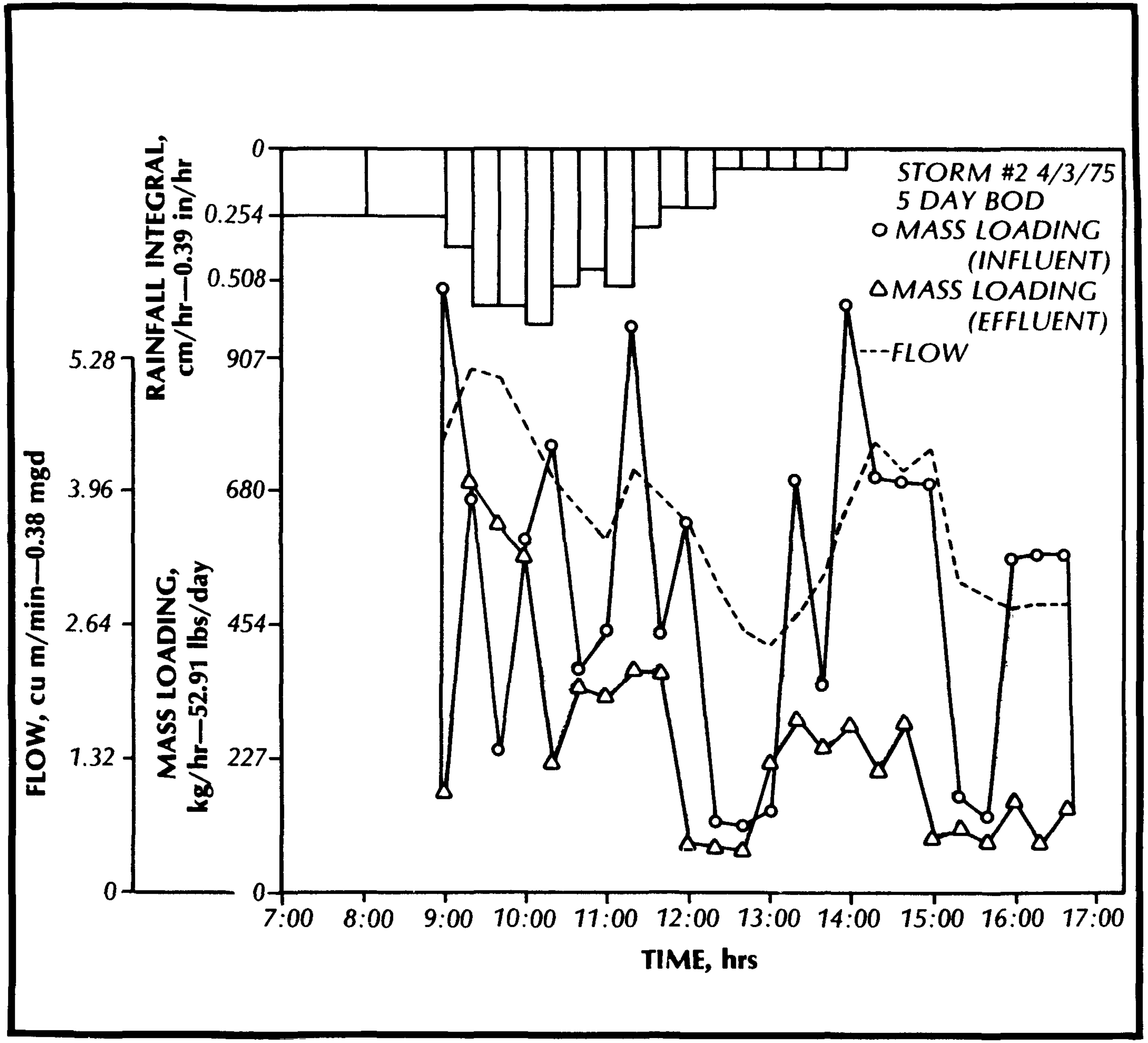
Source EPA Technology Transfer Capsule Report (EPA-625/2-77-012)

The floatables trap has been reported to work well and there has been almost no observable floatables passing over the weir. Adequate testing has not yet been reported for removal of oils and floating debris as sampling is all but impossible in the influent pipe.



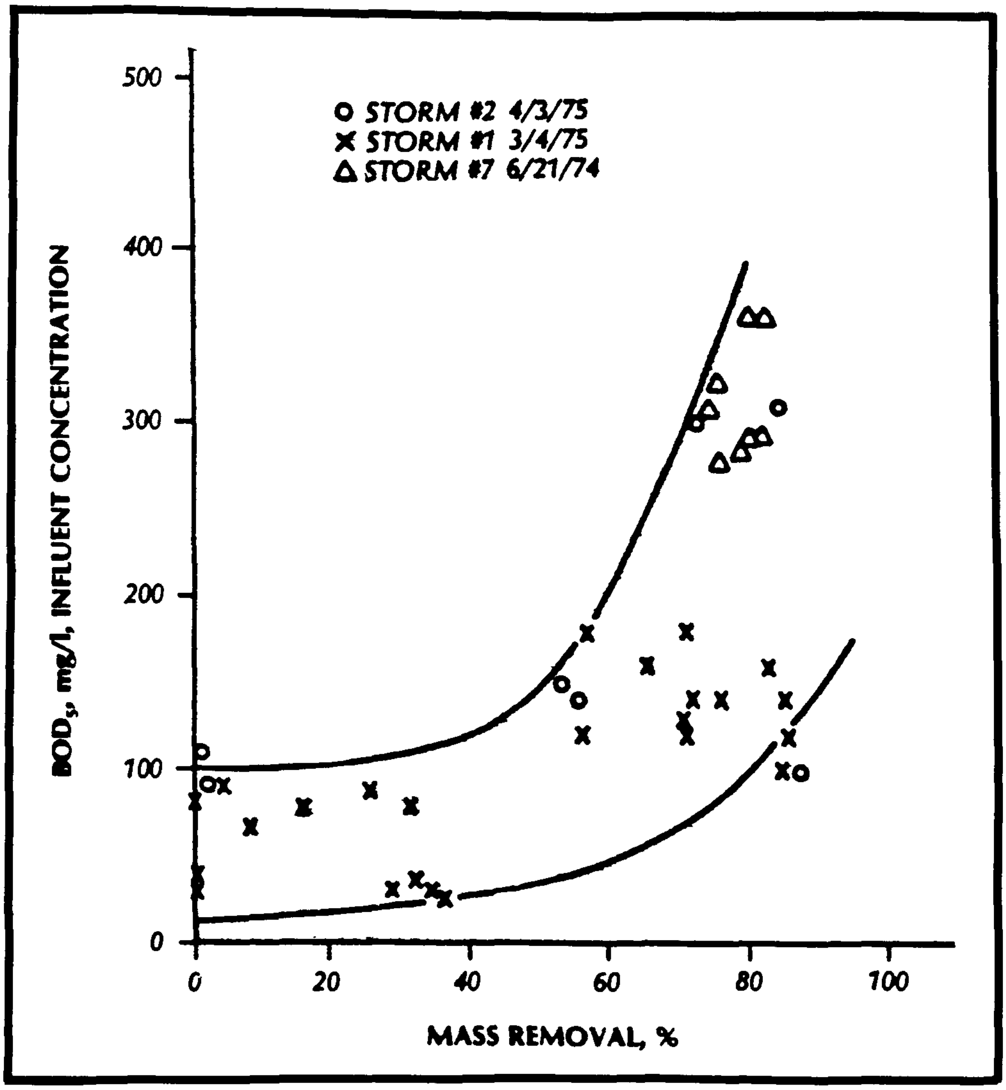
Source EPA Technology Transfer Capsule Report (EPA-625/2-77-012)

Figure 37 BOD$_5$ Removals, Syracuse, NY, Storm # 1



Source EPA Technology Transfer Capsule Report (EPA-625/2-77-012)

Figure 38 BOD$_5$ Removals, Syracuse, NY, Storm#2



Source EPA Technology Transfer Capsule Report (EPA-625/2-77-012)

Figure 39 Swirl Regulator BOD_5 Influent Concentration vs Percent Mass Loading Removal, Syracuse, NY

Section III

HELICAL BEND COMBINED SEWER OVERFLOW REGULATOR/SEPARATOR

DESCRIPTION

The helical bend combined sewer overflow regulator/separator consists of an enlarged section of the sewer which acts as a solids and floatables trap prior to diversion of the overflow to additional treatment or receiving waters. The device requires considerable space to construct. However, right-of-way requirements may be minimal as the bulk of the device may at times be constructed from the overflow point back along the sewer. Operation and maintenance are minimized as there are no mechanical or moving parts within the device. The channel is curved to develop helical secondary motions within the flow. The helical motion effectively captures particles which have a greater settling velocity than the upward velocity of the helical motion. Relatively high velocities are achieved as the chamber empties to treatment at the end of the storm event which will remove deposited solids. Thus, the helical bend separator is unique in that most of the removed solids are released at the end of the storm event.

The helical bend combined sewer overflow separator was developed in Great Britain by T. M. Prus-Chacinski, and a full scale test was made in Nantwich, G.B., following laboratory studies. The design developed for USEPA was based upon English experience as well as hydraulic and mathematical model studies. A demonstration unit has been installed (September, 1979) in Boston, Massachusetts, where it will be tested on both combined sewage and separate stormwater flows.

Figure 40 is an isometric view of the separator. The transition section, which is 15 times the diameter of the inlet combined sewer in length, is covered to allow development of improved flow lines as the width of the chamber is expanded to three times the inlet diameter. Figure 41 shows the transition section in a prototype unit. A straight section of five times the inlet diameter allows the development of a less turbulent flow field and the development of the shape of the bottom cross section. A sixty degree bend with a radius of sixteen inlet diameters uses the outside edge of the curve as an overflow weir. The weir is baffled to trap floatables. At the end of the curved section is a wall with an outlet sized to allow dry-weather flow to flow unimpeded to treatment. The outlet may require a mechanical flow regulating device. The regulator has been designed to allow maximum flows consistent with downstream hydraulic capacity to continue to treatment.

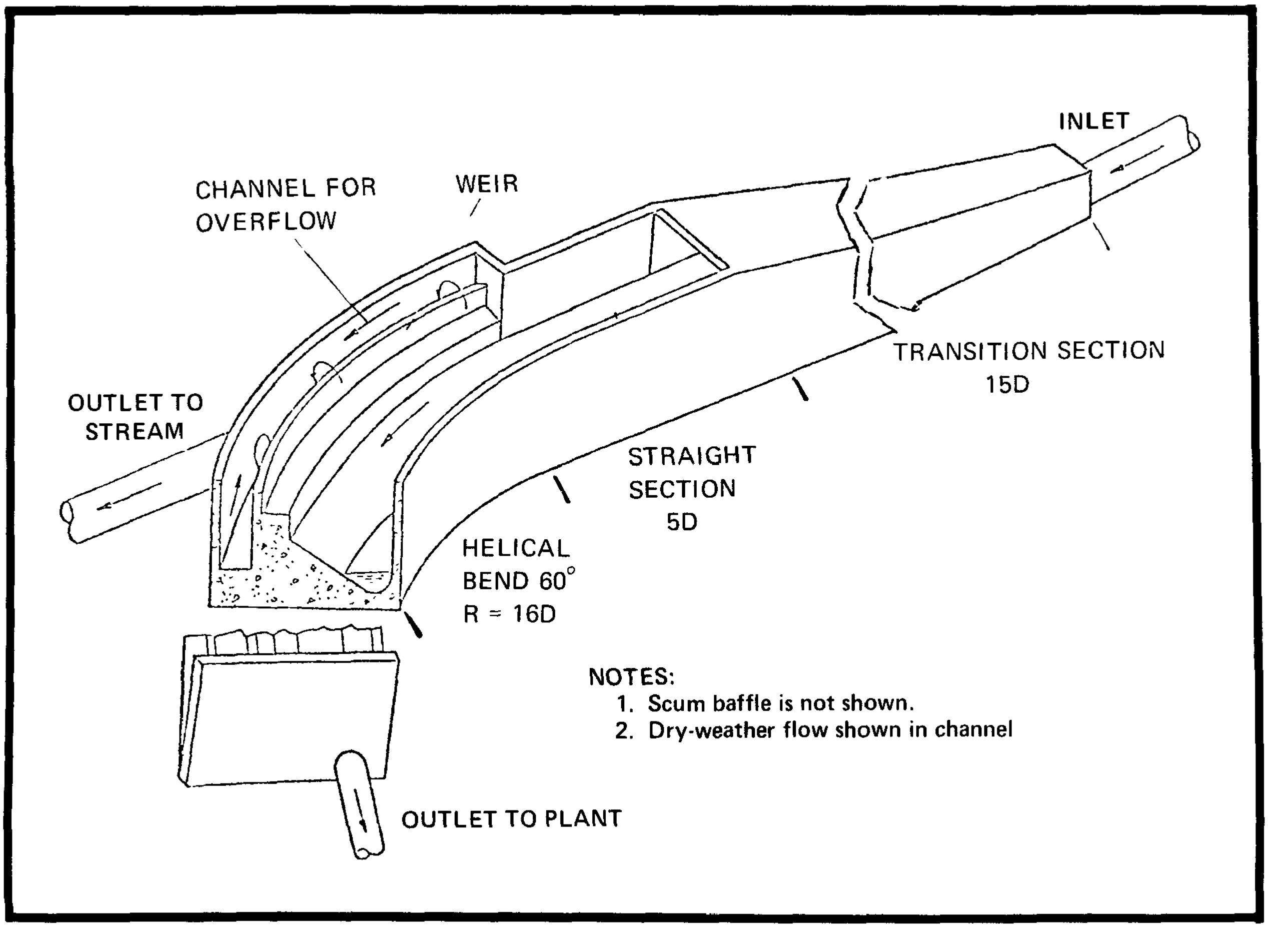


Figure 40 Isometric View of Helical Bend Combined Sewer Overflow Regulator/Separator

Available data indicates that the helical bend separator can be as efficient as the swirl separator/regulator. The helical bend separator compared to the swirl separator should have less head loss, may require less acquisition of additional right-of-way, and allows the solids to be delivered to treatment over a relatively short period of time at the end of the storm event. However, the cost of the device may be up to fifty percent more than an equivalent swirl separator/regulator and almost three times more than a swirl unit designed to remove 80 to 90 percent of the solids.

DESIGN GUIDELINES

The decision as to the design flow should be based upon the same type of considerations as presented in the design flow information of Section I.

Figures 42 to 46 are required for designing the helical separator.

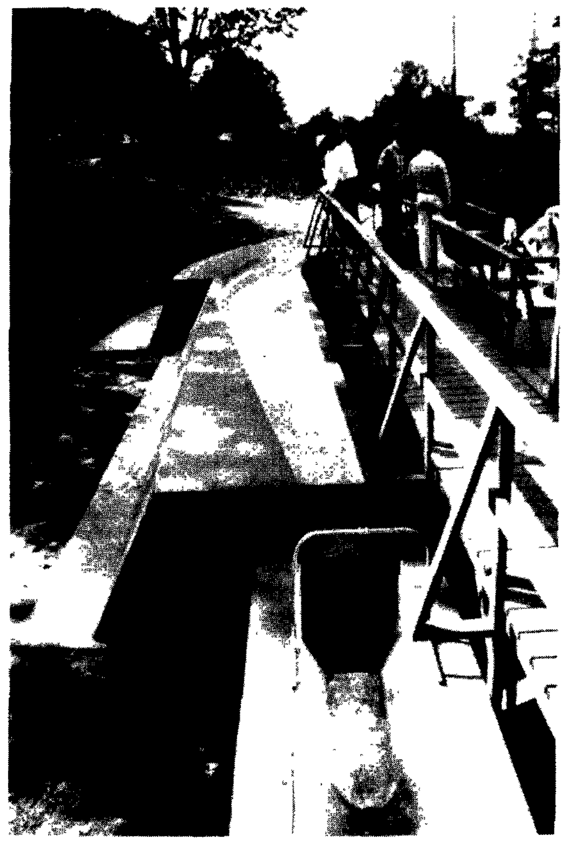


Figure 41 Transition Section, Boston

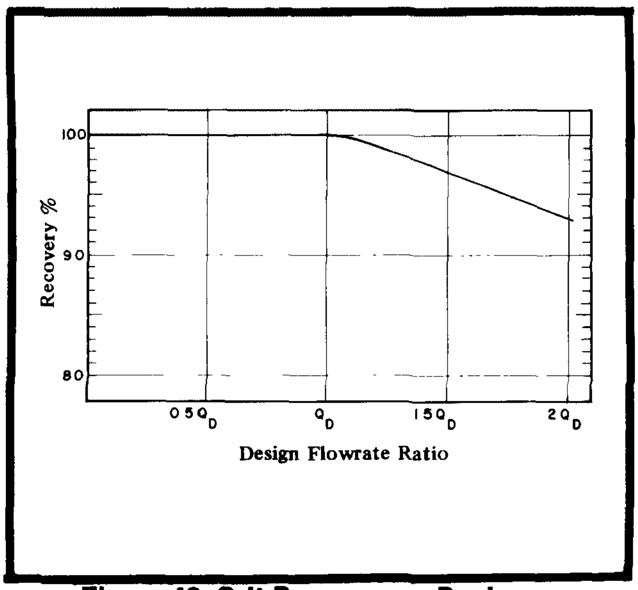
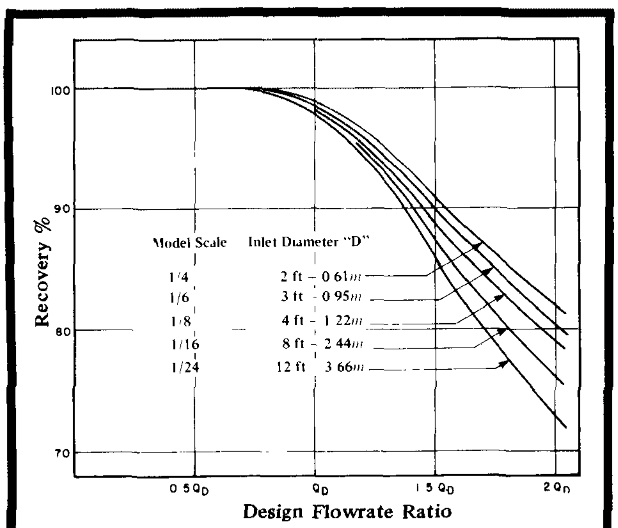


Figure 42 Grit Recovery vs Design Flowrate Ratio



Note: Organic material as defined on Figure 86

Figure 43 Settleable Recovery vs Design Flowrate Ratio

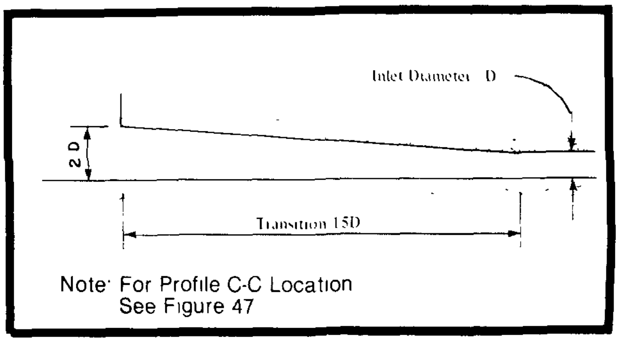


Figure 44 Transition Profile

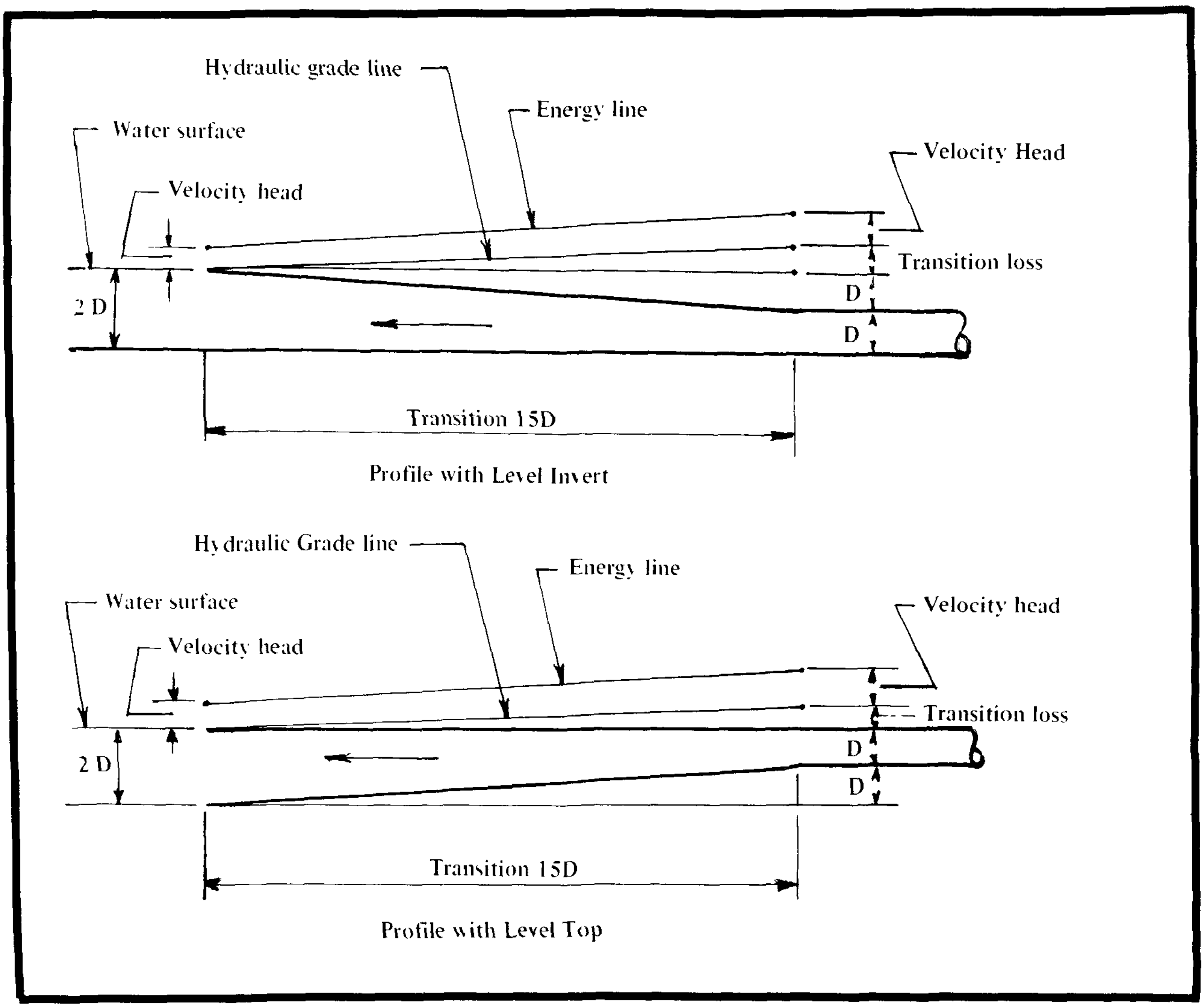


Figure 45 Effect of Transition Slope

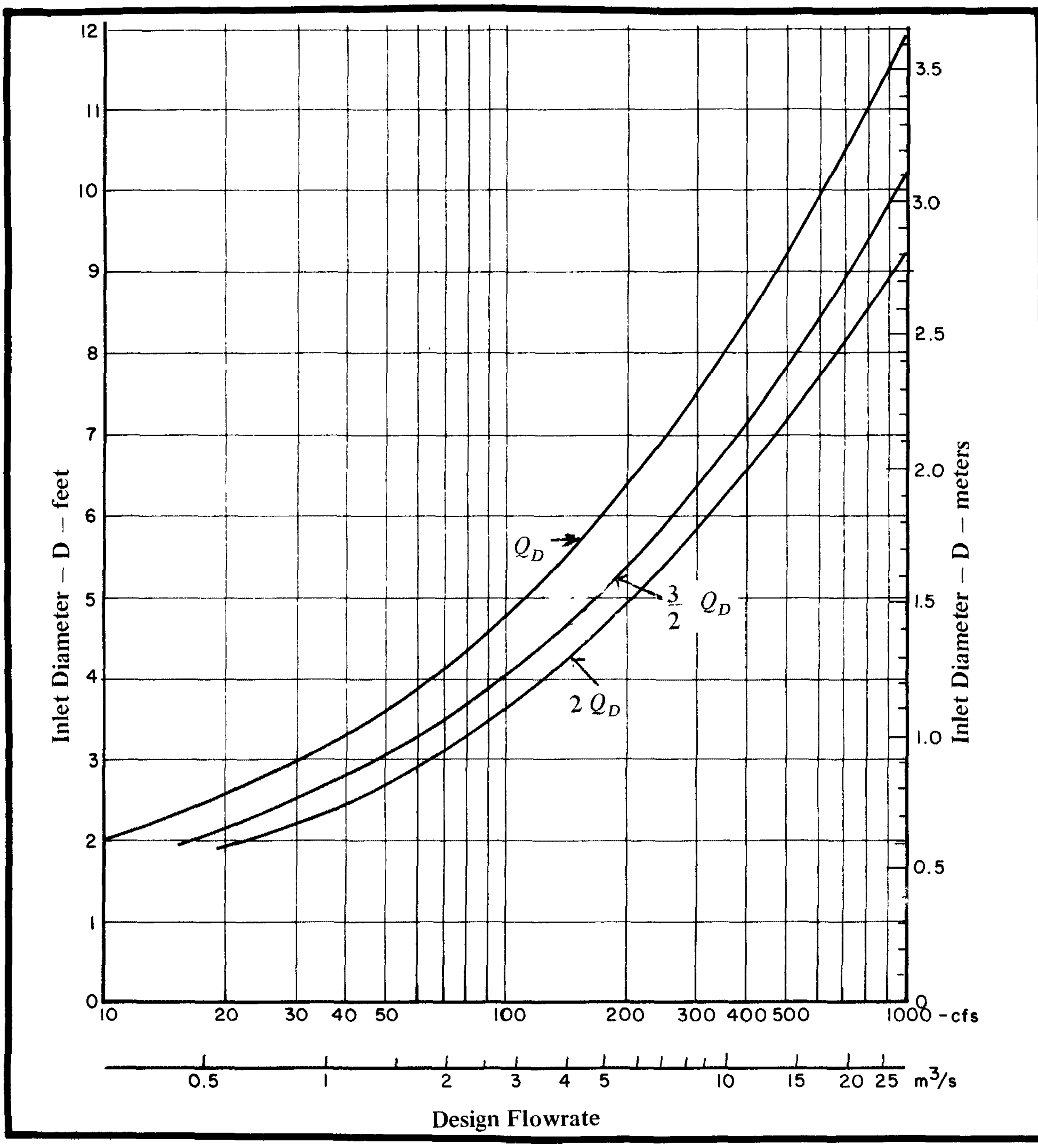


Figure 46 Design Flowrate vs Inlet Diameter

The first step is to determine the design flow rate. As mentioned in Section I, design flows should be less than the system capacity. Figure 42 indicates that the grit recovery will decrease to 97 percent with a flow equal to 1.5 times the design flow rate and to 93 percent with a flow equal to 2 times the design flow rate. Hence, the efficiency of grit removal is not greatly affected by flows up to twice the design flow rate.

Similarly, Figure 43 indicates the decrease in recovery of organic matter with increase in flow. Thus, for 1.5 times design flow rate, the efficiency decreases to about 87 percent; and for 2 times design flow rate, the efficiency decreases to about 75 percent.

From the foregoing, it would appear that a considerable increase in flow above the design flowrate can occur without greatly affecting the operating efficiency of the helical separator.

Transition Slope

As shown in Figure 44, the recommended transition has a length of 15 D and a height of D at the inlet and 2 D at the outlet.

The invert should have some slope. To prevent any surcharge at the inlet, the top of the transition should be kept level, and the invert should be either the slope of the inlet or the slope that will satisfy the hydraulic slope S in the Manning equation, whichever is greater.

The resultant hydraulic conditions either with the invert level or the top level is shown in Figure 45. The transition with the level top has the following advantages: 1) The sewer is not surcharged upstream of the transition except for loss of head in the transition, which may be minor; and 2) the slope will increase the velocity through the helical separator as the storm flow subsides which may aid in flushing deposits out of the helical section. The chief disadvantage of providing too great a slope in the transition is that the outlet pipe to the stream from the helical separator may be lowered so much that the extension of the existing sewer cannot be utilized for this purpose. Therefore, each situation must be evaluated before selecting the slope. Again as a minimum, the transition should have the same slope as the incoming sewer.

Transition Length

The transition length as given in Figure 45 is 15 D. The value of D is selected from Figure 46. Assume the designer selects a D of 1.83 m (6 ft) from Figure 46 as appropriate for the design flow rate. Then the recommended transition length is 27.4 m (90 ft). However, assume the existing sewer has a diameter of 1.52 m (5 ft) rather than 1.83 m (6 ft). The problem is how to effect the connection from the existing sewer to the transition. The most logical way would be to extend the transition to meet the existing sewer while reducing the area at the same rate as occurs in the transition. The area of the transition at the entrance would be 0.785 D squared and at the exit 4.70 D squared. These areas are equivalent to squares with a side of 0.885 D at the entrance and of 2.16 D at the exit. Accordingly, the slope of the side of the transition would be equal to 1.28 D divided by 30 D or 0.0426. This slope has an angle of 2 degrees 26 minutes. Thus, if the diameter of the existing sewer is 0.30 m (1 ft) smaller than the selected D, the transition should be extended by 3.5 m (11.6 ft).

It would also seem logical to reduce the length of the transition by a similar process if the area of the existing sewer is larger than the area of the transition inlet selected from the design charts.

Transition Inlet Size

All dimensions of the helical separator are related to D, the diameter of the transition inlet. After determining the design flow rate, the designer should select the inlet diameter, D, from Figure 46 which shows the simple scaled-up values according to the Froude Law, for the design discharge, Q_D, as well as 1.5 Q_D and 2 Q_D. These have been computed covering the likely range of applicable flood discharges and pipe sizes that will be encountered in any prototype installations.

Seldom will the value of D be that of a standard pipe size. Hence, the designer should select the nearest D corresponding to a standard pipe size. If the indicated D falls between two pipe sizes, the larger D will give a separator with greater efficiency than the smaller D. For instance, if the design flow rate is 2.83 cu m/sec (100 cfs), the indicated D will be 1.45 m (4.75 ft). The designer can select a D of 1.37 m (4.5 ft), equivalent to a design flow rate of 2.40 cu m/sec (85 cfs). If the latter capacity is chosen, the design flow rate will be 18 percent larger than the separator capacity. From Figure 42, the grit removal efficiency will be reduced to 99 percent of the total grit load. From Figure 43 the settleable organic removal will be reduced to about 96 percent.

If, in the example given above, the existing sewer should have a diameter equal to one of the possible D selections, then it would be logical to select the D which matches the existing sewer size. Otherwise, the transition should be extended as discussed previously.

The overall length of the helical separator is approximately 37 D including the transition and straight sections, as shown in Figure 47. If a D of 1.37 m (4.5 ft), is selected, the length will be 50.9 m (167 ft) whereas if a D of 1.52 m (5 ft) is selected, the length will be 56.4 m (185 ft). A third possibility, assuming the existing sewer is 1.37 m (4.5 ft) and the D indicated by the chart is 1.4 m (4.75 ft), would be to base all dimensions on the indicated D and to extend the transition according to the method explained previously. In this case, the transition would be extended an amount equal to one-half the difference in diameters, divided by 0.0426 or 0.91 m (3 ft). The overall length in this case would be 37 times D plus 0.91 m (3 ft), or 54.5 m (179 ft).

Thus, the designer is faced with the choice of three lengths--either 50.9 m (167 ft), 56.4 m (185 ft) or 54.5 m (179 ft). Obviously the largest helical separator will provide the most efficient operation.

The design calculations in this section have been based upon the need to provide regulation of the combined sewer overflow. If treatment is the primary objective, the design should be modified to further enhance solids separation.

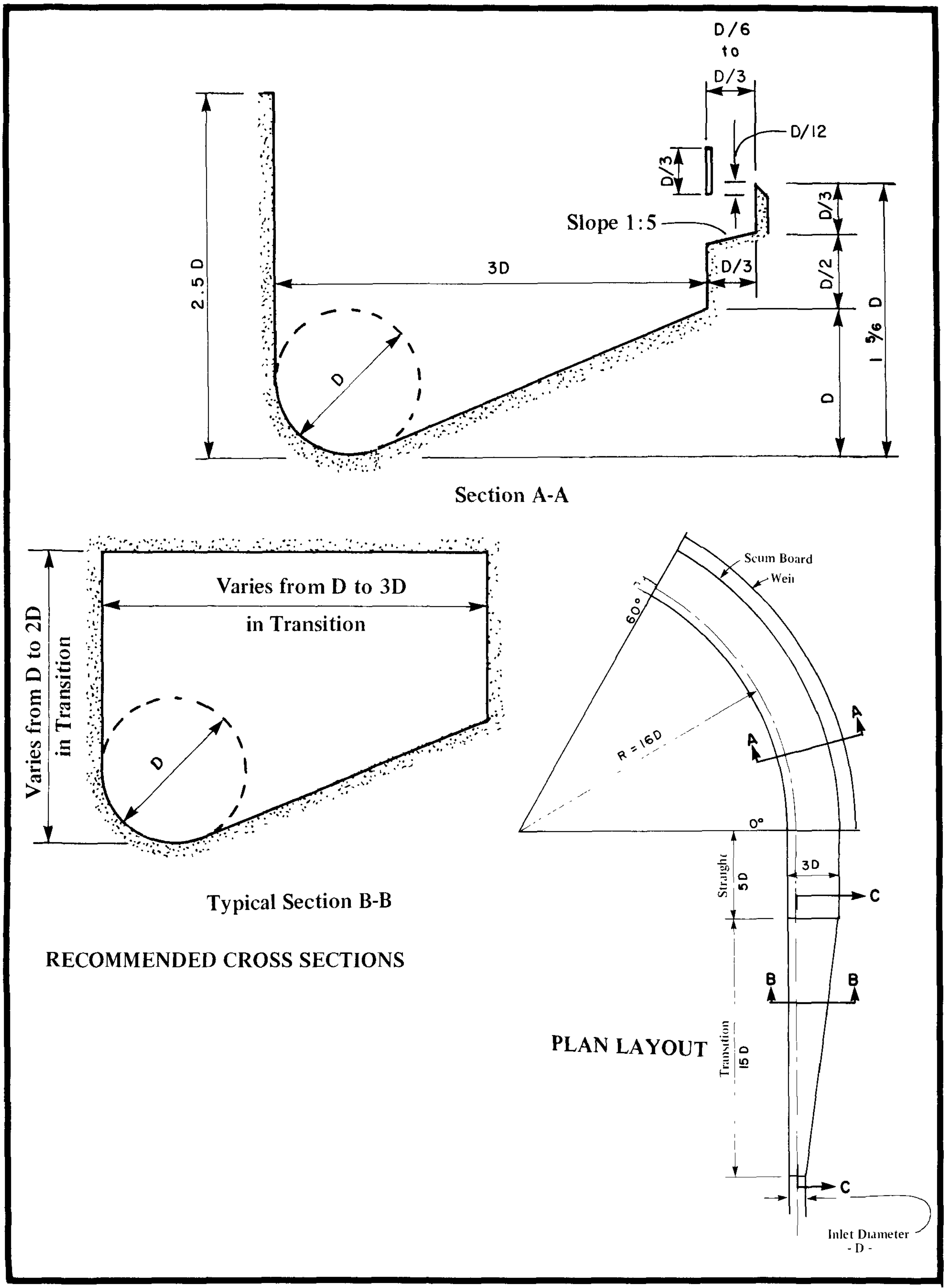


Figure 47 Plan Layout and Cross Sections

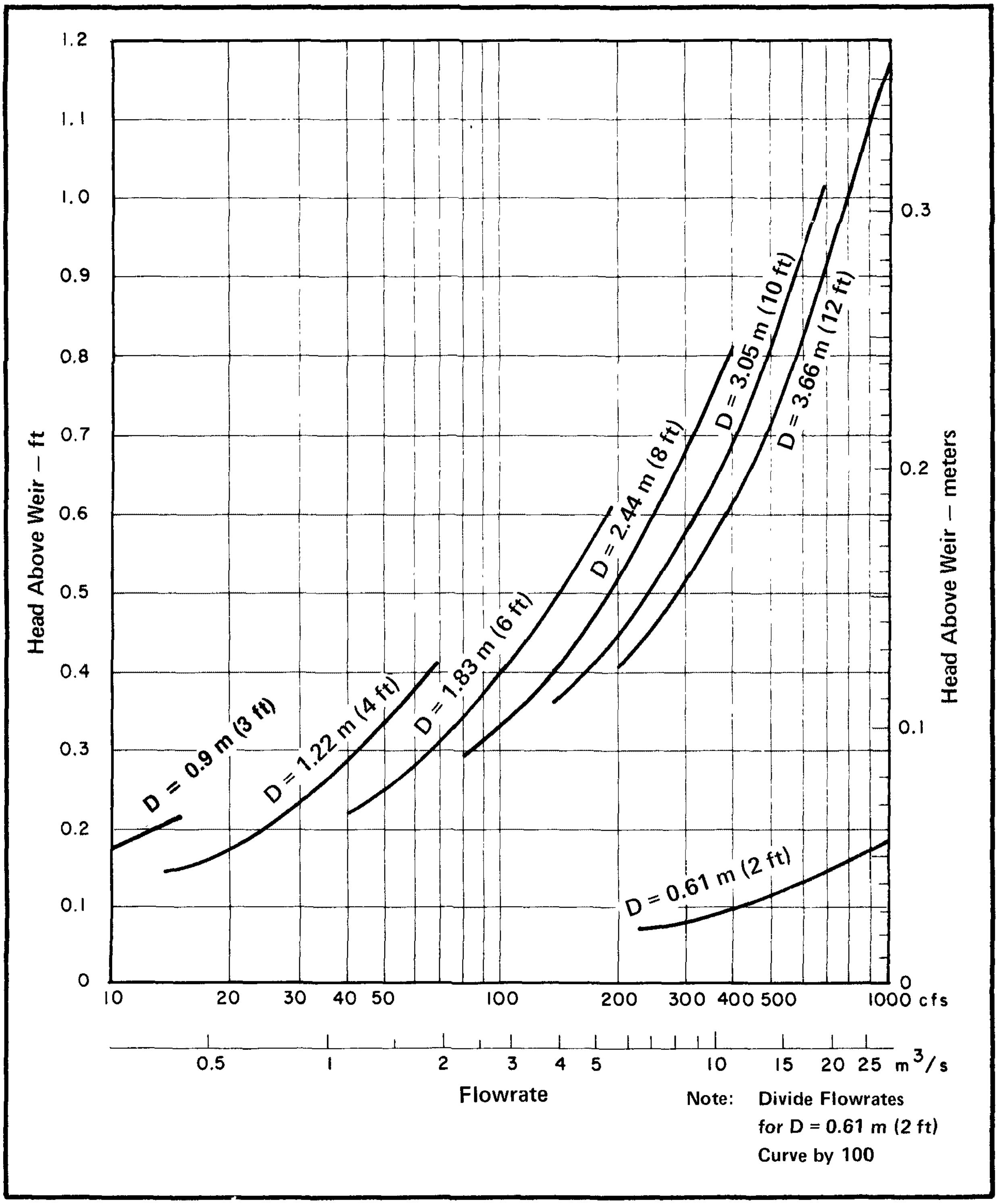


Figure 48 Water Level in Regulator Above Weir

Velocities in Transition

As a matter of interest the velocities at the inlet and outlet ends of the transition were computed for five values of D, from Figure 48. The D values selected were 0.91, 1.22, 1.52, 1.83 and 2.13 m (3,4,5,6, and 7 ft). The results are shown in Table 12. These data indicate an outlet velocity from the transition ranging from 0.22 m/sec (0.71 fps) to 0.36 m/sec (1.17 fps). This compares with the usual criteria of velocities between 0.23 m/sec (0.75 fps) and 0.38 m/sec (1.25 fps) in a rectangular grit channel with velocity control. In general, the outlet velocities are about one-sixth the inlet velocities. All velocities are based on the sections flowing full.

Channel Slope

The channel should be given enough slope to maintain a self-cleansing velocity of 0.61 m/sec (2.0 fps) with DWF (average dry-weather flow).

Table 12
Velocities in Transitions

Inlet Diameter	Design Discharge	Area Inlet	Area Outlet	Velocity Inlet	Velocity Outlet
0.91 m	0.85 m^3/s	0.65 m^2	3.93 m^2	1.31 m/s	0.22 m/s
(3 ft)	(30 cfs)	(7.0 sf)	(42.3 sf)	(4.3 fps)	(0.71 fps)
1.22 m	1.84 m^3/s	1.17 m^2	6.98 m^2	1.58 m/s	0.26 m/s
(4 ft)	(65 cfs)	(12.6 sf)	(75.2 sf)	(5.2 fps)	(0.86 fps)
1.52 m	3.11 m^3/s	1.82 m^2	10.9 m^2	1.71 m/s	0.29 m/s
(5 ft)	(110 cfs)	(19.6 sf)	(117 sf)	(5.6 fps)	(0.94 fps)
1.83 m	4.96 m^3/s	2.63 m^2	15.7 m^2	1.89 m/s	0.30 m/s
(6 ft)	(175 cfs)	(28.3 sf)	(169 sf)	(6.2 fps)	(1.0 fps)
2.13 m	7.65 m^3/s	3.58 m^2	21.4 m^2	2.13 m/s	0.36 m/s
(7 ft)	(270 cfs)	(38.5 sf)	(230 sf)	(7.0 fps)	(1.17 fps)

The following example is based upon what would be a maximum ratio of dry-weather flow to design flow. Generally the ratio will be less.

Assume the following:

D	=	0.91 m (3.0 ft)
Design flow rate	=	0.85 cu m/sec (30 cfs)
DWF	=	1 percent of design flow rate
	=	0.008 cu m/sec (0.3 cfs)
Peak DWF	=	0.025 cu m/sec (0.9 cfs)

From a chart showing hydraulic properties of circular sections when the flow rate is one percent of the full section, the depth is seven percent of the full depth and the velocity is 31 percent of the velocity of the full section when flowing full at a velocity of 0.61 m/sec (2.0 fps), divided by 0.31, or 1.98 m/sec (6.5 fps). From a nomograph of flow for Manning n = 0.013, the required slope of the channel for a diameter of 0.91 m (3.0 ft) is 0.48 percent. If the peak dry-weather flow is 3 DWF the following data prevail:

When slope is 0.48%

$Q = 0.025$ cu m/sec (0.90 cfs)
$d = 0.11$ m (0.36 ft)
$v = 0.58$ m/sec (2.0 fps)

The foregoing assumes a circular section in the channel when flow is 1 percent and 3 percent of design flow rate. From a visual comparison of a large-scale section of channel with a circular section it is evident that flow conditions in a circular section will prevail for the depths of flow considered above. *The foregoing indicates that peak dry-weather flows should* cause no deposition in the channel.

Weir Discharge

Previous research on side overflow weirs indicates that with a relatively high weir, as proposed in the helical separator, the usual weir discharge equations provide a reasonable basis of design. The usual equation is as follows:

$$Q = C L H^{3/2}$$

where

Q = flow rate in cu m/sec (cfs)
C = coefficient
L = length of weir in m (ft)
H = head on weir in m (ft)

The coefficient C varies depending on whether the weir is sharp crested or broad crested and depending on the head and width of the weir.

Experience in Great Britain, where side overflow weirs have been used more widely than in the United States, favors the use of a weir with a semicircular shape. This shape seems preferable for the helical separator.

The coefficient of a broad crested weir varies with the width of crest and head on the weir. For the widths and heads likely to occur in the helical separator the value of C (for U.S. customary units) may range from 2.8 to 3.3. The use of a C value of 3.0 (for U.S. customary units) for design purposes is suggested. An example follows:

Design flow rate	=	0.85 cu m/sec (30 cfs)
D	=	0.91 m (3.0 ft)
L (weir length)	=	18.83 D
	=	17.2 m (56.5 ft)
Assume flow to plant	=	0.028 cu m/sec (1.0 cfs)
Q (over weir)	=	0.82 cu m/sec (29.0 cfs)
In U.S. Customary Units Q	=	$CLH^{3/2}$
If C = 3.0 then H	=	0.095 m (0.32 ft)

The weir height is 1-5/6 D from Figure 47. Therefore:

Weir Height	=	1.68 m (5.50 ft)
Head on weir	=	0.095 m (0.32 ft)
Water Depth	=	1.77 m (5.81 ft)

However, to meet the laboratory demonstrated requirements that the transition be flowing full, the water depth should be 2D or 1.83 m (6.0 ft). Therefore, in this case the weir height should be a minimum of 1.73 m (5.68 ft) so that the transition outlet is flowing full when design flow rate occurs.

Outlet Control

Various methods of controlling the flow from combined sewer overflow regulators are discussed in an EPA report (13). This report indicates that close control of the outlet flow requires the use of a sluice gate controlled by a float and actuated by either water power or an electric motor. On smaller structures where the use of such devices is not justified, one method of control is by use of a manually-operated gate. A HydroBrake(R) would also be effective for all structure sizes. The intent is to only operate such gates to clear them of debris or to change the opening size.

The use of such gates may result in considerable variation in the flow diverted to the treatment plant. This may not be serious when this flow is only a small percentage of the total tributary to the plant. To indicate the possible range in flow, the following example is based on the use of a manually-operated gate on the outlet to the treatment plant. The mininum size gate used should be 0.20 m (0.67 ft) square but a gate with a minimum size of 0.30 m (1.0 ft) square is preferable.

Legend

A	=	Cross-sectional Area
D	=	Diameter
V	=	Velocity
d	=	Depth of flow
Q	=	Flow rate
b	=	Width of opening
C	=	Coefficient - 0.7
DWF	=	Average dry-weather flow
g	=	9.81 m/sec^2 (32.2 ft/sec^2)

Pertinent Data

DWF	=	0.008 cu m/sec (0.30 cfs)
Peak DWF	=	0.025 cu m/sec (0.90 cfs)

Try sluice gate 0.30 m (1.0 ft) square
Assume opening 0.10 m (0.33 ft) high
Then A = 0.03 sq m (0.33 sf)

Determine depth upstream of gate when:

Q = 0.025 cu m/sec (0.90 cfs)
Q = C A $\sqrt{2gH}$
0.025 cu m/sec (0.9 cfs)
= 0.7 x 0.031 sq m (0.33 sf) x 4.43 (8.03) x$\sqrt{H}$
H = 0.21 m (0.69 ft) on center line of orifice

Depth of flow is H, plus one-half height of orifice, or 0.26 m (0.86 ft). This is much greater than the normal depth of flow at the peak dry-weather flow of 0.11 m (0.36 ft) computed previously. Therefore, the velocity will be much less than the 0.58 m/sec (2.9 fps) computed previously and may cause deposition of grit at peak dry-weather flow.

Determine flow to the treatment plant when the water level is at weir crest.

Depth of flow in chamber is 1.83 m (6.0 ft)
Head on center of orifice is 1.77 m (5.83 ft)
Q = C A $\sqrt{2gH}$
= 0.7 x 0.03 sq m (0.33 sf) x 4.43 (8.03) 1.33 m (2.41 ft)
= 0.127 cu m/sec (4.46 cfs) = 15 DWF

Hence, the flow to the plant will exceed 15 DWF during periods of design discharge if there is no further restriction to flow downstream of the sluice gate. One way to restrict the flow is to design a sewer between the sluice gate manhole and the interceptor in such a way that it will convey the peak dry-weather flow without surcharge but will become surcharged when the flow exceeds the peak dry-weather flow. This procedure is described and illustrated by an example in an EPA report (13).

Principles of sewer design must be maintained in the design of the outlet. The size of the pipe must be sufficient to allow blockages to be removed readily from a convenient point of access. The slope of the line must be sufficient to maintain self-cleansing velocities through the unit and the outlet pipe. To be self-cleansing, velocities through the unit should exceed 0.6 m (2 ft) per second at low flow in order that solids will not be retained.

Spillway Channel

The side channel in the helical section which conveys the overflow from the weir to the outlet sewer leading to the stream should be designed for the maximum flow expected to pass through the separator. The maximum flow will depend on the storm frequency for which the combined sewer is designed, as well as on the extent to which the combined sewer can be surcharged by storm flows greater than the design flow. It should also be assumed that the pipe outlet to the treatment plant is not in use either by design or by accident. On this basis it is possible for the maximum flow to exceed the design flow rate by 50 to 100 percent.

As an example, assume that the design flow rate is 0.85 cu m/sec (30 cfs) and the maximum flow is 1.27 cu m/sec (45 cfs). The side channel can be designed as a lateral spillway channel with the weir discharge spilling into it throughout its length. To aid in self-cleaning, it is desirable to set the downstream end of the channel above the invert of the outlet pipe and to provide a slope in the channel so that at low depths of flow the velocity will exceed 0.31 m/sec (1 fps).

The channel should be designed large enough so that the upstream water surface will not cause submergence of the weir.

The general equation for determing the depth of flow in a lateral spillway channel is the following:

$$h_o = \sqrt{\frac{h_c^3}{h_1} + \left(h_1 - \frac{1}{3}\,\underline{il}\right)^2} - \frac{2}{3}\,\underline{il}$$

where

h_o = upstream water depth
h_c = critical depth
h_1 = downstream water depth when flow is submerged
$\underline{i}$ = slope of channel
$\underline{l}$ = length of channel

and

$$h_c^3 = \frac{Q^2}{gb} - 2$$

where

b = width of channel

The factors in the foregoing equation are depicted in Figure 49.

Actually, only 17.83/18.83 or 95 percent of the maximum flow discharges directly into the spillway channel. The balance is discharged by the foul sewer. In the following example, however, it is assumed that all the maximum flow is conveyed by the channel.

Assume the following data:

Design flow rate	=	0.85 cu m/sec (30 cfs)
Q (side channel)	=	1.27 cu m/sec (45 cfs)
Outlet pipe diameter	=	0.91 m (3.0 ft)
Weir height	=	1.83 m (6.0 ft)

Then:

Outlet velocity	=	1.95 m/sec (6.4 fps)
Entrance loss = $\frac{1.5V^2}{2g}$	=	0.29 m (0.96 ft)
Elevation invert outlet pipe	=	0.00 m (0.00 ft)
Elevation weir	=	1.83 m (6.00 ft)

Elevation water at entrance	=	1.21 m (3.96 ft)
Length of channel = length of weir, less outlet diameter	=	16.3 m (53.5 ft)

Initial computation indicated that a channel 1.83 m (6.0 ft) deep and 0.31 m (1.0 ft) wide would cause submergence of the weir. For maintenance purposes a minimum width of channel of 0.61 m (2 ft) is considered desirable.

Preliminary computation with zero slope and the downstream end of the channel at elevation 0 indicated a water depth at the upstream end of the channel of 1.49 m (4.9 ft).

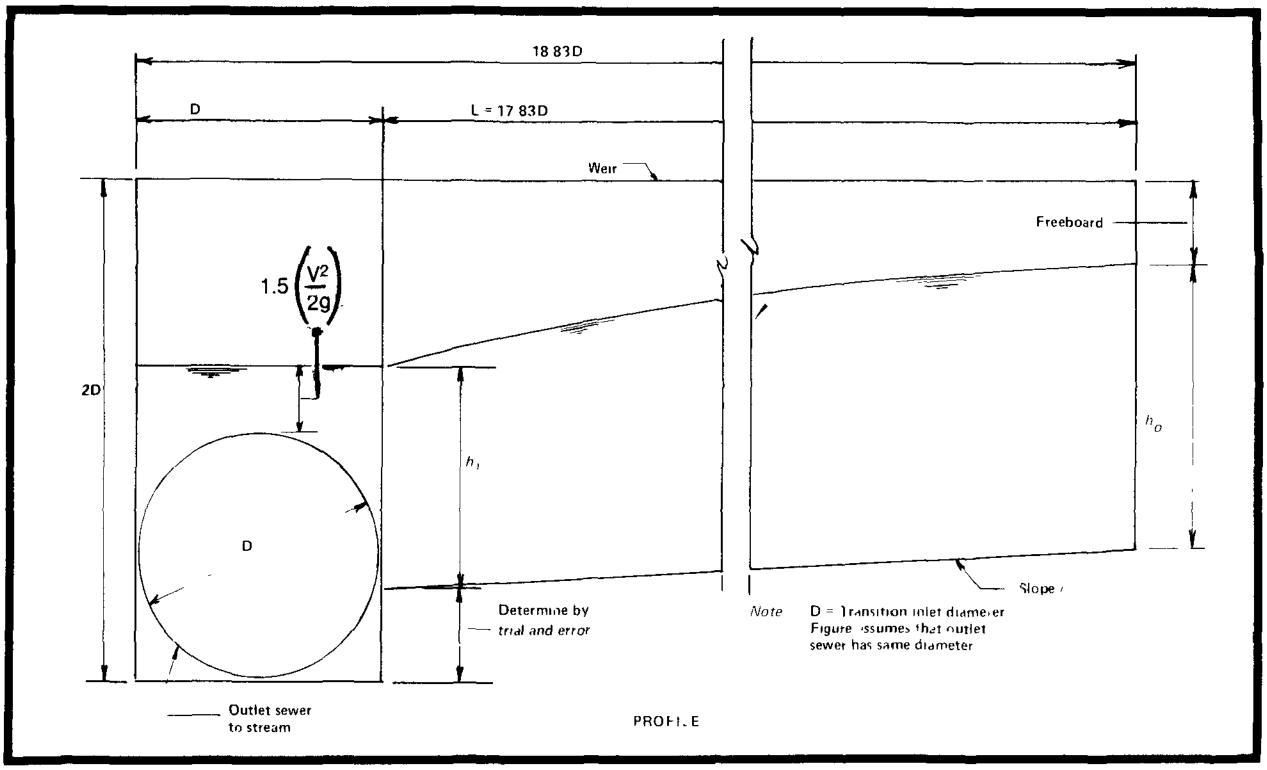


Figure 49 Spillway Channel Profile

The effect of submergence on broad crested weirs is surprisingly small. If necessary the fall in the water surface over the weir can be limited to 50 percent of the head on weir without affecting the discharge over the weir. As computed previously the head on the weir is 0.095 m (0.32 ft) and little elevation can be gained by assuming a submerged weir. Therefore, design can be based on no submergence of weir. It is also desirable to locate the downstream end of the discharge channel above the outlet pipe invert to prevent deposition in the channel. Therefore the downstream end of the channel was set at elevation 0.30 m (1 ft) and the channel slope set at 0.005. The resultant freeboard of 0.18 m (0.6 ft) indicated that the channel could have been raised an additional 0.18 m (0.67 ft). The final data are as follows:

Elevation downstream invert of channel	0.3 m (1.0 ft)
Rise in channel	0.09 m (0.3 ft)
Elevation upstream invert of channel	0.40 m (1.3 ft)

h_o	1.25 m (4.1 ft)
Elevation upstream water surface	1.65 m (5.4 ft)
Elevation weir	1.83 m (6.0 ft)
Freeboard	0.18 m (0.6 ft)

Design Example

The design calculations for a helical bend combined sewer overflow regulator/separator are illustrated in Table 13.

Table 13

Design Example, Helical Bend Combined Sewer Overflow Regulator/Separator

Sample Computations

Note: Conversion factors

1 ft = 0.305 m
1 cfs = 28.32 l/sec

D = Diameter inlet pipe
D_0 = Diameter outlet orifice
S = Slope
D_1 = Diameter outlet pipe
n = Manning roughness coefficient
v = velocity, max flow
v_1 = velocity, design flow
v_2 = velocity, peak DWF
v_D = velocity, discharge pipe
v_3 = peak velocity, discharge pipe
Q = DWF
Q_D = design flow
Q_2 = max flow
Q_3 = peak dry-weather flow (3 DWF)
Q_4 = flow, discharge pipe
d_1 = depth of design flow in pipe
d_2 = depth of peak DWF
d_D = depth of flow in discharge pipe
DWF = dry-weather flow
g = 32 ft/sec/sec
A = area
K = Rehbock K
L = unit length of weir
L_1 = length of throttle pipe
H = head on weir
C = coefficient for orifice discharge

Straight Pipe

Assume pipe design so that at DWF the velocity is about 2 fps

$D = 3.0$ ft $\qquad S = 0.44\%$ $\qquad n = 0.013$

$Q_2 = 45$ cfs $\qquad v = 6.5$ fps

For Q_D (design) $= 30$ cfs

$$\frac{Q_D}{Q_2} = \frac{30}{45}$$

$$\frac{d_1}{D} = 0.6 \text{ from standard charts}$$

$$d_1 = 0.6 \times 3.0 = 1.8 \text{ ft}$$

$$\frac{v_1}{v} = 1.07 \text{ from standard charts}$$

$$v_1 = 1.07 \times 6.5 = 7.0 \text{ fps}$$

For Q (DWF) $= 0.3$ cfs

$$\frac{Q_D}{Q_2} = \frac{0.3}{45} = 0.007$$

$$\frac{d_1}{D} = 0.06 \qquad d_1 = 0.06 \times 3 = 0.18 \text{ ft}$$

$$\frac{v_1}{v_2} = 0.29 \qquad v_1 = 0.29 \times 6.5 = 1.9 \text{ fps}$$

OK, almost 2.0 fps

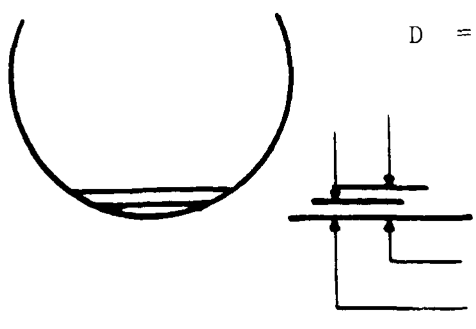
For Q_3 (Peak DWF) $= 0.9$ cfs

$$\frac{Q_3}{Q_2} = \frac{0.9}{45} = 0.02$$

$$\frac{d_2}{D} = 0.1 \qquad d_2 = 0.1 \times 3 = 0.3 \text{ ft}$$

$$\frac{v_1}{v_2} = 0.4 \qquad v_2 = 0.4 \times 6.5 = 2.6 \text{ fps}$$

Straight Pipe



$D = 3$ ft

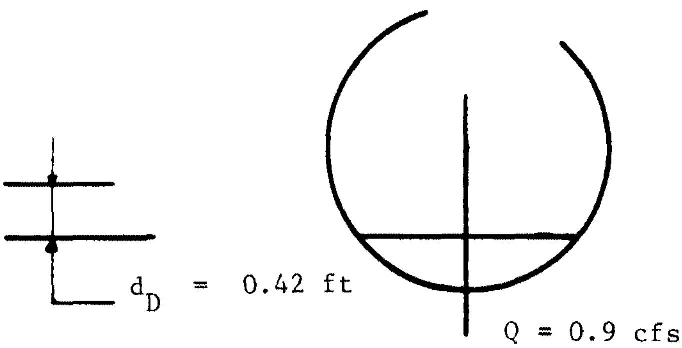
$d_2 = 0.3$ ft $\qquad Q_3 = 0.9$ cfs

$d_1 = 0.18$ ft $\qquad Q = 0.3$ cfs

At 3 DWF the area will only be slightly larger in the separator than in 3 ft diameter pipes. Assume velocity the same

$v_2 = 2.6$ fps

Exit Pipe -- Assume 1.0 ft diameter (D_1)



$S = 0.44\%$

$Q_D = 2.4$ cfs $\quad v_D = 3.0$ fps

$\frac{Q_3}{Q_D} = \frac{0.9}{2.4} = 6.37$

$\frac{d_D}{D_1} = 0.42 \qquad d_D = 0.42$ ft

$\frac{v_D}{v_3} = 0.92 \qquad v_D = 0.92 \times 3 = 2.8$ fps

Lower invert of outlet pipe 0.12 ft below invert of separator so as not to raise water surface

Determine outlet design

when	Q_D	= 30 cfs	
so that	Q_4	= 0.9 cfs	
Weir Length	Angle	= 60^o	D = 3 ft

$$\text{Weir Radius} = 16\ D + 2.5\ D + D/3$$
$$= (16)\ (3) + (2.5)\ (3) + 1$$
$$= 56.5\ \text{ft}$$
$$\text{Weir Length} = \frac{60}{360}\left(2\ \pi\ R\right) = \frac{2\pi}{6}\left(5.65\right)$$
$$= 59\ \text{ft}$$

Head on Weir

$$\text{Q per ft} = \frac{30}{59} = 0.51\ \text{cfs}$$

Use Rehbock K

$$Q = KLH^{3/2} = 3.41\ (1)\ H^{3/2} = 0.51$$
$$H^{3/2} = 0.15$$
$$H = 0.28 \text{ ft}$$

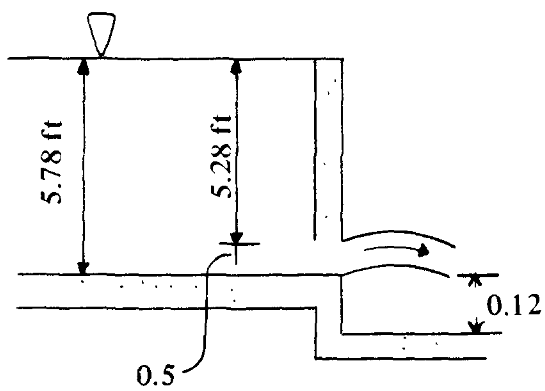
Depth of Water

$$1\text{-}5/6\ D = \frac{11}{6}(3) = 5.50 \text{ ft}$$
$$\text{Head on weir} = 0.28$$
$$\text{Total water depth} = 5.78 \text{ ft}$$

Assume short tube exit

$$D_0 = 1.0 \text{ ft}$$
$$A = \frac{\pi D^2}{4} = 0.785 \text{ sf}$$
$$Q_4 = CA\sqrt{2gH}$$
$$= (0.7)\ (0.785)\ (8.03)\ \sqrt{5.28}$$
$$= 10.1 \text{ cfs}$$

Too large



Outlet Design

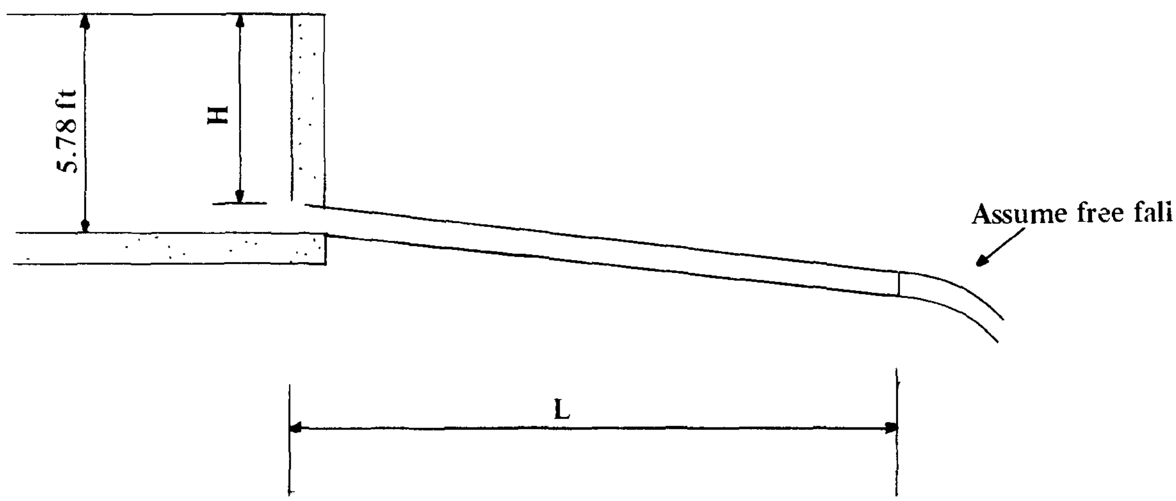
Determine orifice area for

$$Q_3 = CA\ \sqrt{2gH}$$
$$0.9 = (0.7)\ (A)\ 8.03\ \sqrt{5.28}$$
$$A = 6.9 \div 12.9 = 0.07$$
$$A = \frac{\pi D^2}{4}$$
$$D_1 = \sqrt{\frac{0.07}{0.785}} = \sqrt{0.09} = 0.3 \text{ ft}$$

Orifice should be greater than 0.67 ft. If orifice is made this size so that only 0.9 cfs or 3 DWF would pass when unit is full, then it is apparent that the unit would fill up whenever 3 DWF occurs which might be the peak daily flow.

To prevent deposition of solids on the separator floor and to prevent cleaning the separator in dry weather periods, the separator and outlet should pass up to 3 DWF without raising levels in separator to weir levels.

Try throttle pipe on outlet



Outlet Design

Use 8-in. pipe as minimum
Use S = 0.4% as minimum
For n = 0.013

Q_4 = 0.75 cfs $\quad v_D$ = 2.2 fps $\frac{v^2}{2g}$ = 0.08 ft

For

QDWF	=	0.3 cfs	
$\frac{Q_1}{Q_2}$	=	$\frac{0.3}{0.75}$	= 0.4

If discharge ratio is 0.4, then depth is 44% and velocity 94%

D_0 = 0.44 x 0.67 = 0.3 ft
V_D = 0.94 x 2.2 = 2.1 fps

Assume

L_1 = 100 ft $\quad$ H = 5.78 - 0.67 = 5.11 ft

For

Q_3DWF = 0.9 cfs
v_3 = 2.5 fps $\quad \frac{v^2}{2g}$ = 0.1 $\quad$ S = 0.5%

Entrance & exit loss	1.5 x 0.1	=	0.15
Slope hydraulic gradient	100 x 0.5%	=	0.50
			0.65
Actual pipe slope			0.40
Water surface above top pipe			0.25
Water surface in tank			0.67
Depth water in tank			0.92 ft

Determine Q_4(max) when separator is full

Assume entrance and exit loss = 1.2 ft

$$\text{Slope H.G} = \frac{2(5.11 - 1.2)}{100} 100 + 0.4 = 4.3\%$$

$$Q_{4\,(max)} = 1.6 \text{ cfs} \qquad v = 7.2 \text{ fps} \qquad \frac{v^2}{2g} = 0.8$$

Thus with 8 in. throttle pipe 100 ft long the maximum Q will be 1.6 cfs or about 5 times DWF.

Try other lengths of pipe

L ft	5% HG	Q_4(max) cfs	$v_3$fps	$\frac{v^2}{2g}$	Q_4(DWF)
200	2.45	1.8	5.4	0.5	6
300	1.9	1.6	4.7	0.4	5.3
400	1.45	1.4	4.0	0.3	4.7
600	1.2	1.3	3.8	0.2	4.3

A length of 400 ft should be the maximum for an 8 in. sewer. Therefore, it is obvious that if the discharge is to be limited to 3 DWF some type mechanical device should be used to close the outlet opening as the water level rises in the separator.

Determine depth of water in separator with throttle pipe 400 ft long, when Q_3= 0.9 cfs

Required S = 0.5% (See previous page)		
400 x 0.5% = 2.0 ft		2.0 ft
Slope sewer 400 x 0.4%		1.6
		0.4 ft
Entrance & exit loss	$1.5 \frac{v^2}{2g}$	0.15
Head on top of pipe		0.55 ft
Diameter pipe		0.67
Depth of water in tank		1.22 ft

TYPICAL DIMENSIONS

In order to develop cost estimates, three design discharges were chosen and helical bend separators sized for each. Table 14 presents typical dimensions for flows of 1.42, 2.83, and 4.67 cu m/sec (50,100, and 165 cfs).

Table 14
Helical Bend Combined Sewer Overflow Regulator/Separator Dimensions

Design discharge	m^3/s	1.42	2.83	4.67
	(cfs)	(50)	(100)	(165)
Inlet diameter	D·m	1.07	1.52	1.83
	(ft)	(3.5)	(5.0)	(6.0)
Transition length	15D·m	16.0	22.9	27.4
	(ft)	(52.5)	(75.0)	(90.0)
Straight section — length	5D·m	5.33	7.62	9.14
	(ft)	(17.5)	(25.0)	(30.0)
Radius	16D·m	17.1	24.4	29.3
	(ft)	(56.0	(80.0)	(96.0)
Width	3D·m	3.20	4.57	5.49
	(ft)	(10.5)	(15.0)	(18.0)
Minimum wall height	2.5D·m	2.67	3.81	4.57
	(ft)	(8.75)	(12.5)	(15.0)
Channel to top weir	1 5/6 D·m	1.95	2.77	3.35
	(ft)	(6.4)	(9.1)	(11.0)
Height end of transition	2D·m	2.13	3.05	3.66
	(ft)	(7.0)	(10.0)	(12.0)
Scum baffle height	D/3·m	0.36	0.52	0.61
	(ft)	(1.2)	(1.7)	(2.0)
Distance from weir to bottom of baffle	D/12·m	0.09	0.13	0.15
	(ft)	(0.3)	(0.4)	(0.5)
Weir height	D/3·m	0.36	0.52	0.61
	(ft)	(1.2)	(1.7)	(2.0)
Distance wall to weir (Max)	D/3·m	0.36	0.52	0.61
	(ft)	(1.2)	(1.7)	(2.0)
(Min.)	D/6·m	0.18	0.26	0.30
	(ft)	(0.6)	(0.85)	(1.0)

SITE REQUIREMENTS

The location and depth of the combined sewer will determine the area required for its installation. The depth of the sewer may suggest that an underground chamber is appropriate. If this is the case or if adjacent land is expensive, it may be desirable to construct a chamber for the separator along the existing right-of-way of the sewer. Figure 50 shows the site requirements assuming that 2 m (6 ft) is allowed for construction clearance and the thickness of the structure. Figure 51 shows the site requirements for a swirl unit.

The size of the buffer or protective zone required around the uncovered helical separator will depend to a large extent on the environment of the neighborhood. In any locality, a buffer zone at least 15.2 m (50 ft) wide would be desirable. Therefore, the site requirements given herein are based on a 15.2 m (50 ft) buffer zone around all open or above ground parts of the facility. Because the transition is below the surface, no buffer zone is required for that part of the structure; however, it is assumed that all of the transition section is located on the site.

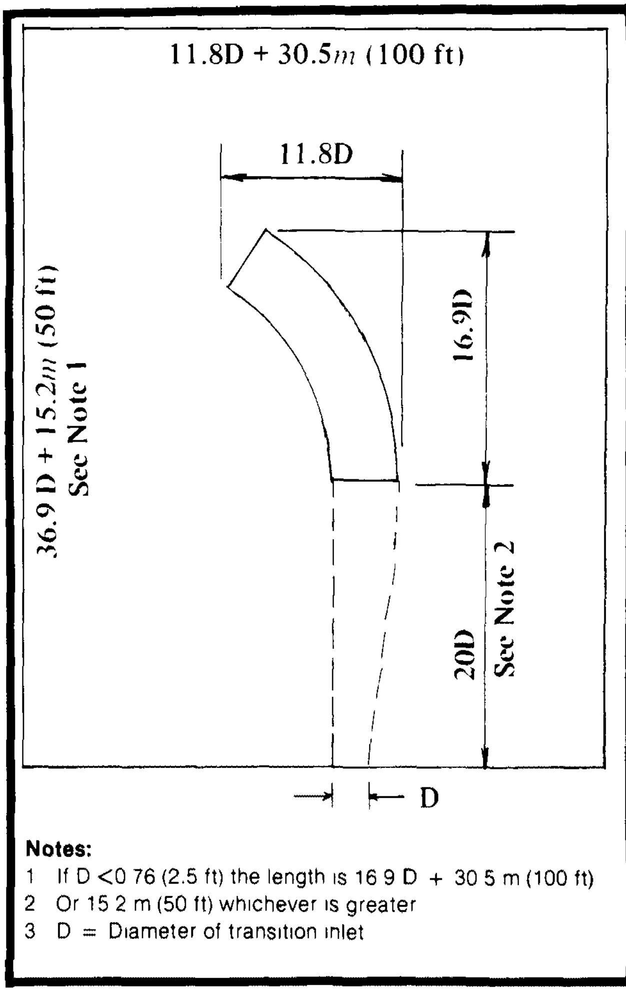


Figure 50 Site Requirements, Helical Bend Combined Sewer Overflow Regulator/Separator

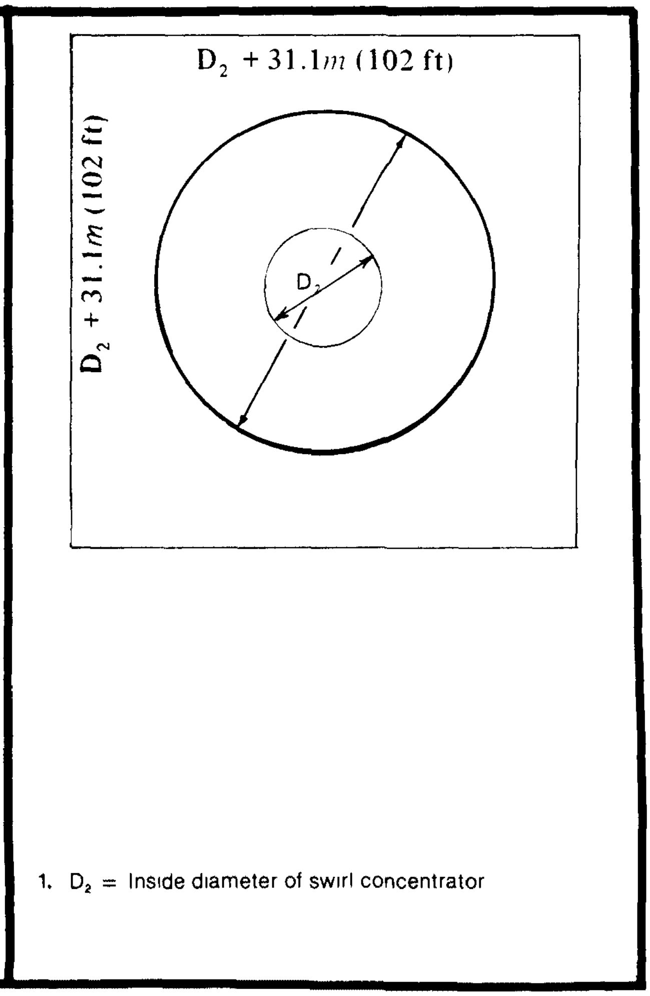


Figure 51 Site Requiements, Swirl Concentrator Combined Sewer Overflow Regulator/Separator

HYDRAULIC HEAD LOSSES

The available head at a specific site may be a critical factor in the choice of the specific type of combined sewer regulator to be used. The head loss must be considered for two conditions: 1) For periods of dry-weather flow, and 2) for periods of wet-weather flow. The available head during dry-weather flow will depend on the difference in elevation between the combined sewer and the interceptor that will convey the flow to the wastewater treatment plant. The available head during wet-weather flow will depend on the difference in elevation between the combined sewer and the water surface of the receiving stream. A further consideration in the latter case is whether the existing combined sewer is to be used to convey the overflow from the regulator to the receiving stream or to any holding or treatment facilities involved.

First, consider the case where there is to be no surcharge on the inlet during design discharge.

In the helical separator the transition will have a level top. The drop in the invert of the transition will be 1 D. Therefore, the drop in the invert from the inlet to the foul outlet will be 1 D (neglecting the slope of the channel through the regulator). The loss in the hydraulic gradient will also be 1 D. The invert of the clear outlet will be at approximately the same elevation as the invert of the separator, as explained previously in the discussion of the weir overflow spillway channel. Therefore, the drop in the invert between the inlet and the clear outlet will also be 1 D. The loss in the hydraulic gradient may be the same as the drop in the invert or it may be slightly different depending on outlet design.

When the inlet sewer is surcharged different hydraulic conditions will exist in the helical separator. If the inlet is surcharged an amount equal to D, the transition invert will be level, as shown in Figure 45. The drop in the invert from the inlet to the foul outlet will be zero (neglecting the channel slope through the separator). The drop in hydraulic gradient will also be zero. Likewise, the drop in the invert from the inlet to the clear outlet will be zero. However, the loss in hydraulic gradient for this case will be 1 D.

CONSTRUCTION DETAILS

Means of access must be provided to the curved section of the separator for maintenance purposes, including possible washing down after each storm event. The provision of a superstructure over this section is desirable for safety and aesthetic reasons and for confining possible odors. The type of superstructure used will depend on the character of the locality. As a minimum and for purposes of this report, the walls are assumed to be of concrete block and the roof of precast concrete units. For roof spans exceeding about 8.5 m (28 ft), it will be necessary to provide structural steel framing. The facility could be constructed of poured concrete, Gunite or fiber glass.

A cross section of the helical separator with a superstructure is shown in Figure 52.

In this Figure, for cost estimating purposes, the width of structure is indicated as 4.1 D and the width of spillway channel as 0.67 D. For any specific case the width and elevation of the spillway channel may vary from that shown in Figure 52 as explained previously.

The hydraulic conditions require that the transition section be provided with a roof. These conditions do not apply to the straight section, having a length of 5 D, preceding the curved section. It is believed that this section will not require the same maintenance as the curved section. Accordingly, there appears to be no need to make this section accessible.

For purposes of costing it has been assumed that the straight section will have walls 2.5 D high and will be provided with a concrete roof at that elevation.

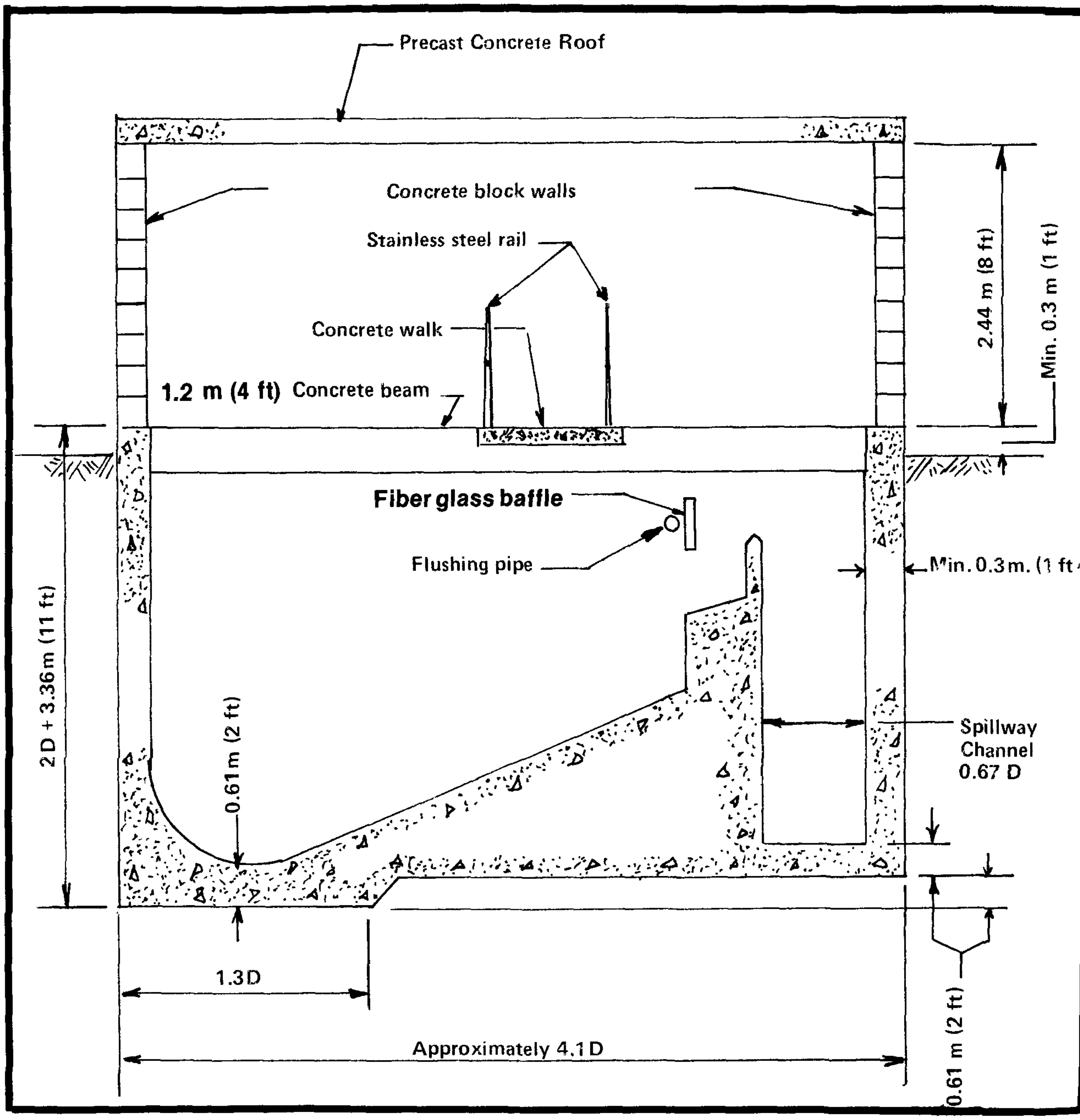


Figure 52 Typical Cross Section — Helical Bend Separator

Other construction details considered necessary or desirable are as follows:

A. Provide concrete walls with a minimum thickness of 0.3 m (1 ft) extending above grade a minimum height of 0.3 m (1 ft).

B. Coat all concrete surfaces with an epoxy paint. This will reduce maintenance by decreasing deposition of solids on the walls of the structure.

C. Provide a concrete walk 1.2 m (4.0 ft) wide.

D. Provide a stainless steel railing on each side of the walk.

E. Provide a fiber glass scum baffle hung from the beams or supported from the weir.

F. Provide a flushing water pipe on the channel side of the scum baffle and hung from the beams. Connect this line to the public supply with a backflow device if this is permitted by local code. If this is not permissible, provide a storage tank to store overflow from the weir and a submersible pump to use for washing down. The usual criteria of 3.1 l/sec (50 gpm) at 28.120 N sq m (40 psi) for flushing purposes at treatment plants should be applicable to the helical separator facility. Hose connections should also be provided in case the stream from the wash water pipe is not effective. Provide additional nozzles at the end of the structure to assist in the removal of floatables which will be concentrated at this location as flow returns to dry-weather conditions.

G. Provide concrete block walls with a height of 2.4 m (8.0 ft).

H. Provide a precast concrete roof.

I. Provide adequate electric lights.

J. Provide roof ventilators.

K. Provide doors at both ends of the structure for ventilation and access.

Cost estimates of the helical separator were made for two purposes: 1) to indicate the probable construction cost of the facility; and 2) to compare its costs with that of the swirl separator used as a combined sewer regulator.

The cost estimates are considered to be reasonable engineers' estimates. However, during periods of economic inflation, it is not unusual for contractors' bids to materially exceed engineers' estimates.

In making a choice between the helical separator and the swirl separator, it is possible that other factors such as space available or depth of the combined sewer, related to the specific site of the facility, greatly influence construction costs.

QUANTITIES COST ESTIMATE

The estimated quantities are based on the following:

A. The transition will be constructed with a drop in the invert equal to D as shown in Figure 45 so that the sewer upstream of the transition will not be surcharged.

B. The straight section preceding the curved section will have walls 2.5 D high and concrete roof.

C. The superstructure over the curved section will be as shown in Figure 51.

D. The width of the curved section is assumed to be 4.1 D: the width of the spillway channel is assumed to be 0.67 D.

E. The cover on the sewer at the transition inlet will be 2.44 m (8 ft).

F. The ground is level and the subsurface is earth with no groundwater problems.

G. All concrete walls will have a minimum thickness of 0.3 m (1 ft) except the weir.

H. Sheet piling will be required about 0.6 m (2 ft) outside the structure.

I. Transverse concrete beams will be required at 4.5 m (15 ft) intervals with a cross section 0.45 m (1.5 ft) square.

J. The continuous concrete walk will be 1.22 m (4 ft) wide and 0.20 m (0.67 ft) thick.

COST CALCULATION

The costs are based on the following:

A. The Engineering News-Record Construction Cost Index average for the United States is 3140.

B. Unit prices are as follows:

Steel Sheet Piling	\$ 129/sq m	\$ 12/sq ft
Excavation	\$ 24/cu m	\$ 18/cy
Reinforced Concrete	\$ 490/ cu m	\$375/cy
Concrete Block Walls	\$ 129/sq m	\$ 12/sq ft
Roof	\$ 150/sq m	\$ 14/sq ft

C. Miscellaneous costs are assumed to be 25 percent of the foregoing items and to include a manual sluice gate and manhole handrail, flushing water facilities, scum baffle, electrical work, roof ventilators and doors.

D. The estimated cost of the bypass sewer during construction is based on providing a sewer of the same diameter as a transition inlet around the proposed separator, plus an allowance for temporary connections at each end.

E. Contingent and engineering costs will be 35 percent of the foregoing items.

Table 15 presents the estimated costs for units with 1.42, 2.83 and 4.67 cu m/sec (56, 100 and 165 cfs) flow.

Table 15
Construction Cost of Helical Bend Separator

Construction Cost of Helical Bend Regulator

Capacity 1.42 m^3/s (50 cfs)

Item	Quantity		Amount
Sheet piling	420 m^2	(4,550 sf)	$ 54,600
Excavation	950 m^3	(1,240 cy)	22,320
Reinforced concrete	250 m^3	(330 cy)	123,750
Concrete block walls	114 m^2	(1,230 sf)	14,760
Roof	85 m^2	(910 sf)	18,200
		Subtotal	233,630
Miscellaneous costs		25%	58,400
Bypass sewer			30,000
		Subtotal	322,030
Contingent and engineering costs		35%	112,710
		Total	$434,740

Capacity 2.83m^3 (100 cfs)

Item	Quantity		Amount
Sheet piling	710 m^2	(7,700 sf)	$ 92,400
Excavation	2,200 m^3	(2,800 cy)	50,400
Reinforced concrete	475 m^3	(620 cy)	232,500
Concrete block walls	160 m^2	(1,740 sf)	20,880
Roof	170 m^2	(1,800 sf)	25,200
		Subtotal	421,380
Miscellaneous costs		25%	105,350
Bypass sewer			58,300
		Subtotal	585,030
Contingent and engineering costs		35%	204,760
		Total	$789,790

Capacity 4.67 m^3/s (165 cfs)

Item	Quantity		Amount
Sheet piling	950 m^2	(10,200 sf)	$122,400
Excavation	3,200 m^3	(4,180 cy)	75,240
Reinforced concrete	679 m^3	(888 cy)	333,000
Concrete block walls	200 nm^2	(2,130 sf)	25,560
Roof	250 m^2	(2,700 sf)	37,800
		Subtotal	594,000
Miscellaneous costs		25%	148,500
Bypass sewer			83,700
		Subtotal	826,200
Contingent and engineering costs		35%	289,170
		Total	$1,115,370

PROTOTYPE

The helical bend separator was tested extensively in Nantwich, England. The first full size unit has been built in Boston, Massachusetts. A purpose of the demonstration project is to compare the efficiency of the unit as compared to a swirl separator/regulator. The unit will be tested on combined sewer overflows and stormwater discharges. Test results were not available at the time of preparation of this manual. Construction costs were very low due to the fact that the unit was prefabricated from wood and is not intended for permanent use. Figure 53 shows the completed facility in place. Design details are given in Table 16.

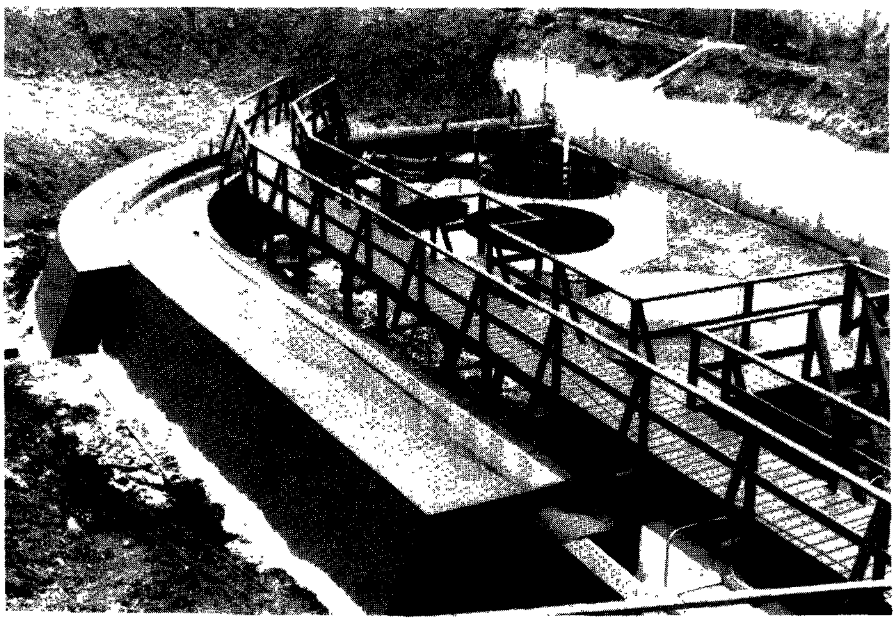


Figure 53 Helical Bend Regulator/Separator Prototype, Boston, MA

Table 16
Design Details — City of Boston
Helical Bend Combined Sewer Overflow
Regulator/Separator Prototype

Inlet diameter	m	0.45
	(ft)	(1.5)
Overall length	m	18.3
	(ft)	(60)
Outlet diameter		
Overflow	m	0.6
	(ft)	(2)
Weir length	m	9.15
	(ft)	(30)
Outlet to plant diameter	m	0.24
	(ft)	(0.66)
Design flow	l/s	170
	(cfs)	(6)
Maximum flow	l/s	340
	(cfs)	(12)
Maximum underflow	l/s	7.6
	(cfs)	(0.27)

Section IV

COMPARISON OF SWIRL REGULATOR/SEPARATOR AND HELICAL BEND COMBINED SEWER OVERFLOW REGULATOR/SEPARATOR

Design and cost information have been presented in the preceding two sections for different types of combined sewer overflow regulators. Both regulators have been designed to accomplish both the control of quantity and quality of the discharge to receiving waters. This section has been prepared to offer a basis for comparing the two separators.

It appears that the principal advantages of the helical bend separator are the low head requirements and the discharge to treatment of the captured solids at the end of the storm event. The swirl regulator/separator in turn requires less space and should be less expensive to construct. Use of the swirl regulator/separator where insufficient hydraulic head is available for its normal mode of operation may require dry weather bypassing the device. As both units have been designed to minimize the cost of operation and maintenance problems, they are considered comparable.

SITE REQUIREMENTS

The site requirements for both the helical and the swirl separator are shown in Figure 50. The required lot dimensions and area for three sizes of each facility are shown in Table 17.

The site dimensions are based on a helical separator to remove 100 percent of grit and a swirl separator to remove 90 percent of grit.

It is evident from Table 17 that the site requirements for the helical bend are greater than for the swirl separator and that the larger the design flow the greater the difference. For the design flows of 1.42 cu m/sec (50 cfs), 2.83 cu m/sec (100 cfs), and 4.67 cu m/sec (165 cfs); the helical bend requires a site 63 percent, 115 percent, and 145 percent greater, respectively, than the swirl separator.

HEAD LOSSES

A discussion of the computation of head losses for each regulator has been presented in the previous sections.

In the following comparisons, the head losses are given as a multiple of the inlet dimension: D, the inlet diameter of the helical separator; and D_1, the side of the square inlet of the swirl separator. To show that D and D_1 are approximately the same for the same discharge, their values for three discharges are given in the following section on Design.

Table 17
Site Dimensions and Areas for Helical Bend and Swirl Regulator/Separator

	Swirl Regulator	Helical Separator
Capacity 1.42 cu m/s (50 cfs)		
Site size	38.0 m x 38.0 m (124.5 ft x 124.5 ft)	43.0 m x 54.6 m (141 ft x 179 ft)
Site area	1,440 sq m (15,500 sf)	2,340 sq m (25,200 sf)
Relative area	1.00	1.63
Capacity 2.83 cu m/s (100 cfs)		
Site size	40 m x 40 m (131.5 ft x 131.5 ft)	48.5 m x 71.5 m (159 ft x 234 ft)
Site area	1,600 sq m (17,300 sf)	3,460 sq m (37,200 sf)
Relative area	1.00	2.15
Capacity 4.67 cu m/s (165 cfs)		
Site size	42 m x 42 m (138 ft x 138 ft)	52.0 m x 82.8 m (171 ft x 272 ft)
Site area	1,770 sq m (19,000 sf)	4,300 sq m (46,500 sf)
Relative area	1.00	2.45

DESIGN

First, consider the case where there is to be no surcharge on the inlet during design discharge.

Discharge	cu m/sec	1.42	2.83	4.67
	(cfs)	(50)	(100)	(165)
D	m	1.07	1.52	1.83
	(ft)	(3.5)	(5.0)	(6.0)
D_1	m	0.90	1.52	1.83
	(ft)	(3.0)	(5.0)	(6.0)

In the helical regulator the transition will have a level top. The drop in the invert of the transition will be 1 D. Therefore, the drop in the invert from the inlet to the foul outlet will be 1 D (neglecting the slope of the channel through the regulator). The loss in the hydraulic gradient will also be 1 D. The invert of the clear outlet will be at approximately the same elevation as the invert of the separator, as explained previously in the discussion of the weir overflow spillway channel. Therefore. the drop in the invert between the inlet and the clear outlet will also be 1 D. The loss in the hydraulic gradient may be the same as the drop in the invert or it may be slightly different depending on outlet design.

In the swirl regulator, if there is to be no surcharge on the inlet sewer, the crown must be at a distance above the invert of the chamber equal to H_1 (the height of weir above the chamber invert), plus the head on the weir. The drop in the invert of the sewer will be this distance less D_1, the dimension of the inlet. The foul outlet is located below the chamber bottom. Assuming a foul outlet diameter of 0.31 m (1 ft) and concrete cover over the outlet to the same amount, the distance from the chamber invert to the outlet invert is 0.61 m (2 ft).

Excluding the channel slope through the separator, the drop in the invert from the inlet to the foul outlet is, therefore, 0.8 D_1 to 1.5 D_1, plus 0.61 m (2 ft). The foul outlet pipe diameter may exceed 0.31 m (1 ft) diameter for larger flows, thus increasing the total drop somewhat. The hydraulic gradient will have a similar drop.

The clear outlet is also located below the chamber floor and, if a 0.31 m (1 ft) concrete cover is provided over the outlet, the vertical distance from the chamber invert to the invert of the clear outlet will be 1 D_1, plus 0.31 m (1 ft). Combining this with the entrance drop of 0.8 D_1 to 1.5 D_1, will result in a total drop in the invert from the inlet to the clear outlet of 1.8 D_1 to 2.5 D_1, plus 0.31 m (1 ft). The drop in the hydraulic gradient in this case will be different. The circular weir is set a distance equal to the head on the weir below the top of the inlet sewer. If there is no submergence of the weir then the loss in the hydraulic gradient will be equal to this head. Trial computations indicate the head on the weir is about 0.2 D_1. Allowing for friction losses in the outlet pipe and some freeboard downstream of the weir, the drop in hydraulic gradient is about 0.4 D_1.

When the inlet sewer is surcharged, different hydraulic conditions will exist in the helical separator. If the inlet is surcharged an amount equal to D, the transition invert will be level; as shown in Figure 45, the drop in the invert from the inlet to the foul outlet will be zero (neglecting the channel slope through the separator). The drop in hydraulic gradient will also be zero. Likewise, the drop in the invert from the inlet to the clear outlet will be zero. However, the loss in hydraulic gradient for this case will be 1 D.

In the case of the swirl separator if a surcharge of D_1 is permitted, then the crown of the sewer can be set a distance of D_1 below the water surface of the chamber. The drop from the chamber invert to the foul outlet will be 0.61 m (2 ft), as previously computed. Therefore, the drop in the invert from the inlet to the foul outlet will be 0 to 0.5 D_1, plus 0.61 m (2 ft). The drop in hydraulic gradient will be the same. The drop from the chamber invert to the clear outlet invert will be 1 D_1, plus 0.31 m (1 ft), as before. Therefore, the total drop from the inlet invert to the clear outlet invert will be 1 D_1 to 1.5 D_1 plus 0.31 m (1 ft). The drop in hydraulic gradient will be about 0.4 D_1 as computed previously.

The data relative to the foregoing discussion are shown in Table 18.

Table 18
Typical Head Losses in Helical Bend and Swirl Regulator/Separator

	Helical Separator	Swirl Concentrator w/o Surcharge	Swirl Concentrator Surcharged
Dry-weather flow — drop in invert			
Helical separator			
Transition invert level	none		
Transition roof level	1 D		
Swirl regulator		0.8 to 1.5D_1+ 61cm(2ft)	NA
Wet-weather flow			
Helical separator			
Transition invert level			
Hydraulic grade	1 D		
Drop in invert	none		
Transition roof level			
Hydraulic grade	1 D		
Drop in invert	1 D		
Swirl regulator			
Hydraulic grade		0.4D_1	0.4D_1
Drop in invert		1.8 to 2.5D_1+ 30cm(1ft)	1 to 1.5D_1+ 30cm(1ft)

Note: Friction losses not included in above table

From the Table, it is apparent that the drop in the invert is always greater in the swirl separator than in the helical.

When the inlet sewer is not surcharged, the drop to the foul outlet is only slightly greater, but the drop to the clear outlet is about twice as great. When the inlet sewer is surcharged an amount equal to D or D_1, the drop in the invert is zero in the helical separator, compared to the minimum drop of 0.61 m (2 ft) to the foul outlet and of 1 D_1, plus 0.31 m (1 ft) to the clear outlet in the swirl separator. For dry-weather flows, the drop in hydraulic gradient is similar to the drop in the invert. For wet-weather flows, the drop in hydraulic gradient from the inlet to the clear outlet in the swirl separator is about one-half that for the helical separator.

CONSTRUCTION COSTS

Utilizing the cost data developed in Sections II and III, Table 19 compares construction costs for flows of 1.42, 2.83 and 4.67 cu m/sec (50, 100, and 165 cfs).

Table 19
Comparison of Costs of Helical Bend and Swirl Regulator/Separator

Flow Capacity Efficiency		Helical Bend 100%	Swirl Regulator 90%
cu m/s	cfs		
1.42	50	370,000	152,000
2.83	100	687,000	246,000
4.67	165	972,000	340,000

Note: Cost based upon total project — land cost not included

These costs are shown graphically in Figure 54 as explained in Section II and III. Although the two regulators are sized for the same discharge, the helical separator will remove 100 percent of the grit compared to 90 percent for the swirl regulator, the usual design efficiency.

The costs of swirl regulator to remove 100 percent of grit were estimated and the results shown in Figure 54.

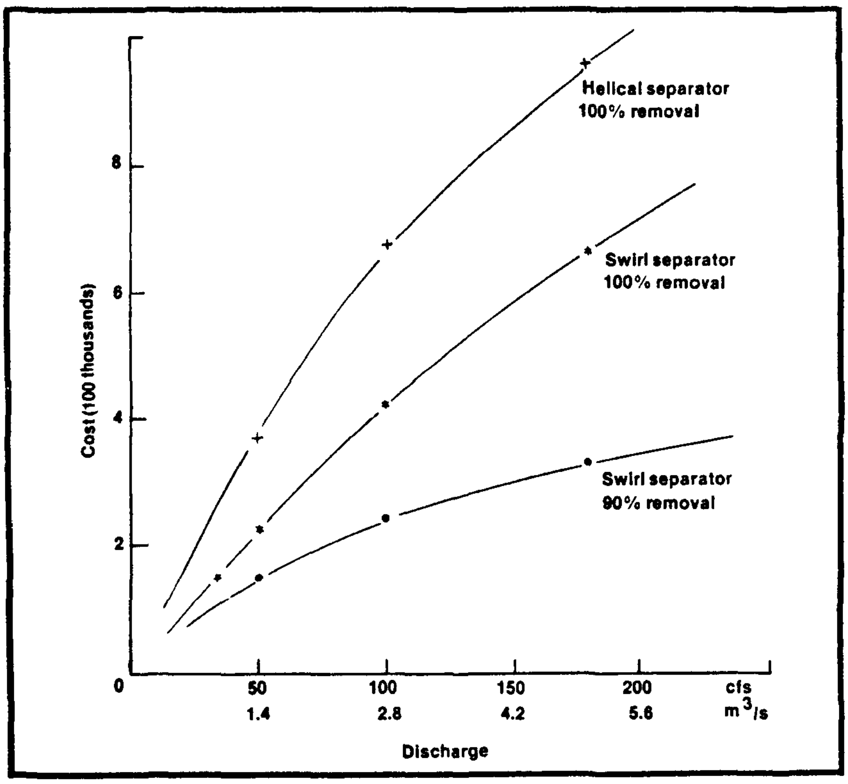


Figure 54 Estimated Construction Costs Helical Bend and Swirl Regulator/Separator

SECTION V

SWIRL DEGRITTER

The swirl degritter was originally conceived as an auxiliary combined sewer overflow treatment device to protect pumps, wet wells and downstream facilities from the large amount of grit removed by the swirl separator/regulator. Accordingly, the device was designed without moving parts for ease in unattended operation and maintenance. Conventional grit washing and removal equipment was deemed the most practical method of removing collected grit from the swirl. The unit's high efficiency and essential lack of moving parts has made its use suitable for use with stormwater, sanitary sewage, and raw potable water.

DESCRIPTION

An isometric view of the swirl degritter is shown in Figure 55. The principal difference in the configuration is the shape of the floor. A conical hopper is required to concentrate the solids for discharge. The principal features of the unit include:

A. Inlet: The inlet dimension is normally designed to allow an inlet velocity of 0.61 m (2 ft) per second. On this basis the inlet diameter becomes the controlling dimension for sizing the unit. A set of curves has been developed to express the relationship between the flow, inlet dimension, and chamber width. The flow is directed tangentially so that a "long path" pattern, maximizing solid separation in the chamber, may be developed.

B. Deflector: The covered inlet is a square extension of the inlet which is the straight line extension of the interior wall of the inlet extending to its point of tangency. Its location is important, as flow which is completing its first revolution in the chamber strikes, and is deflected inwards, forming an interior water mass which makes a second revolution in the chamber, thus creating the "long path" flow pattern.

Without the deflector, the rotational forces would quickly create a free vortex within the chamber, destroying the solid separations efficiency. The height of the deflector is the height of the inlet port, insuring a head above the elevation of the inlet, a feature which tends to rapidly direct solids down towards the floor.

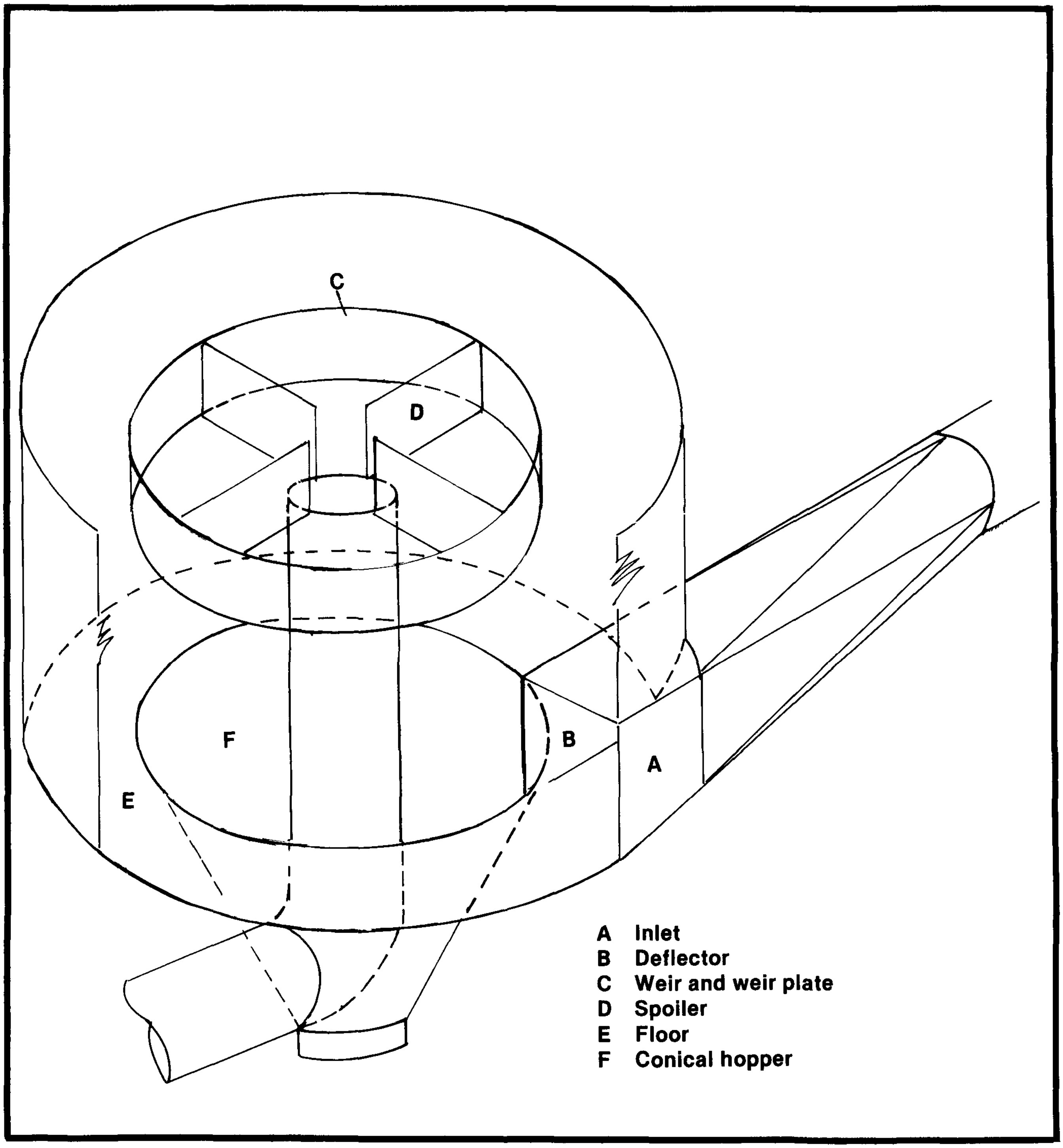


Figure 55 Isometric View, Swirl Degritter

C. Overflow Weir and Weir Plate: The diameter of the weir is a function of the diameter of the chamber, and of the inlet dimension. The weir diameter is equal to two-thirds the inlet dimension. The depth, or vertical distance from the weir to the flat floor, is normally twice the inlet dimension. The height, or rise, of the weir plate is normally 0.25 times the inlet diameter.

The weir plate connects the overflow weir to a central column, carrying the clear overflow to the interceptor and primary treatment. The horizontal leg of the downshaft should leave the chamber parallel to the inlet.

D. Spoilers: Spoilers are radial flow guides, vertically mounted on the weir plate, extending from the center shaft to the edge of the weir. They are required to break up the rotational flow of the liquid above the weir plate, thus increasing the efficiency of the weir and the downshaft.

The height of the spoilers is the same as the inlet diameter. This proportionately large size, as compared to the combined sewer overflow regulator, is required because of the possible large variations in flow which may be anticipated if the unit is used on a continuous basis.

E. Floor: The floor of the unit is level and is in effect a shelf, the width of the inlet.

F. Conical Hopper: The conical hopper is used to direct the settling grit particles to a single delivery point where they may be removed to a conveyor for washing and removal from the system.

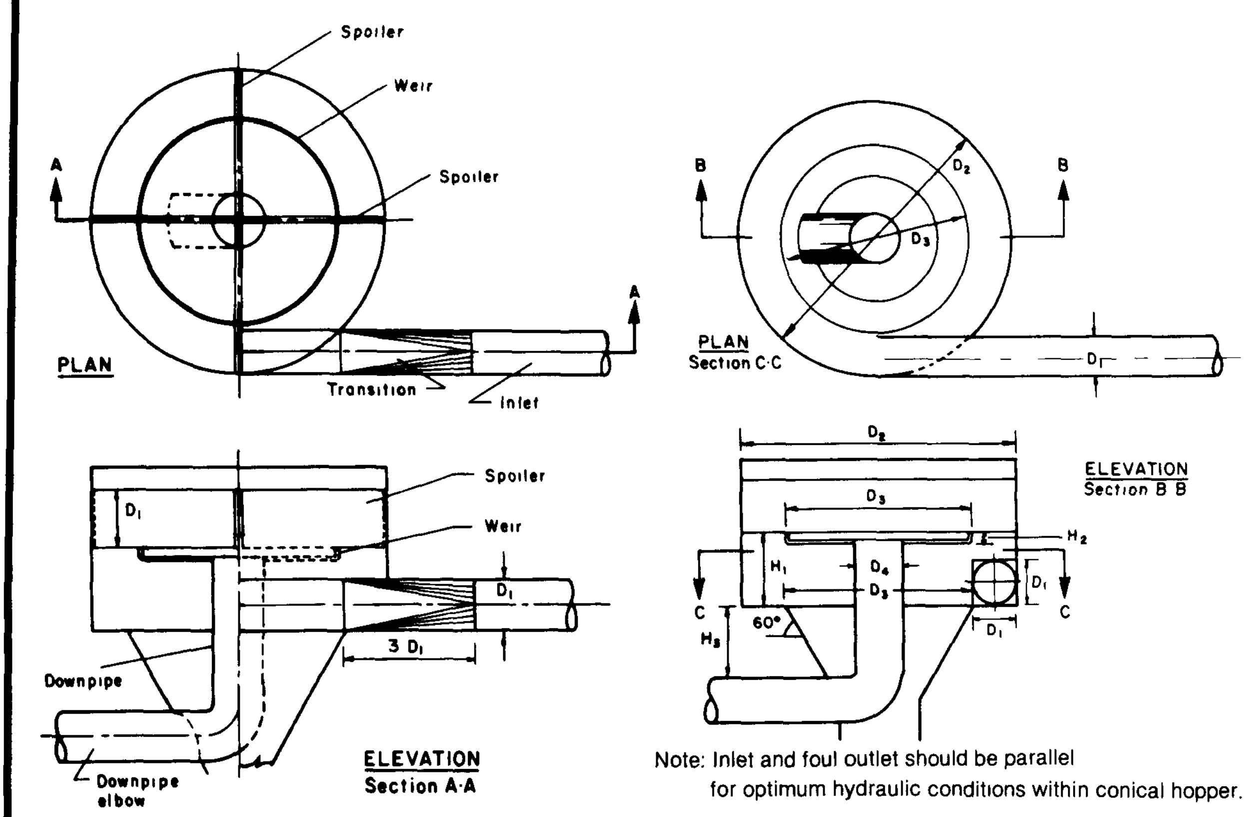
The hopper is at an angle of 60 degrees to the floor. If the angle is less than 45 degrees, particles will build up at the lip. As the angle is increased, the problem decreases to an optimum condition at 60 degrees.

The downshaft elbow must be sufficiently below the floor to prevent formation of eddy currents. This depth appears to be one inlet diameter. Structural supports for the elbow and actual pipe connections must be designed to prevent rags from being caught on a protruding bolt head, flange or strut. The downshaft should exit parallel to the inlet to assure minimum hydraulic interference for settling particles.

Figure 56, General Design Dimensions, lists the various important dimensions, which are given as a function of the inlet diameter, D_1. The ratio of D_2 to D_1 is given on Figures 57 to 59.

FACILITY FACTORS TO BE CONSIDERED

Before using the swirl concentrator as a degritter, designers should make a comparison of the various alternatives. For large flows the swirl degritter with a cone-shaped hopper may require a depth greater than the more conventional grit chambers. The presence of high groundwater or bed rock may affect cost estimates appreciably if the deeper structure is used.



D_3 = 2/3 D_2
D_4 = D_1 (larger will spoil flow outside pipe)
H_1 = 2 D_1
H_2 = 1/4 D_1
H_3 = D_1 or larger
Inlet transition = 3 D_1
D_1 and D_2 to be determined from design curves, Figs. 57-59

Figure 56 General Design Dimensions, Swirl Degritter

Another major factor is the head available and the effect of the swirl degritter on the hydraulic flow line of the plant. If a particular type of grit chamber requires the addition of pumping facilities it is doubtful if its use can be justified on an economic basis.

The maintenance of the swirl degritter should not be materially different from the maintenance of conventional grit chambers. In line with the usual practice, at least two units--one for standby--should be constructed so that the removal of grit can be continued when one unit is taken out of service if the unit is used at the wastewater treatment plant. For combined sewer overflow use, only one unit should be considered.

When used at a wastewater treatment plant the mechanical equipment should be provided with electrical devices so that the equipment can be operated either continuously, or intermittently as regulated by a time clock, or manually. It is not certain to what extent organic matter will settle out in the conical hopper during low flow periods. For this reason, it may be necessary to operate the grit washer intermittently at such times to prevent such accumulations of organic matter in the hopper. For combined sewer overflow use it would appear desirable to bypass the dry-weather flow and not use the degritter to prevent septic conditions from developing.

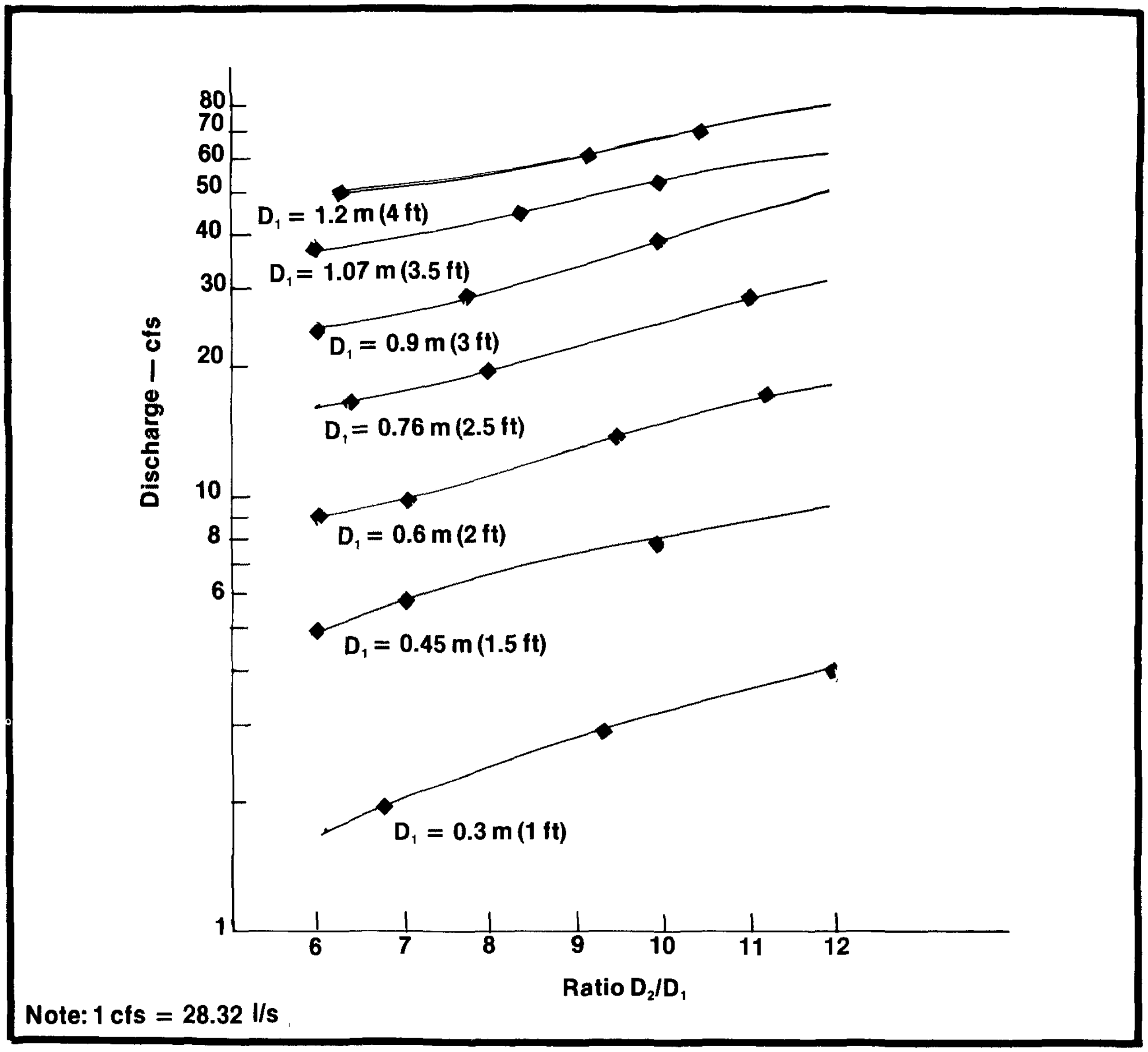


Figure 57 D_2/D_1 vs Discharge for 95% Efficiency

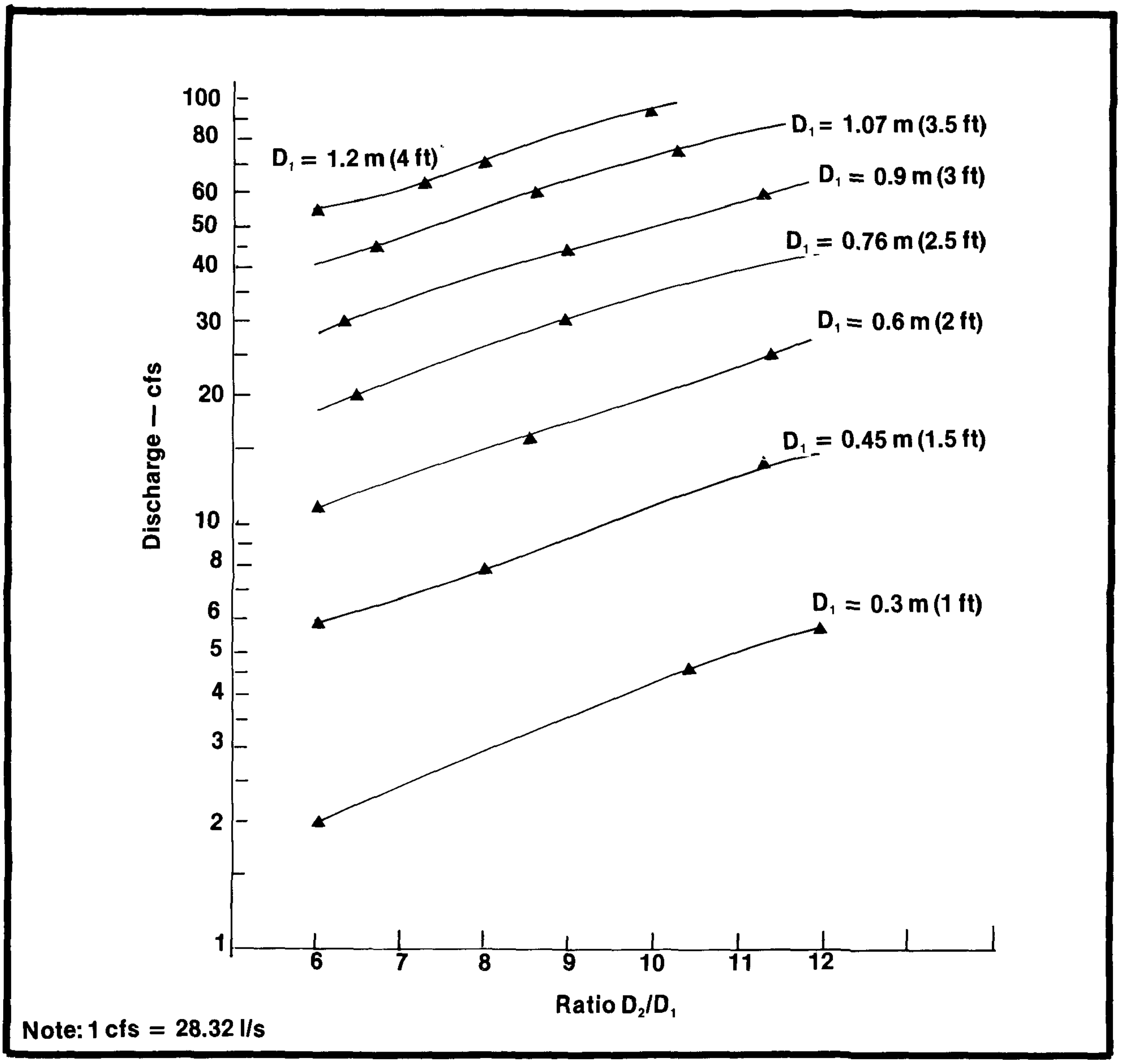


Figure 58 D_2/D_1 vs Discharge for 90% Efficiency

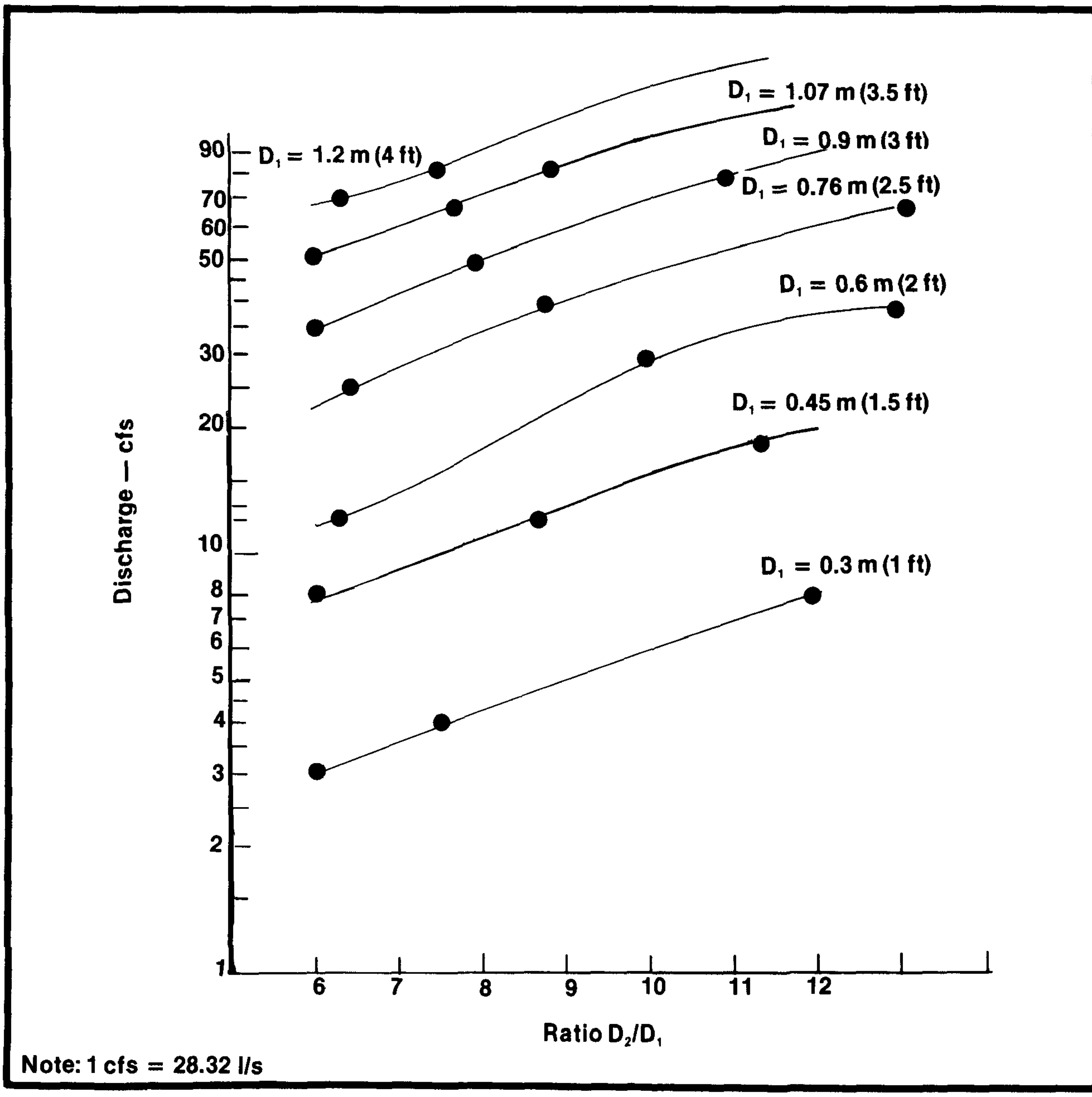


Figure 59 D_2/D_1 vs Discharge for 80% Efficiency

The area of the space above the screw should be designed so that the velocity of the wash water flowing upward will be between 0.045 to 0.075 m/sec (1.5 to 2.5 fps). Velocities lower than this may permit organic matter to settle out and velocities above this may produce an upward movement and loss of the grit. The adjustable weir of the grit washer must be set so that the required flow of wash water is obtained. If the weir is set high so that the wash water rate is lower than the design rate, the grit will contain a larger amount of organic matter. The area of the water surface upstream of the adjustable weir must be such that the surface loading of the wash water rate shall be at least 0.55 cu m/min/sq m (13.5 gpm/sq ft).

DESIGN

The following sequence is recommended for the design of the swirl degritter.

1. Select Design Discharge: The design engineer must select the design discharge appropriate to each project based on the design criteria for the project.
   One application of the swirl degritter chamber would be its use in a wastewater treatment plant. In such an application the grit chamber should be designed for the maximum design flow.

   Another application of the swirl degritter considered in connection with this study was as a grit removal device for the foul flow from a swirl regulator/separator. In that case the design flow for the grit chamber should be based on the foul flow discharge from the overflow regulator. A third application would be as a combined sewer overflow or stormwater treatment plant unit process. The swirl unit can also be used where grit is a problem prior to syphons or pumping stations within the collection system. Therefore, the designer must select the design flow based on the particular application.

2. Select the Operating Efficiency: With a discharge determined as above, 90 percent recovery is suggested as an acceptable operation. However, if there is the possibility for any future but undefined increase in the discharge, using 95 percent recovery would provide some extra capacity.

3. Find the Square Inlet Dimension, D_1: Having selected the desired recovery rate and the design discharge, the corresponding figure in the series of Figures 57 to 59 would be used. Enter the figure with the design discharge and go horizontally to the curve which most closely represents the supply sewer diameter. It might be advantageous to select a larger or smaller D_1 to coincide exactly with the supply sewer size. In the model tests, the square inlet dimension was the same as the supply sewer diameter, so these are the ideal operating conditions for this unit.

In cases where the square inlet dimension cannot conveniently be made the same as the supply sewer, a reducing or expanding transition would be necessary. If the supply sewer is concentrically aligned with the inlet, the transition should have a length of at least three times D_1 ($3D_1$). Another possibility would be to have the supply sewer discharge into an inspection manhole. Leaving the manhole would be the square inlet cross section leading into the swirl degritter. The distance from this manhole offtake to the square inlet discharge in the chamber should also be a minimum of 3 times D_1 ($3D_1$). This arrangement could be used to provide the transition in directions, levels or sizes between the supply sewer and the square inlet.

It may be noted that a smaller diameter of inlet sewer, D_1, results in a higher inlet velocity, a larger diameter, D_2, and a smaller chamber depth, H_1.

4. Find Grit Chamber Diameter, D_2: The intersection point found in (3) above defines the chamber diameter, D_2 on the abscissa scale, inasmuch as D_1 is known and the scale is the ratio D_2/D_1. In the consideration for choosing D_1 it might be a valuable aid to check the D_2 size as well. Taking a smaller D_1 means a larger D_2 is necessary; there could well be an economical or practical optimum relation between the two dimensions.

5. Determine Head Over Weir: Using Figure 60, determine head over the weir by entering the flow and proceeding to the appropriate D_2/D_1 curve.

6. Example: Assume the designer decides to remove 90 percent of the grit over an effective diameter of 0.2 mm size: The average daily sanitary sewage flow varies from 85 to 425 l/sec (3 to 15 cfs) with an inlet pipe diameter of 61 cm (2 ft).

Enter Figure 58 with 425 l/sec (15 cfs)
at: D_1 = 61 cm (2 ft), $D_2/D_1 = 8$
or: D_2 = 4.88 m (16 ft)

On Figure 60, the intersection of 0.43 cu m/sec (15 cfs) and the D_2/D_1 curve for 8 lies within the curves and the head on the weir is about 24 cm (0.78 ft).

Interpolation between curves on Figures 57 through 59 can be done without extreme care as slight changes in the ratio are not critical to the structures.

7. Find Dimensions of Complete Unit Using D_1 and D_2: Use Figure 56 to compute dimensions of all pertinent elements in the structure.

8. Find Water Level in Chamber: With the unit completely dimensioned, it would then have to be set with respect to the level of the incoming sewer.

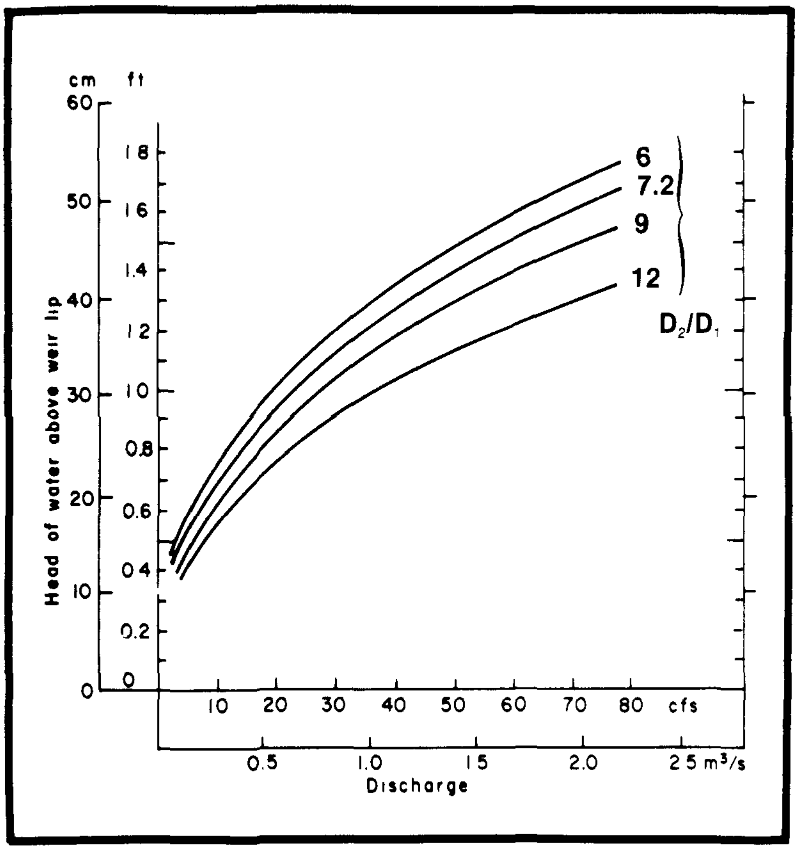


Figure 60 Approximate Stage and Discharge Curves over Weir

CONSTRUCTION COST

For comparative purposes estimates were made of construction and annual operation costs of the swirl degritter and the standard aerated grit chamber. Estimates were made for three sizes of each type for average flows of 43.8, 131.4 and 438 l/sec (1, 3, and 10 mgd). Present worth was determined for each size and type based on a 20 year period and 7-1/8 percent interest rate.

The principal diameter of the chamber, D_2 was obtained from Figure 58 for 90 Percent Recovery and $H_1/D_1 = 2$, using a ratio of D_2/D_1 of 6. The remaining dimensions were obtained from Figure 56. The derived dimensions are as shown in Table 20.

The type of unit used for estimate purposes was similar to that shown in Figure 61, with the following revisions: (1) the exterior wall of the grit separator was assumed to be of concrete with a vertical exterior face, (2) a horizontal passage through the concrete assumed to provide access for lubricating the bottom fitting of the inclined screw conveyor and (3) a manhole, 0.91 m (3 ft) sq, was provided to give access to the bottom fitting of the screw conveyor.

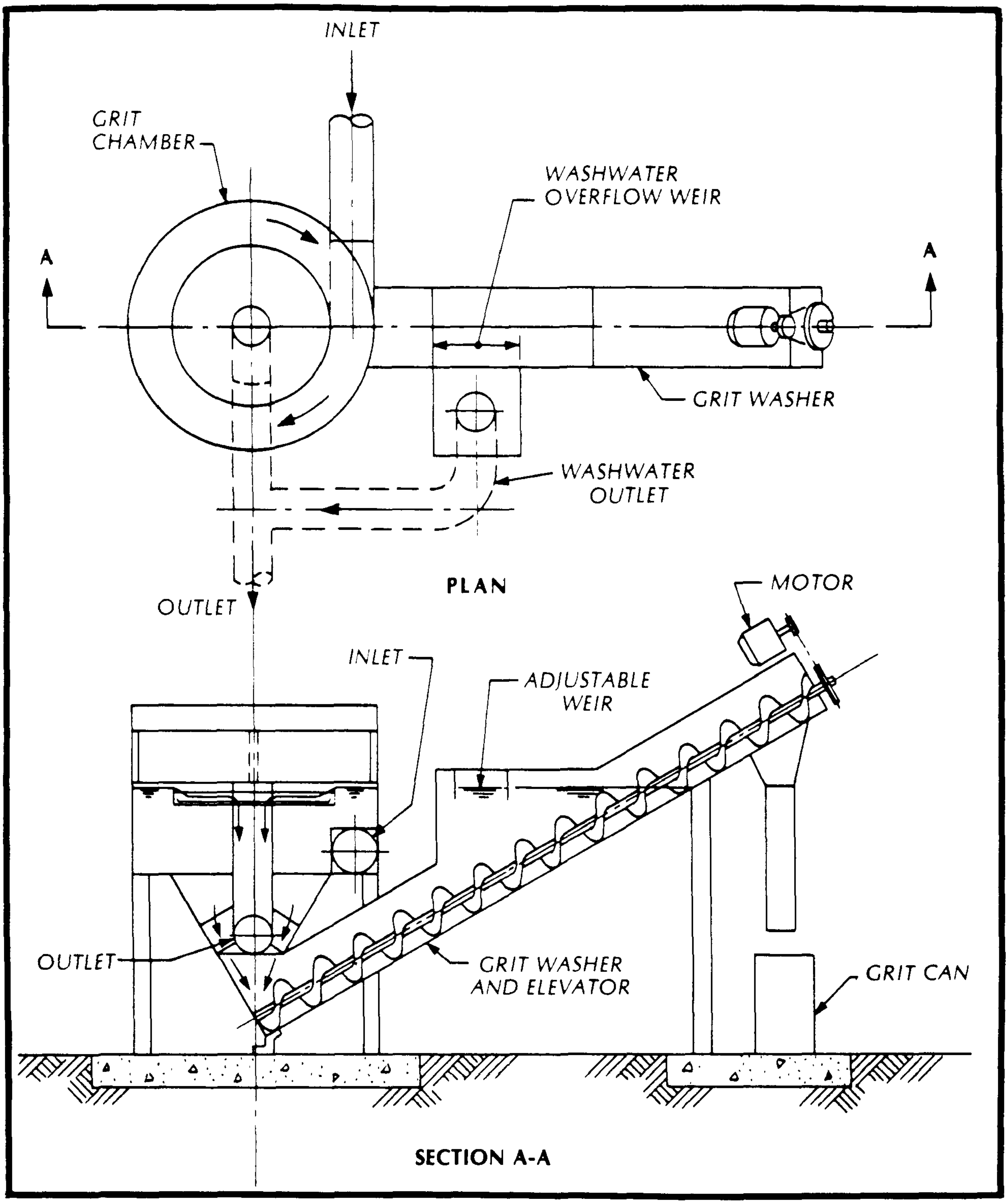


Figure 61 Grit Chamber with Inclined Screw Conveyor

Table 20
Swirl Degritter Dimensions for 3 Flowrates

Average flow	43.8 l/s (1 mgd)	131.4 l/s (3 mgd)	438 l/s (10 mgd)
D_2	2.67 m (8.75 ft)	3.90 m (12.8 ft)	5.95 m (19.5 ft)
D_1 & D_4	0.30 m (1.0 ft)	0.40 m (1.33 ft)	0.71 m (2.33 ft)
D_3	1.22 m (4.0 ft)	1.62 m (5.33 ft)	2.85 m (9.34 ft)
H_1	0.61 m (2.0 ft)	0.81 m (2.67 ft)	1.42 m (4.67 ft)
H_2	0.08 m (0.025 ft)	0.10 m (0.33 ft)	0.17 m (0.58 f)
H_3 min	0.30 m (1.0 ft)	0.40 m (1.33 ft)	0.71 m (2.0 ft)

Cost estimates of the swirl degritter were made for two purposes: (1) to indicate the probable construction cost of the facility; and (2) to compare its cost with that of a conventional aerated grit chamber.

The cost estimates are considered to be reasonable engineer's estimates. However, during periods of economic inflation, it is not unusual for contractor's bids to materially exceed engineers' estimates.

Cost Basis

The costs are based on the following:

A. Engineering News Record Construction Cost Index average for United States is 3140.

B. Unit prices as follows:

Steel Sheet Piling (for temporary use during construction)	\$129/sq m	\$12/sq ft
Excavation	\$ 24/cu m	\$18/cy
Reinforced Concrete	\$490/cu m	\$375/cy

C. Contingent and engineering costs are assumed to be 35 percent of the foregoing items.

The swirl degritter dimensions are derived in the previous section. It is assumed that the ground surface is 0.61 m (2 ft) above the crown of the inlet pipe and the top of tank is 0.30 m (1 ft) above the crown of the inlet pipe, this will provide 0.61 m (2 ft) of freeboard above the weir.

Aerated Grit Chamber

The aerated grit chamber was sized to provide a detention period of 3 minutes at the maximum rate of flow. Peak flow factors were based on Figure 4 in American Society Civil Engineers Manual No. 37 (16). The resultant dimensions are shown in Table 21.

The conventional aerated grit chamber is set to provide a freeboard 0.46 m (1.5 ft) with a top of wall 0.30 m (1 ft) above ground surface.

The following assumptions are made for both structures:

A. Excavation is all earth. The unit price includes cost for backfilling and crushed stone under the structures.

B. Temporary steel sheet piling is required to 0.61 m (2 ft) outside the exterior walls of the structures. Sheeting assumed to extend 0.61 m (2 ft) below lowest point of excavation and 0.30 m (1 ft) above the existing ground elevation.

C. Equipment costs for the aerated grit chamber include the cost of bucket elevator, screw conveyor, transverse baffle, diffuser piping, motors, and electrical work.

D. Miscellaneous costs for the aerated grit chamber include the cost of the longitudinal and effluent baffles, compressors, slide gates, baffle supports, and grating for by-pass channel.

E. Equipment costs for the swirl degritter include the cost of a grit wash screw.

F. Miscellaneous costs for the swirl degritter includes the cost of piping, skirt, weirs and plates.

Table 21
Aerated Grit Chamber Dimensions for 3 Flowrates

Average flow	**43.8 l/sec (l mgd)**	**131.4 l/sec (3 mgd)**	**438 l/sec (10 mgd)**
Peak flow factor	**3.0**	**2.5**	**2.0**
Maximum flow	**131.4 l/sec (3 mgd)**	**328.5 l/sec (7.5 mgd)**	**876 l/sec (20.0 mgd)**
Required volume	**23.6 cu m (835 cf)**	**59.2 cu m (2,090 cf)**	**157.9 cu m (5.560 cf)**
Selected depth	**2.44 m (8.0 ft)**	**3.05 m (10.0 ft)**	**3.66 m (12.0)**
Selected width	**2.29 m (7.5 ft)**	**3.05 m (10.0 ft)**	**4.27 m (14.0 ft)**
Selected length	**4.27 m (14.0 ft)**	**6.41 m (21.0 ft)**	**10.06 m (33.0 ft)**
Selected volume	**23.65 cu m (835 cf)**	**59.08 cu m (2,085 cf)**	**157.09 cu m (5.544 cf)**

Cost of Swirl Degritter

The estimated construction cost of a swirl degritter with a capacity of 43.8 l/sec (1 mgd) is $72,980, for 131.4 l/sec (3 mgd), $84,090, and for

438 l/sec (10 mgd), $97,830. The breakdown of these costs is shown in Table 22.

Cost of Aerated Grit Chamber

The estimated construction costs of a conventional aerated grit chamber with a capacity of 43.8 l/sec (1 mgd) is $87,240, for 131.4 l/sec (3 mgd), $112,530, and for a 438 l/sec (10 mgd), $155,650, as shown in Table 23.

OPERATION AND MAINTENANCE COSTS

The estimated operation and maintenance costs for the swirl degritter and the aerated grit chamber for capacities of 43.8 l/sec (1 mgd), 131.4 l/sec (3 mgd) and 438 l/sec (10 mgd) are shown in Table 24. For units with capacity of 43.8 l/sec (1 mgd) the annual expenses are estimated at $7,020 for the aerated chamber and $6,355 for the swirl degritter. For capacity of 131.4 l/sec (3 mgd) the annual expenses are $11,720 for the aerated chamber and $10,450 for the swirl degritter. For capacity of 438 l/sec (10 mgd) the annual expenses are $22,280 for the aerated chamber and $18,630 for the swirl degritter.

The operator labor is assumed to be 1.5 hours per day for the 131.4 l/sec (3 mgd) unit. This assumes 1 hour for operation of the equipment and 0.5 hours for disposal of the grit. This is based on the actual experience at a unit with the capacity where the daily operation ranges from 0.5 to 1 hours with occasional periods of 1.5 hours following storm periods.

The labor rate used of $10.00 per hour is intended to include the actual labor cost plus all benefits but excludes administration and general expenses of the overall plant.

Based on the results shown in Table 24, the annual operation costs of the aerated grit chamber will exceed the annual costs of the swirl degritter by about 10 percent for each size unit.

Present Worth

The present worth of the grit removal units is shown in Table 25. The present worth is based on a life of 20 years and an interest rate of 7-1/8 percent. Hence the present worth of the operation and maintenance costs for a 20 year period is 10.49 times the annual cost.

For the unit with capacity of 43.8 l/sec (1 mgd) the present worth of the aerated chamber is $160,940 and the swirl degritter is $139,980. Thus the present worth of the aerated chamber is 15 percent greater than that of the swirl degritter.

For the unit with capacity of 131.4 l/sec (3 mgd) the present worth of the aerated chamber is $235,530 compared to $194,090 for the swirl degritter. Thus the present worth of the aerated chamber is 20 percent greater than that of the swirl degritter.

Table 22
Construction Cost of Swirl Degritter

Item	Quantity	Amount
Capacity 43.8 l/s (1.0 mgd)		
Sheet piling	72 sq m (780 sq ft)	$ 9,360
Excavation	115 cu m (150 cy)	2,700
Reinforced concrete	12 cu m (16 cy)	6,000
Equipment	Job	24,700
Miscellaneous and bypass	job	11,300
Subtotal		54,060
Contingent and engineering costs	35%	18,920
Total		$72,980
Capacity 131.4 l/s (3.0 mgd)		
Sheet piling	89 sq m (960 sq ft)	$11,520
Excavation	142 cu m 185 cy	3,330
Reinforced concrete	15 cu m (20 cy)	7,500
Equipment	Job	27,380
Miscellaneous and bypass	Job	12,560
Subtotal		62,290
Contingent and engineering costs	35%	21,800
Total		$84,090
Capacity 438 l/s (10.0 mgd)		
Sheet piling	102 sq m (1,100 sq ft)	$13,200
Excavation	184 cu m (240 cy)	4,320
Reinforced concrete	20 cu m (26 cy)	9,750
Equipment	Job	31,400
Miscellaneous and bypass	Job	13,800
Subtotal		72,470
Contingent and engineering costs	35%	25,360
Total		$97,830

Table 23
Construction Cost of Aerated Grit Chamber

Item	Quantity	Amount
Capacity 43.8 l/s (1.0 mgd)		
Sheet piling	67.5 sq m (725 sq ft)	$ 8,700
Excavation	78 cu m (101 cy)	1,820
Reinforced concrete	11 cu m (14 cy)	5,250
Equipment	Job	38,680
Miscellaneous	Job	10,170
Subtotal		64,620
Contingent and engineering costs	35%	22,620
Total		$ 87,240
Capacity 131.4 l/s (3.0 mgd)		
Sheet piling	98 sq m (1066 sq ft)	$ 12,800
Excavation	99 cu m (127 cy)	2,290
Reinforced concrete	21.2 cu m (27 cy)	10,120
Equipment	Job	45,720
Miscellaneous	Job	12,430
Subtotal		83,360
Contingent and engineering costs	35%	29,170
Total		$112,530
Capacity 438 l/s (10.0 mgd)		
Sheet piling	157 sq m (1,710 sq ft)	$ 20,500
Excavation	276 cu m (361 cu m)	6,500
Reinforced concrete	34.2 cu m (44.7 cy)	16,800
Equipment	Job	56,500
Miscellaneous	Job	15,000
Subtotal		115,300
Contingent and engineering costs	35%	40,350
Total		$155,650

Table 24
Operation and Maintenance Costs for Grit Removal

Capacity 43.8 l/s (1.0 mgd)	Aerated Chamber	Swirl Separator
Labor		
Operation 1.5 hr/day at $10/hr	$ 5,480	$ 5,480
Maintenance 0.2 hr/day at $10/hr	730	730
Materials and supplies	250	130
Power		
1 Compressor at 1 hp, 24 hr/day x $0.06/kwh	530	—
1 Screw conveyor at ½ hp, 1 hr/day x $0.06/kwh	15	15
1 Bucket conveyor at ½ hp, 1 hr/day x $0.06/kwh	15	—
Total Annual Costs	$ 7,020	$ 6,355
Capacity 131.4 l/s (3.0 mgd)		
Labor		
Operation 2.5 hr/day at $10/hr	$ 9,130	$ 9,130
Maintenance 0.3 hr/day at $10/hr	1,100	1,100
Materials and supplies	380	190
Power		
1 Compressor at 2 hr, 24 hr/day x $0.06/kwh	1,050	—
1 Screw conveyor at ½ hp 2 hr/day x $0.06/kwh	30	30
1 Bucket conveyor at ½ hp, 2 hr/day x $0.06/kwh	30	—
Total Annual Costs	$11,720	$10,450
Capacity 438 l/s (10.0 mgd)		
Labor		
Operation 4.5 hr/day at $10/hr	$16,430	$16,430
Maintenance 0.5 hr/day at $10/hr	1,830	1,830
Materials and supplies	750	310
Power		
1 Compressor at 6 hr, 24 hr/day x $0.06/kwh	3,150	—
1 Screw conveyor at ½ hr, 4 hr/day x $0.06 kwh	60	60
1 Bucket conveyor at ½ hp, 4 hr/day x $0.06 kwh	60	—
Total Annual Costs	$22,280	$18,630

Table 25
Present Worth Grit Removal Units

Capacity 43.8 l/s (1.0 mgd)	Aerated Chamber	Swirl Degritter
Construction cost	$ 87,240	$ 72,980
Operation and maintenance cost	73,700	67,000
Cost Total Present Worth	$160,940	$139,980
Capacity 131.4 l/s (3.0 mgd)		
Construction cost	$112,530	$ 84,090
Operation and maintenance cost	123,000	110,000
Total Present Worth	$235,530	$194,090
Capacity 438 l/s (10.0 mgd)		
Construction cost	$155,650	$ 97,830
Operation and maintenance cost	233,300	195,000
Total Present Worth	$388,950	$292,830

For the 438 l/sec (10 mgd) unit, the present worth of the aerated chamber is $388,950 compared to $292,830 for the swirl degritter, or 33 percent greater.

PROTOTYPE INSTALLATIONS

Three units have been constructed. Results of the extensive testing of the unit at the Metropolitan Denver Sewage Disposal District No. 1 have been published. (5) The unit was tested using sanitary sewage and sewage spiked with fine sand. A swirl degritter has been constructed at Lancaster, Pennsylvania, to remove grit from the tank underflow of a swirl regulator/separator to protect downstream pumping facilities. This unit is identical in construction to the Denver unit. Operating results are not presently available.

A third unit has been constructed in the City of Tamworth, New South Wales, Australia. The unit has been designed to protect raw water treatment equipment. Again, operating results are not yet available.

Details of the three installations will be reviewed.

1. Metropolitan Denver Sewage Disposal District No. 1, Denver, Colorado. The test program was designed to determine the grit removal efficiency of the test system and to compare the results with the removal performance of the plant's conventional aerated grit chamber (AGC). Figure 62 shows the layout for the test program and Figure 63 is a plan of the test unit while Figure 64 is a photograph of the test installation.

   The 43.8 l/sec (1 mgd) swirl degritter was constructed in 1974 at a cost of $4,500 exclusive of pumps, valves, and grit washer elements which the district had available. The cost of a conventional AGC for the same flow was estimated to be $57,000.

   Extended tests were made at flows of 43.8, 87.6 and 131.4 l/sec (1, 2, and 3 mgd). Grit ash was used as a basis of efficiency comparison. Grit ash was used as it represents the inorganic, heavier material that a grit chamber is designed to remove. The testing program found that the recovery of grit less than 0.2 mm was 10 percent or less. Therefore, tests were run with the flow spiked with fine blasting sand (0.25 mm diameter) at concentrations ranging from 288 gm/cu m (2,400 lb/MG) at flows of 21.9 l/sec (0.5 mgd) to 48 gm/cu m (400 lb/MG) at flows of 131.4 l/sec (3 mgd).

   It was found that the percent dry grit removal in the AGC for raw sanitary sewage was consistantly higher (77.3 percent) than accumulated in the swirl degritter (66.4 percent), the AGC retained an undesirably higher percentage of organic particles (volatile solids) than the swirl (12-30 percent for the AGC as compared to 3-10 percent for the swirl). The swirl degritter when tested under conditions which might be encountered in removing grit from a combined sewer overflow and overflow concentrate had a removal efficiency of 50 to 87 percent.

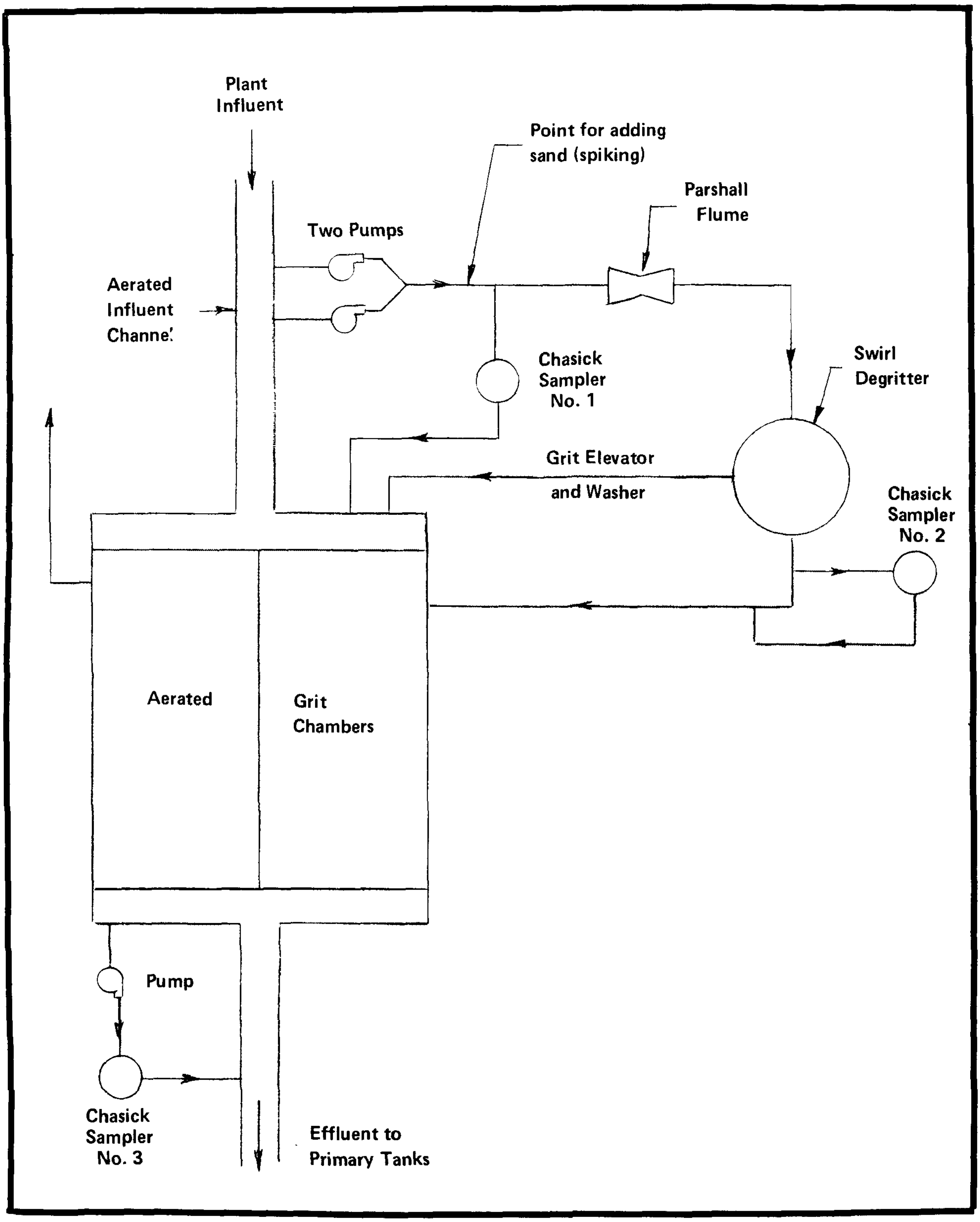


Figure 62 Layout for Denver Tests

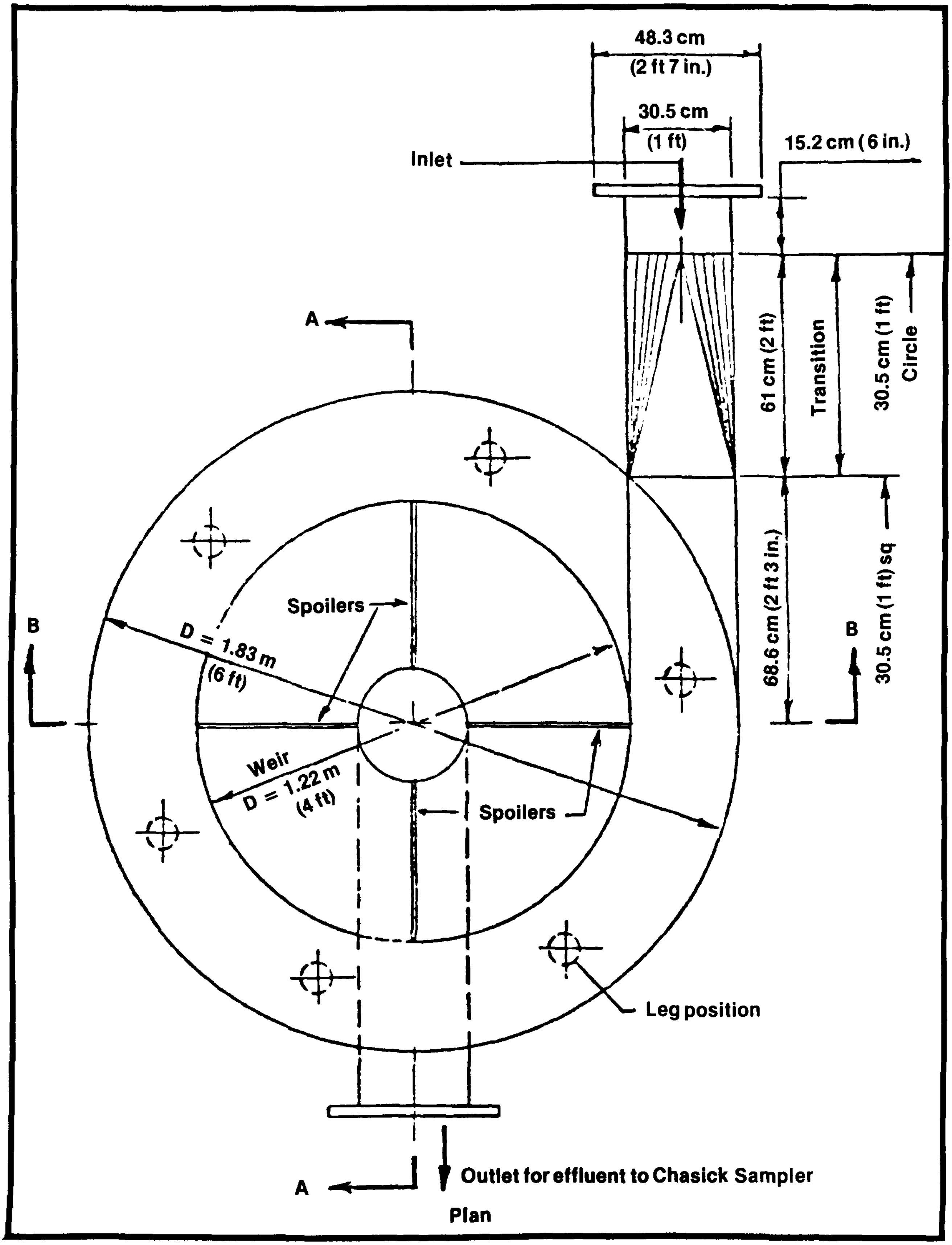


Figure 63 Plan of Swirl Degritter, Denver

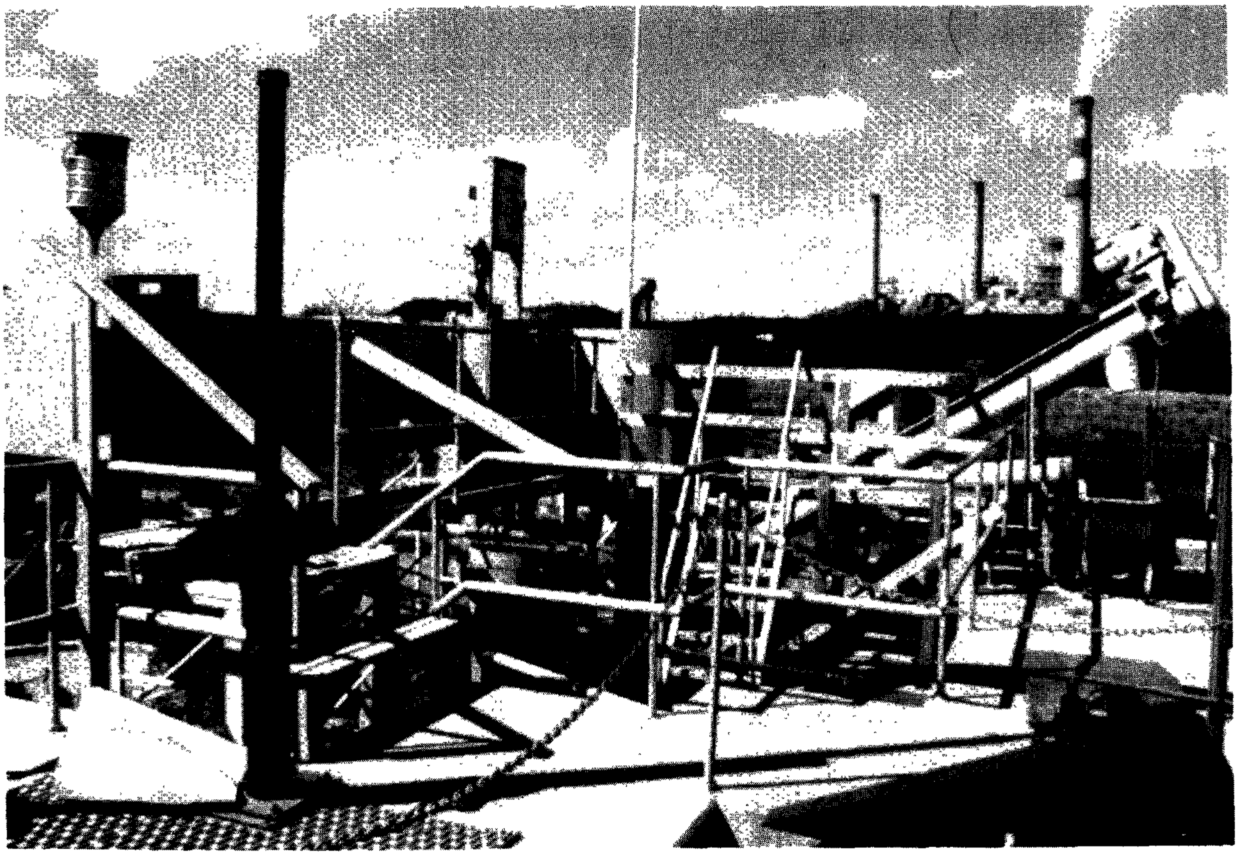


Figure 64 Photograph, Denver Test Facility

The swirl degritter remained effective at flows twice the design flow with a marked decrease in performance at three times design flow, which is similar to other types of grit removal facilities.

The residual concentration of grit in the effluent is a major factor in evaluating performance. Limited studies of wastewater treatment plants have shown concentration of grit removal in all subsequent treatment processes.

Thus there is some, as yet undefined, concentration in the effluent which can be tolerated. The results of the Denver tests tend to point to a relationship whereby with an increase in grit concentration in the influent an increase in the efficiency ratio may also be observed.

2. Lancaster, Pennsylvania. The swirl degritter, of the same size as Denver, was constructed to remove grit from the foul underflow from the swirl regulator/separator. Modeling studies prior to construction indicated for the six storms studied a maximum suspended solids concentration of 380 kg/cu m (172 lb/MG) could be expected. The swirl degritter was built to protect a wet well and pump needed to discharge the foul flow into the interceptor sewer. Dry weather flow is diverted directly to the wet well and is not treated.

   The sampling program at Lancaster has been delayed and detailed results are not yet available. One important operating factor that is readily apparent is the need to provide a means to easily dewater the facility when it is not in use. An intermittant use, such as Lancaster, will have many long periods when there is no flow and the contents of the chamber will rapidly become septic.

3. Tamworth, New South Wales, Australia. The Department of Public Works of New South Wales has recently constructed a 5 m (16.4 ft) diameter chamber to remove grit from a raw river water supply subject to intermittant high solid loads from a normally dry water course. Again details of the operating performance are not now available.

Section VI

SWIRL PRIMARY SEPARATOR

DESCRIPTION

The swirl primary separator was designed to incorporate the use of secondary motions to hasten the settling of suspended solids to achieve primary treatment. Figure 65 is an isometric view of the unit.

It was found that removal efficiency sufficient to meet the standard of primary sewage treatment could only be obtained at flows of less than 6.5 l/sec (0.15 mgd). Without chemical additives the swirl was not found useful for removal of more than 50 percent suspended solids due to the size and cost of the required units. Thus, application of the swirl primary unit with the design given is limited. The constraints of no moving parts and minimum hydraulic head loss severely limited the capability of the present design.

The key components as shown on Figure 65 include:

A. Inlet: tangential access to the chamber.

B. Baffle: acts to force flow in outside area of the chamber down below the incoming flow and traps floatables.

C. Skirt: a circular baffle which separates the outer chamber where the flow becomes organized from the central area and where essentially quiescent settling takes place. The distance from the bottom of the skirt to the chamber wall has a major effect on removal efficiency.

D. Weirs: conventionally-designed notched weirs to allow discharge of treated effluent.

E. Clear Effluent Outlet: draw off of clear effluent for discharge to additional treatment or to receiving waters.

F. Sludge Baffle: a baffle at the bottom of the sludge collecting cone to assist draw off of concentrated sludge.

G. Sludge Discharge: a valve-controlled line to allow periodic draw off of settled sludge.

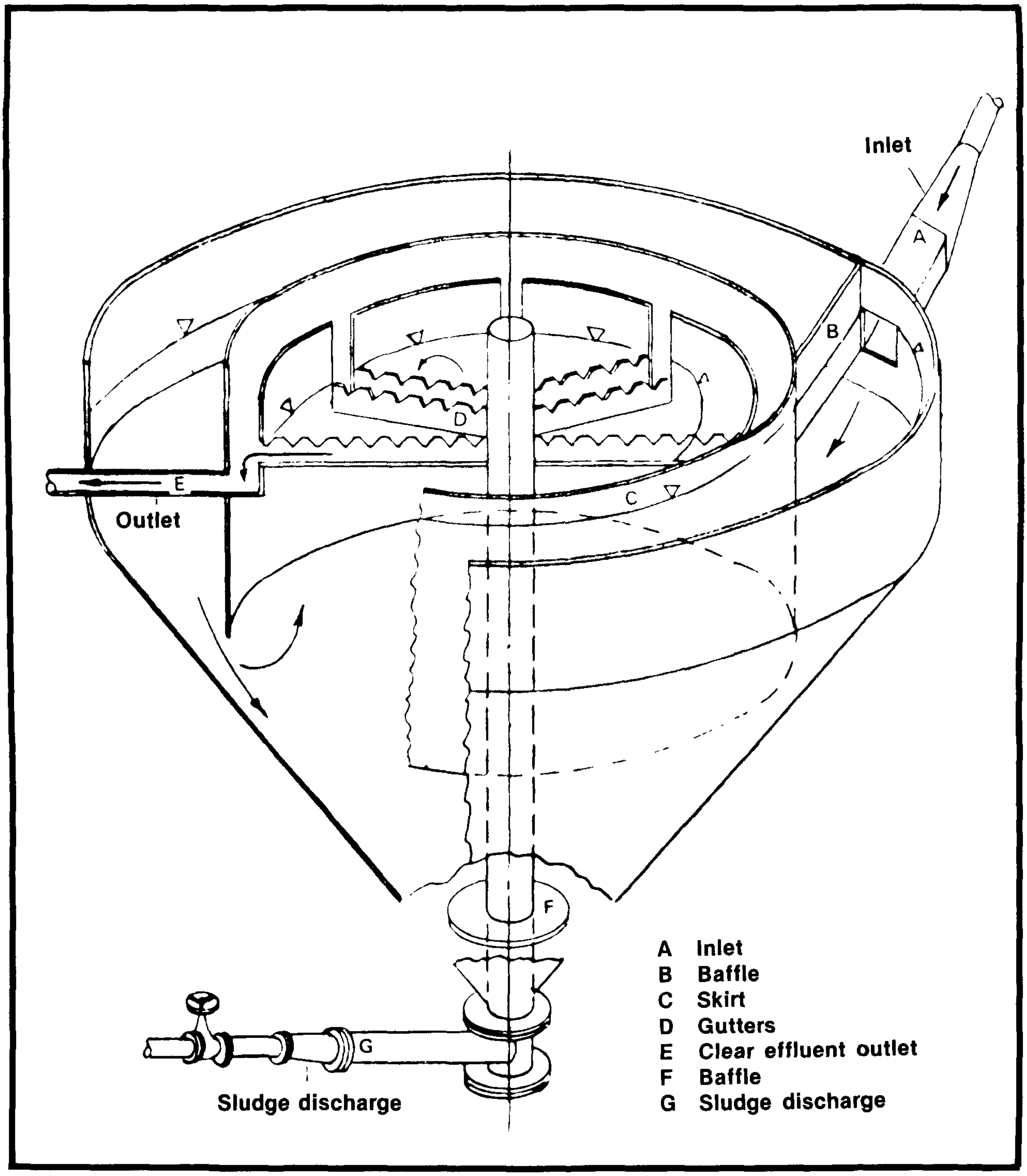


Figure 65 Isometric: Swirl Primary Separator

DESIGN BASIS

Conventional primary sedimentation tanks are generally designed on the basis of overflow rate and, to a lesser extent, on detention time. The term "overflow rate" or "surface settling velocity" is the unit volume of flow per unit of time divided by the unit of tank surface area. In U.S. customary units this is expressed as gallons per day square foot (gpd/sq ft) and in metric units may be expressed as cubic meters per day per square meter (cu m/day/sq m). The American Society of Civil Engineers Manual of Engineering Practice Number 36 (17) lists data on various primary settling tanks which indicate removal of suspended solids ranging from 20 to 80 percent. Figure 6 of that publication indicates the relation between removal of suspended solids and overflow rate. Many tanks fall in the range of 60 to 70 percent removal of suspended solids. If we accept 60 percent removal of suspended solids as a desirable objective then Figure 6 indicates the necessary overflow rate is 36.67 cu m/day/sq m (900 gpd/sq ft). The ASCE manual's curve in this range of suspended solids removal has been verified by recent analyses of field data by Smith. (18) Detention time is no longer considered as the only factor in design of primary settling tanks. However, the use of tanks with liquid depths of 2.13 to 3.66 m (7 to 12 ft) combined with accepted overflow rates will result in nominal detention times of 1 to 2 hours. For instance, the use of a 3.05 m (10 ft) liquid depth with an overflow rate of 36.67 cu m/day/sq m (900 gpd/sq ft) will result in a detention time of 2 hours.

The equation developed by Smith (18) from the analyses of field data can be used to estimate the removal efficiency (percent) of suspended solids, η as a function of overflow rate (OVFRA) in gpd/sq ft as follows:

$$\eta = 0.82e^{-\mathrm{OVFRA}/2,780}$$

For OVFRA = 36.67 cu m/day/sq m (900 gpd/sq ft) = 54.3 percent. This value is in reasonable agreement with the 60 percent removal estimated by the use of the ASCE figure for a 36.67 cu m/day/sq m (900 gpd/sq ft) overflow rate.

The design of the swirl separator is based neither on overflow rate nor detention time, but on the results of model tests. However, these two parameters are useful in comparing the size of the swirl separator with a conventional tank.

DESIGN PROCEDURE

Figures 66, 67, and 68 are used for design.

As indicated, the swirl separator cannot economically achieve conventional suspended solids removal of 60 percent. Therefore, the following design example is based upon 45 percent suspended solids removal, which is near the upper level of its efficiency for sanitary flow. Settling characteristics of combined sewer overflow solids are usually better than for sanitary sewage.

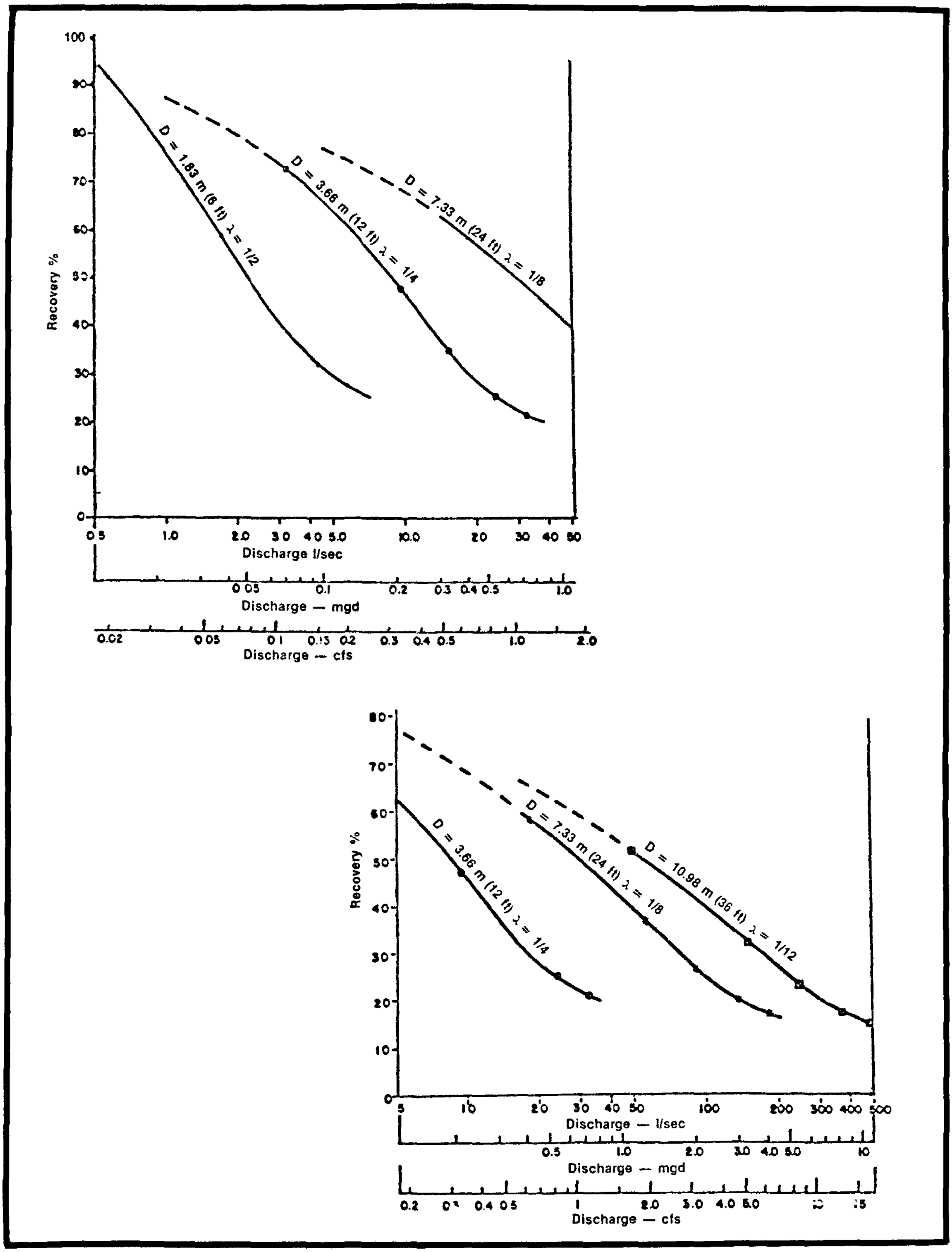


Figure 66 Predicted Prototype Solids Removal Efficiency for Sanitary Sewage

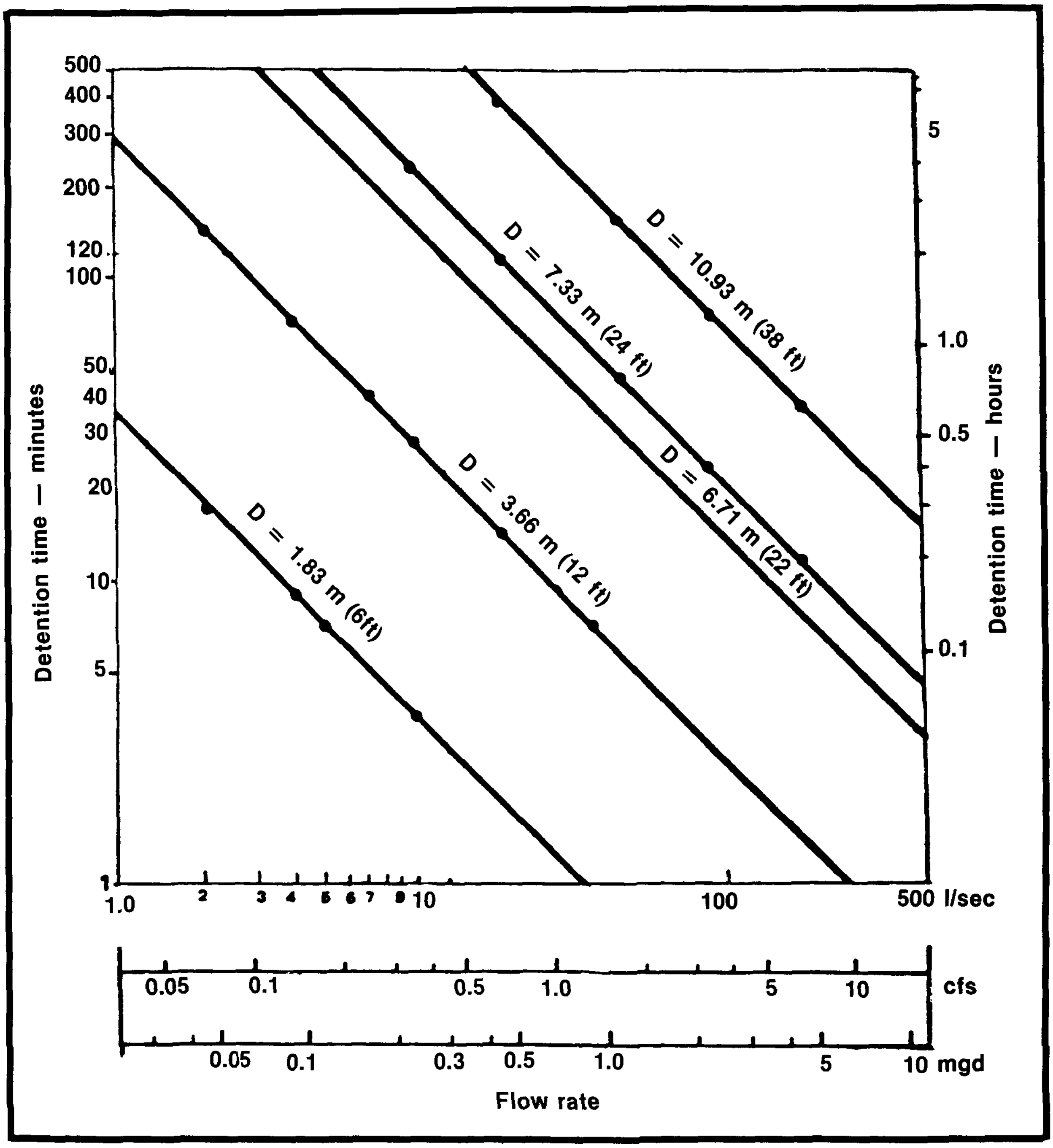


Figure 67 Detention Times

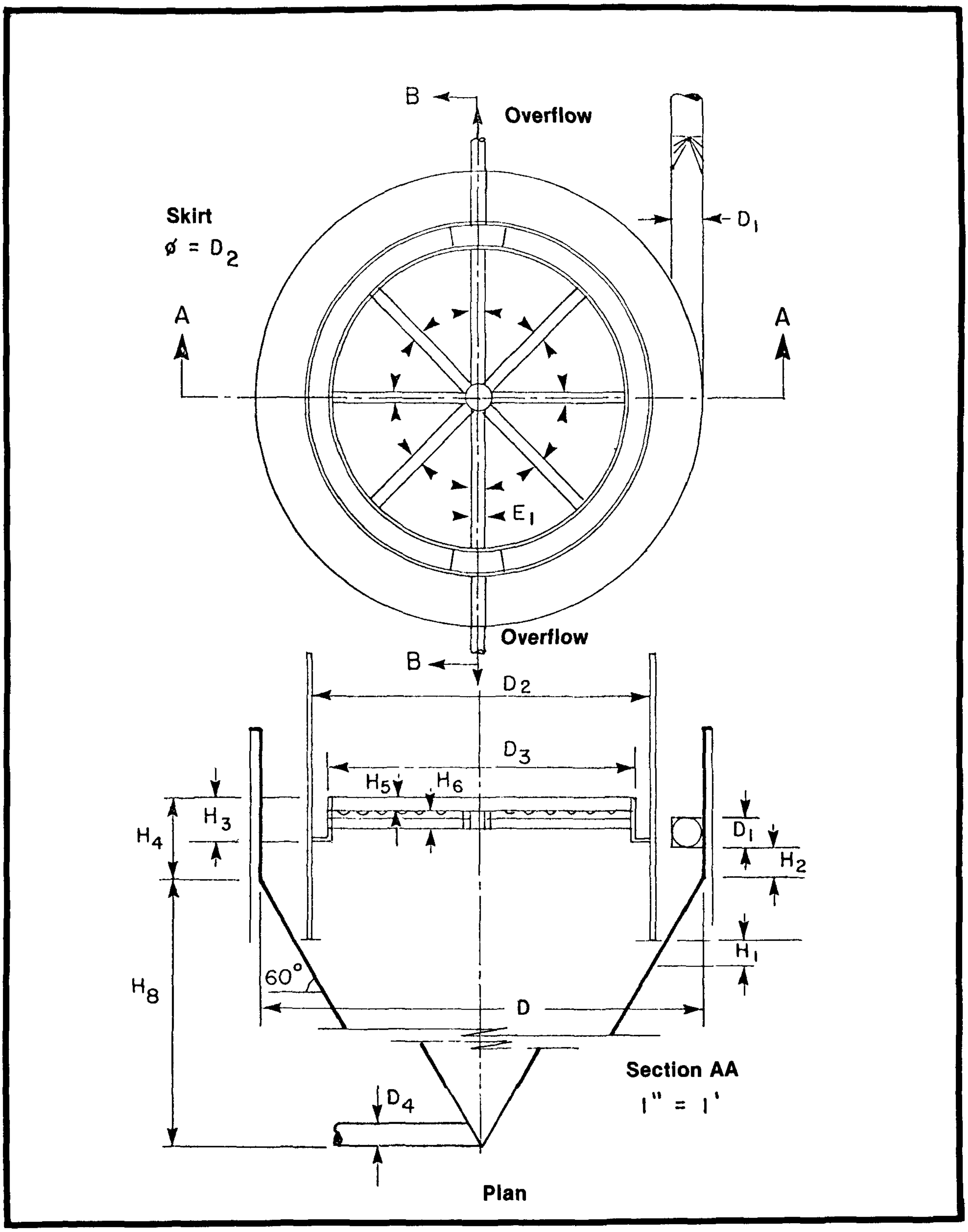
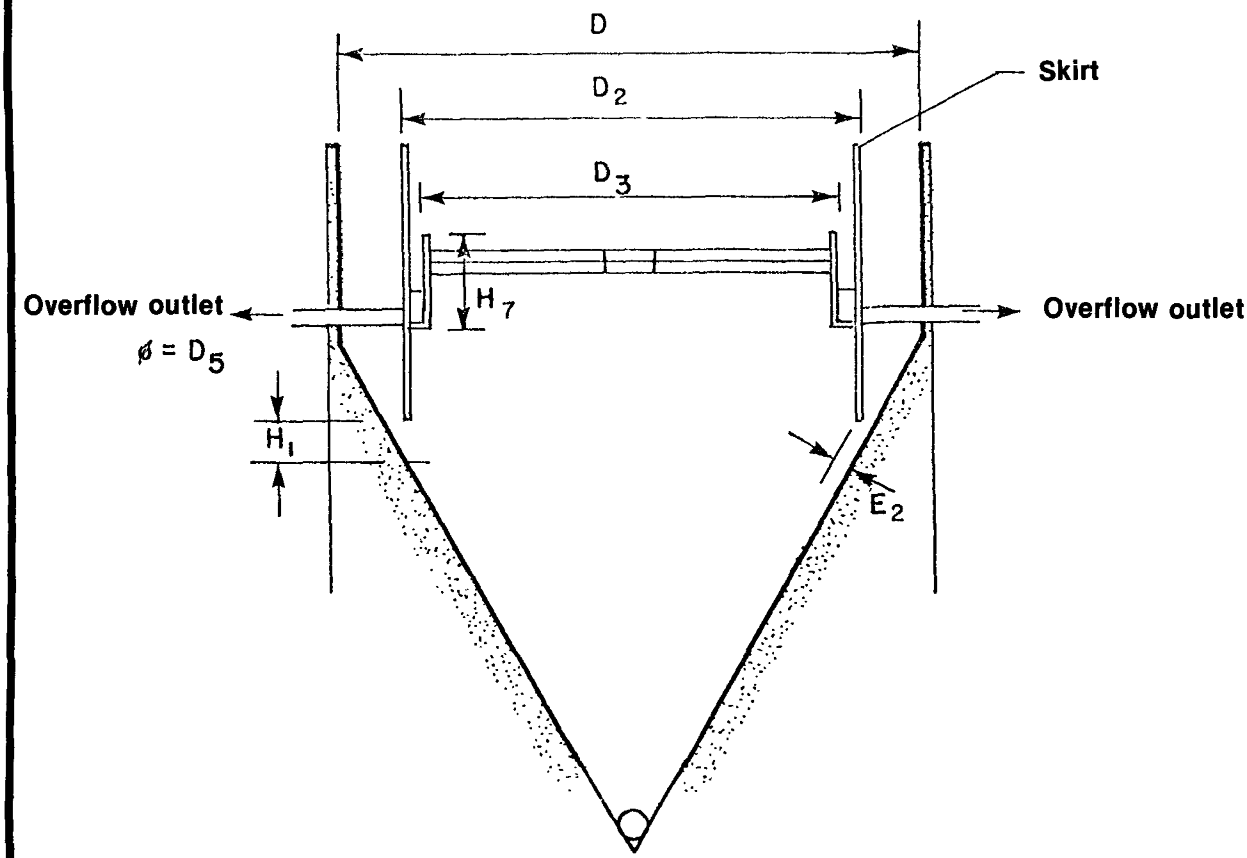


Figure 68a General Design Dimensions, Swirl Primary Separator



Elevation

D = Diameter of chamber (from Fig. 67)
D_1 = 0.066 D Inlet
D_2 = 0.67 D Skirt
D_4 = 0.58 D Gutter
D_5 = 0.056 D Sludge draw off
D_6 = 0.042 D Outlet
E_1 = 0.028 D Weir gutter width
E_2 = 0.028 D Slot width
H_1 = 0.056 D Slot height
H_2 = 0.07 D Invert elevation
H_3 = 0.125 D Circular gutter height
H_4 = 0.2 D Circular gutter top elevation
H_5 = 0.04 D Gutter top above weir lip
H_6 = 0.04 D Weir gutter depth
H_7 = 0.19 D Gutter depth at outlet
H_8 = 0.8 D Cone height

Figure 68b General Design Dimensions, Swirl Primary Separator

Normal practice is to provide a minimum of two plant units of each type in a plant. Thus the initial construction phase would include at least two primary separators.

From the tests which were conducted, the low efficiency of solids removal and long detention times make large units impractical with the present design. Table 26 gives the flow and detention times for several size units as taken from Figures 66 and 67. The table indicates that the swirl separator has less detention time than conventional settling tanks over a small range of flows. At a diameter of 5.5 m (18 ft) the detention time necessary to achieve 40 to 50 percent suspended solids removal is approximately that of conventional units.

From Table 26 it is obvious that if the detention time is to be less than that of a conventional unit, the diameter will be less than 5.5 m (18 ft). Thus, for 40 percent suspended solids removal, the maximum flow would be 10 l/sec (0.22 mgd). Since in conventional practice two tanks are used, the maximum plant design capacity would be 20 l/sec (0.44 mgd) or less.

The design of a swirl primary separator follows:

A. Plant design average daily flow is 15 l/sec (0.34 mgd)

B. Removal efficiency of suspended solids desired is 45 percent

C. Use two swirl primary separators. Design flowrate per unit is 7.5 l/sec (17 mgd). Peak flowrate is 11.2 l/sec (0.26 mgd).

D. Enter Figure 66 with design flowrate. For 45 percent efficiency, select η = 3.7 m (12 ft). Surface area is 10.5 sq m (113 sq ft). Overflow rate is 61,295 l/day/sq m (1,505 gd/sq ft).

E. Enter Figure 67 with design flowrate of 7.5 l/sec (0.17 mgd) and D of 3.7 m (12 ft). Detention time is 37 minutes.

Note: For conventional settling units, the detention time would be 51 to 63 minutes.

F. Enter Figure 66 with peak flow of 11.2 l/sec (0.26 mgd) and D of 3.7 m (12 ft). Read recovery is 38 percent.

G. Enter Figure 67 with peak flow of 11.2 l/sec (0.26 mgd) and D of 3.7 m (12 ft). Read detention time is 25 minutes.

H. Determine dimensions of structure from Figure 68, as follows:

D = 3.7 m (12 ft) inside diameter of tank

D_1 = 0.24 m (0.8 ft) inlet (side of square)

D_2 = 2.4 m (8 ft) skirt diameter

D_4 = 2.1 m (7 ft) gutter diameter

From the values given for D_2 and D_4 the circular gutter width is 0.3 m (1 ft). D_4 does not appear to be a critical dimension insofar as the tank

Table 26
Comparison of Diameter, Detention Time, and Suspended Solids Removal for Swirl Primary Separator and Detention Time for Conventional Settling for Various Overflow Rates

		Swirl % SS Removal											
		30			40			50			60		
Diameter		Flow		Detention Time	Flow		Detention Time	Flow		Detention Time	Flow		Detention Time
m	ft	l/sec	mgd	min	l/sec	mgd	min	l/sec	mgd	min	l/sec	mgd	min
1.8	6	4.5	0.1	8	2.8	0.06	13	2	0.05	18	1.6	0.04	24
3.6	12	15	0.34	19	9.8	0.22	30	6.5	0.15	45			
5.5	18	27	0.60	30	15	0.33	54	10	0.22	75			
6.1	20	28	0.62	35									

CONVENTIONAL SETTLING TANKS
%SS Removal

% SS Removal	Overflow Rate l/day/m^2	Overflow Rate gal/day/ft^2	Detention Time (min) 3.05m (10 ft) depth or over
60	36,653	900	120
50	57,017	1,400	77
40	51,543	2,000	54
30	114,034	2,800	38

Overflow Rate Comparison for Swirl Separator

	Flow l/s	Flow mgd	Overflow Rate l/day/m^2	Overflow Rate gal/day/ft^2
Diameter 1.8 m (6 ft)				
	4.5	0.1	144,170	3,540
	2.8	0.06	86,340	2,120
	2	0.05	72,085	1,770
	1.6	0.04	57,625	1,415
Diameter 3.6 m (12 ft)				
	15	0.34	122,585	3,010
	9.8	0.22	79,415	1,950
	6.5	0.15	54,165	1,330
Diameter 5.5 m (18 ft)				
	27	0.6	96,110	2,360
	15	0.33	52,945	1,300
	10	0.22	35,230	865
Diameter 6.1 m (20 ft)				
	28	0.62	80,435	1,975

performance is concerned and therefore we assume the gutter width could be changed if greater width is necessary to carry off the weir discharge.

D_5 is not a critical dimension. Suggest D_5 = 0.2 m (0.67 ft).

D_6 is not a critical dimension and designer may select size depending on hydraulics. Suggest D_6 = 0.2 m (0.67 ft).

H_1 = 0.2 m (0.67 ft) slot height.

H_2 = 0.25 m (0.80 ft) vertical distance from invert to junction of tank slope and tank side.

H_3 = 0.45 m (1.5 ft) height of circular gutter.

H_4 = 0.73 m (2.4 ft) vertical distance from top of circular gutter to junction of tank slope and tank side.

H_5 = 0.15 m (0.48 ft) vertical distance from circular gutter top to overflow weir.

H_6 = 0.15 m (0.48 ft) depth of weir gutter. Designer should check to make sure this depth is adequate.

H_7 = 0.69 m (2.28 ft) vertical distance from gutter top to invert of outlet pipe.

H_8 = 2.92 m (9.6 ft) depth of chamber with sloping sides. The horizontal dimensions of sludge hopper bottoms are usually no larger than 0.61 m (2.0 ft). If the bottom is given this width then H_8 will be reduced by 0.53 m (1.73 ft). Hence H_8 = 2.4 m (7.9 ft).

E_1 = 0.1 m (0.33 ft) weir gutter width.

E_2 = 0.1 m (0.33 ft) slot width at right angles to slope.

The size of the resultant structure for an average design flow of 7.5 l/sec (0.17 mgd) is shown in Figure 69.

The design and size of the overflow weirs and effluent gutters should be based on principles used in conventional primary tanks and should be revised from the values derived from Figure 68 as required.

CONSTRUCTION COSTS

A conventional round tank designed to handle the same flow and at the same suspended removal efficiency 7.5 l/sec (0.17 mgd), 45 percent suspended solids, would have essentially the same diameter, but less depth.

Cost estimates of the swirl primary separator were made for two purposes: 1) to indicate the probable construction cost of the facility; and 2) to compare its costs with that of a conventional primary settling tank designed for the same efficiency.

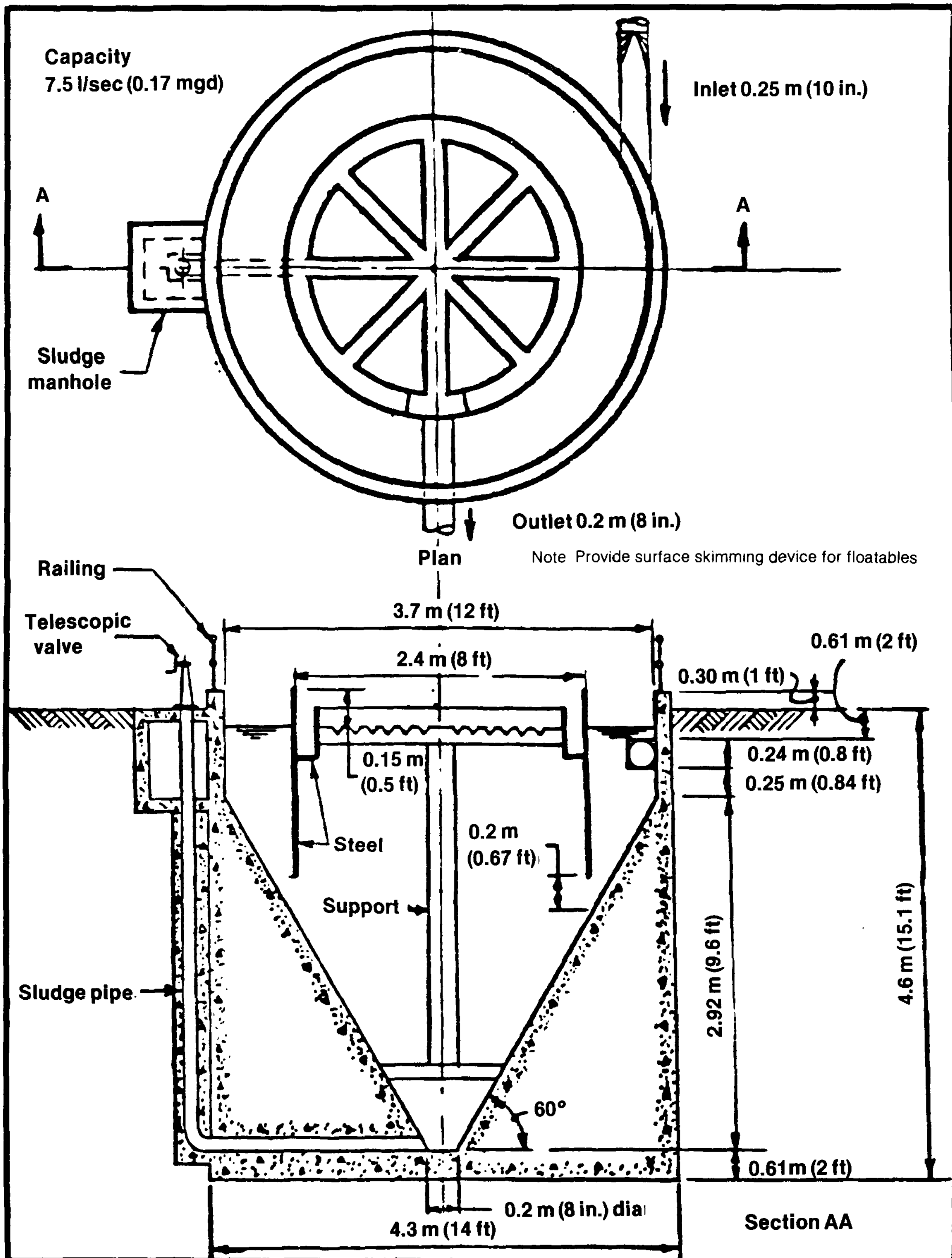


Figure 69 Swirl Primary Separator

The cost estimates are considered to be reasonable engineer's estimates. However, during periods of economic inflation, it is not unusual for contractors' bids to materially exceed engineers' estimates.

COST BASIS

The costs are based on the following:

a. Engineering News Record Construction Cost Index average for U.S. is 3140.

b. Unit prices as follows:

Steel Sheet Piling (for temporary use during construction)	$129/sq m	$ 12/sq ft
Excavation	$ 24/cu m	$ 18/cu yd
Reinforced concrete (swirl)	$390/cu m	$300/cu yd
Reinforced concrete (conventional)	$490/cu m	$375/cu yd

Note: The concrete for the swirl unit will require less reinforcing steel, thus a lower cost.

c. Contingency and engineering costs 35 percent of the foregoing items.

The estimated quantities of materials are based on the dimensions shown in Figure 69.

The swirl separator dimensions are derived in the previous section. It is assumed that the ground surface is 0.6 m (2 ft) above the crown of the inlet pipe and the top of tank is 0.3 m (1 ft) above ground surface. Since the top of overflow weir is 0.2 m (0.7 ft) above crown of inlet pipe, this provides 0.7 m (2.3 ft) of freeboard above the weir.

The conventional primary settling tank dimensions are inside diameter of 3.6 m (12 ft) and side water depth of 2.44 m (8 ft). These dimensions provide an overflow rate of 61,260 l/day/sq m (1,500 gal/day/sq ft) and a detention time of 57 minutes. The tank is set to provide a freeboard of 0.7 m (2.3 ft) with top of wall 0.3 m (1 ft) above ground surface.

The following assumptions are made for both structures:

a. Excavation is all in earth. The unit price includes cost of backfilling.

b. Temporary steel sheet piling is required 0.61 m (2 ft) outside exterior walls of structure.

c. Equipment cost for conventional settling tank includes cost of rake-type sludge collector with fixed bridge and center drive, scum collector, weir plates, telescopic valve, and electrical work.

d. Miscellaneous costs for swirl separator includes cost of skirt, weirs, gutters, telescopic valve, center support for weir gutters, piping, and railing around tank.

e. Miscellaneous costs for conventional settling tank includes piping within limits of structure, gratings, and railing around periphery of tank.

COST OF SWIRL PRIMARY SEPARATOR

The estimated construction cost of a swirl separator with a capacity of 7.5 l/sec (0.17 mgd) is $117,090. The breakdown of this cost is shown in Table 27.

Table 27
Construction Cost of Swirl Primary Separator

Capacity 7.5 l/sec (0.17 mgd)

Item	Quantity	Amount
Sheet Piling	128 sq. m (1,380 sq ft)	$ 16,560
Excavation	340 cu m (440 cu yd)	7,920
Reinforced Concrete	125 cu m (162 cu yd)	48,750
Miscellaneous Costs	Job	13,500
Subtotal		$ 86,730
Contingent and Engineering Costs	35% +	30,360
Total		$117,090

COST OF CONVENTIONAL PRIMARY SETTLING TANK

The estimated construction costs of a conventional primary settling tank with a capacity of 7.5 l/sec (0.17 mgd), based on the dimensions shown in Table 28 is $129,370. The breakdown of this cost is also shown in Table 28.

As the capacity of the swirl unit increases, there is a rapid increase in cost as compared to the cost of conventional units, due mostly to the increased excavation sheeting and amount of reinforced concrete. Similar construction calculations were made for comparable units having a capacity of 21.9 l/sec (0.5 mgd) with a suspended solids efficiency of 60 percent. The construction cost of the swirl unit was estimated to be $458,000 and the conventional unit $207,000. Figure 70 is a plot of the cost comparisons made.

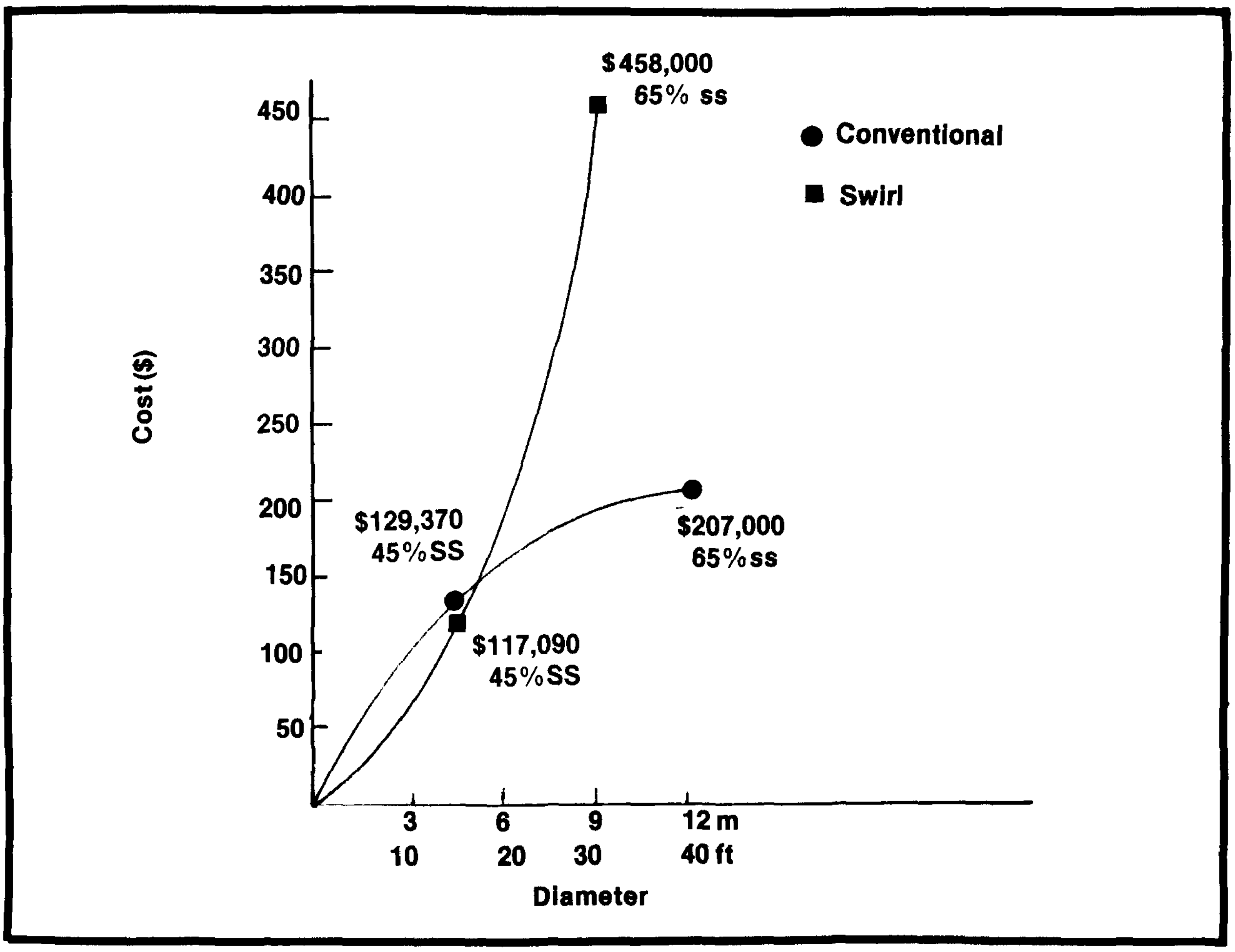


Figure 70 Cost vs Diameter Swirl and Conventional Primary Treatment

Table 28
Construction Cost of Conventional
Primary Settling Tank

Capacity 7.5 l/sec (0.17 mgd)

Item	Quantity	Amount
Sheet Piling	96 sq m (1,050 sq ft)	$ 12,600
Excavation	345 sq m (450 cu yd)	8,100
Reinforced Concrete	40 cu m (51 cu yd)	19,130
Equipment	Job	53,000
Miscellaneous	Job	3,000
Subtotal		$ 95,830
Contingent and Engineering Costs	35% +	33,540
Total		$129,370

Table 29
Comparison of Operation and Maintenance Costs
for Primary Treatment Units

	Conventional	Swirl
1. Labor: operation, 1 hr/day at $10/hr	$ 2600	—
maintenance, 0.54 hr/day at $10/hr	1,400	1,000
2. Materials and supplies	1,000	—
3. Power: 2 pumps at 1/2 hp, 1 hr/day $0.06/kwh	400	400
4. Annual maintenance at 3% of capital cost		
Primary tank sludge collections	150	
Raw sludge plunger pumps	120	120
Total annual O & M	$5,670	$1,520

Operating and maintenance costs for a 43.8 l/sec (1 mgd) unit, the smallest size for which USEPA data is available, can be estimated as shown in Table 29.

COMPARISON OF COSTS

From the foregoing it is seen that the construction cost of the swirl separator will be $117,090 compared to $129,370 for a conventional settling tank of 7.5 l/sec (0.17 mgd). Annual operating and maintenance costs may be $4,000 less with the swirl unit. The surface area required for units of this low volume does not appear to warrant a comparison of land cost savings.

This comparison assumes that the two structures will produce equal results in removal of suspended solids in raw sewage. The sizing of the conventional primary settling tank is based on standard design criteria. The sizing of the swirl primary separator is based on model results in the laboratory using IRA-93 resin as representative of suspended solids in raw sewage.

Cost comparison of large size units do not appear warranted at this time. A different configuration is obviously needed for large units to avoid the adverse construction costs of such a deep structure. A flat bottom with scrapers sacrificing the principal of no moving parts appears reasonable.

The present worth of the swirl separator units is shown in Table 30. The present worth is based on a life of 20 years and an interest rate of 7-1/8 percent. Hence the present worth of the operation and maintenance costs for a 20-year period is 10.49 times the annual cost.

For the unit with capacity of 7.5 (0.17 mgd) at 45 percent removal the present worth of the conventional unit is $188,850 and the swirl separator is $133,035. Thus the present worth of the conventional unit is 42 percent greater than that of the swirl separator.

For the unit with capacity of 21.9 lsec (0.5 mgd) and 60 percent removal the present worth of the conventional unit is $266,480 compared to $473,945 for the swirl separator. Thus the present worth of the conventional unit is 44 percent less than that of the swirl separator.

Table 30
Present Worth Swirl Primary Separator Treatment Units

	Conventional Tank	Swirl Separator
Capacity 7.5 l/sec (0.17 mgd) (45% ss removal)		
Construction cost	$129,370	$117,090
Operation and maintenance cost	59,480	15,945
Cost total present worth	$188,850	$133,035
Capacity 269 l/sec (0.5 mgd) 60% ss removal		
Construction cost	$207,000	$458,000
Operation and maintenance cost [(a)]	59,480	15,945
Cost total present worth	$266,480	$473,945

(a) assumes cost of O & M for both units to be the same

PROTOTYPE TEST

The Municipality of Metropolitan Toronto, Ontario, constructed a 3.66 m (12 ft) diameter unit at the Humber Wastewater Treatment Plant (Figure 71) to evaluate treatment efficiency.

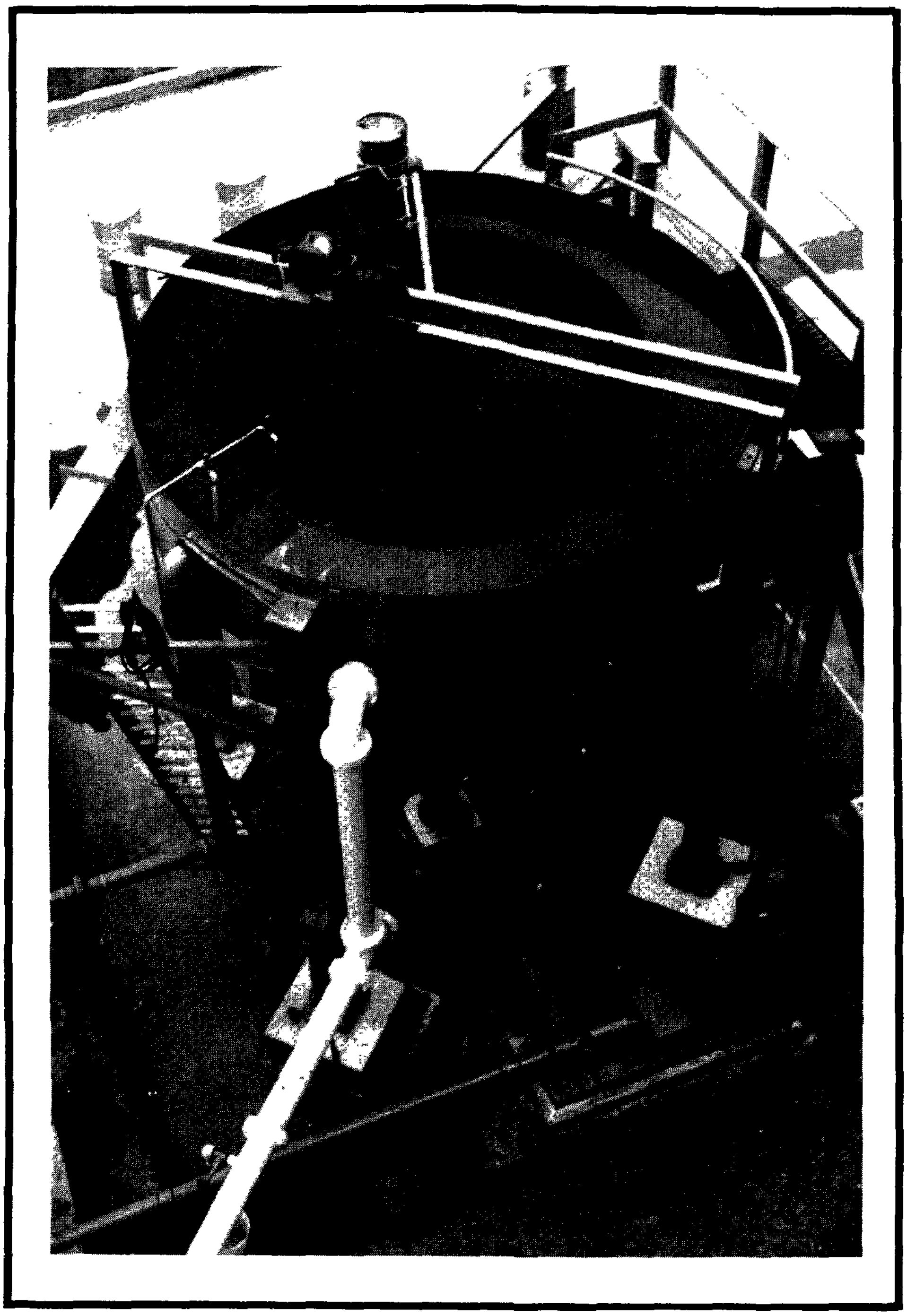


Figure 71 Swirl Primary Separator, Toronto, Ontario

Flowrates were 0.79 cu m/min (0.3 mgd). These studies indicated that the device closely matched the treatment efficiency of conventional primary sedimentation at an overflow rate of 65.2 cu m/sq m/d (1600 gpd/sq ft) which is 2.67 times conventional design (17). Figure 72 gives a comparison of time to achieve treatment between the swirl and the conventional system at Toronto. Its height and diameter are equal, thus providing a relatively deep structure which enhances sludge thickening.

For small treatment facilities the relatively high overflow rates or lower detention times used with swirl concentrator design at various levels of suspended solids removal make the device potentially less costly to construct with less space required, thus enhancing its use in wastewater plant expansion and combined sewer overflow treatment. Its static sludge collection system enhances appeal because of lower operation and maintenance costs.

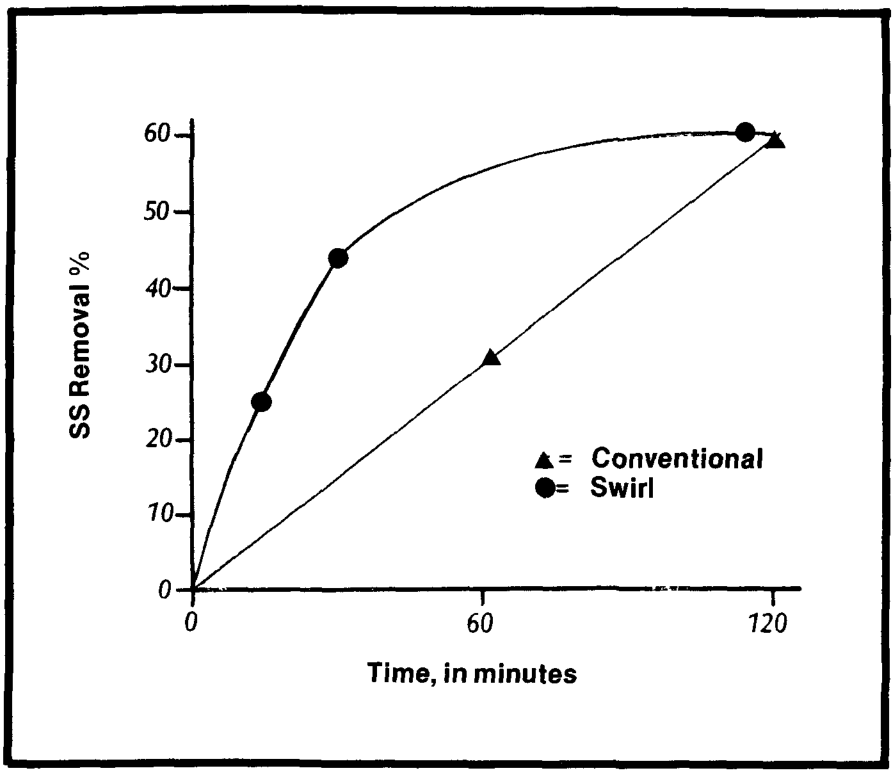


Figure 72 Comparison of Time to Achieve Primary Treatment

Section VII

THE SWIRL CONCENTRATOR FOR EROSION RUNOFF TREATMENT

The swirl secondary flow pattern to separate solids from a moving stream makes its use appropriate for separating settleable erosion sediment from surface runoff. The unit has been designed to concentrate the heavier soil particles. The unit cannot be used alone. Rather good construction management practices are needed throughout the project. The swirl unit can provide additional pollution control before discharge of flows to receiving waters or storage ponds.

BASIC ASSUMPTIONS

Several basic assumptions have been made concerning how and when the swirl device will be used. It is assumed that temporary units will be small, i.e., generally 3.7 m (12 ft) in diameter. Multiple units will be used to treat flows requiring larger capacity.

A permanent facility will require a flow-splitting diversion device where multiple units are used, and bar screens to protect the unit from coarse debris. A solids basin or other facility will be needed for settling the solids in the 5 to 14 percent range of the concentrated underflow. The clarified flow may be discharged into a detention pond, or directly into receiving waters, based upon the degree of protection against erosion solids required by water quality standards.

The unit is designed to be self-cleansing. However, the flow field is not strong enough to move gravel or heavy loadings of sand through the underflow. Thus it is imperative that good erosion and sediment control construction practices be used. This treatment device does not take the place of conventional methods of on-site control.

DESCRIPTION

The swirl concentrator for erosion runoff treatment can be characterized as a shallow tank with a tangential inlet at the bottom, a circular overflow weir with a central downshaft discharge pipe, and a small underflow drain for removal of concentrated solids.

Figure 73 is a schematic view of the unit. The essential features, as indicated in the figure, include:

A. a square inlet

B. a baffled inlet to ensure the development of the swirl flow field

C. flow spoilers to improve the efficiency of the circular discharge weir

D. an internally supported clear water overflow weir with a central downshaft discharge pipe

E. a flat weir plate

F. a central downshaft for the clarified overflow

G. a concentrate discharge take off to a settling or thickening basin

H. a flat floor

DESIGN GUIDELINES

The design procedure is developed in accordance with the various elements formally required for a complete system. These elements are:

Hydrological considerations
Solids analysis
Swirl unit design
Efficiency computation
Assessment of retention volumes
Other design considerations and details

A typical site situation is shown in Figure 74 . This plan shows a large drainage area with a stormwater retention facility. The swirl unit and soil collection pond intercept this flow ahead of the stormwater detention ponds. It is assumed that all runoff from the basin is detained on the property and passed through the swirl unit or units.

Hydrological Considerations

For purposes of determining the quantity of runoff to be expected from the drainage area any of a number of methods can be used. In reference to a survey conducted by APWA, (19) rainfall runoff predictions in practice are based primarily on unit hydrographs and the Rational Method. In general, maximum erosion will occur under conditions of peak runoff. For a device such as the swirl when used as a temporary treatment device, the unit should be designed for a rainfall intensity of less than a one-year recurrence interval, although in many cases the choice of a design rainfall is determined for a specific project by the requirements of the local or state public agency having jurisdiction.

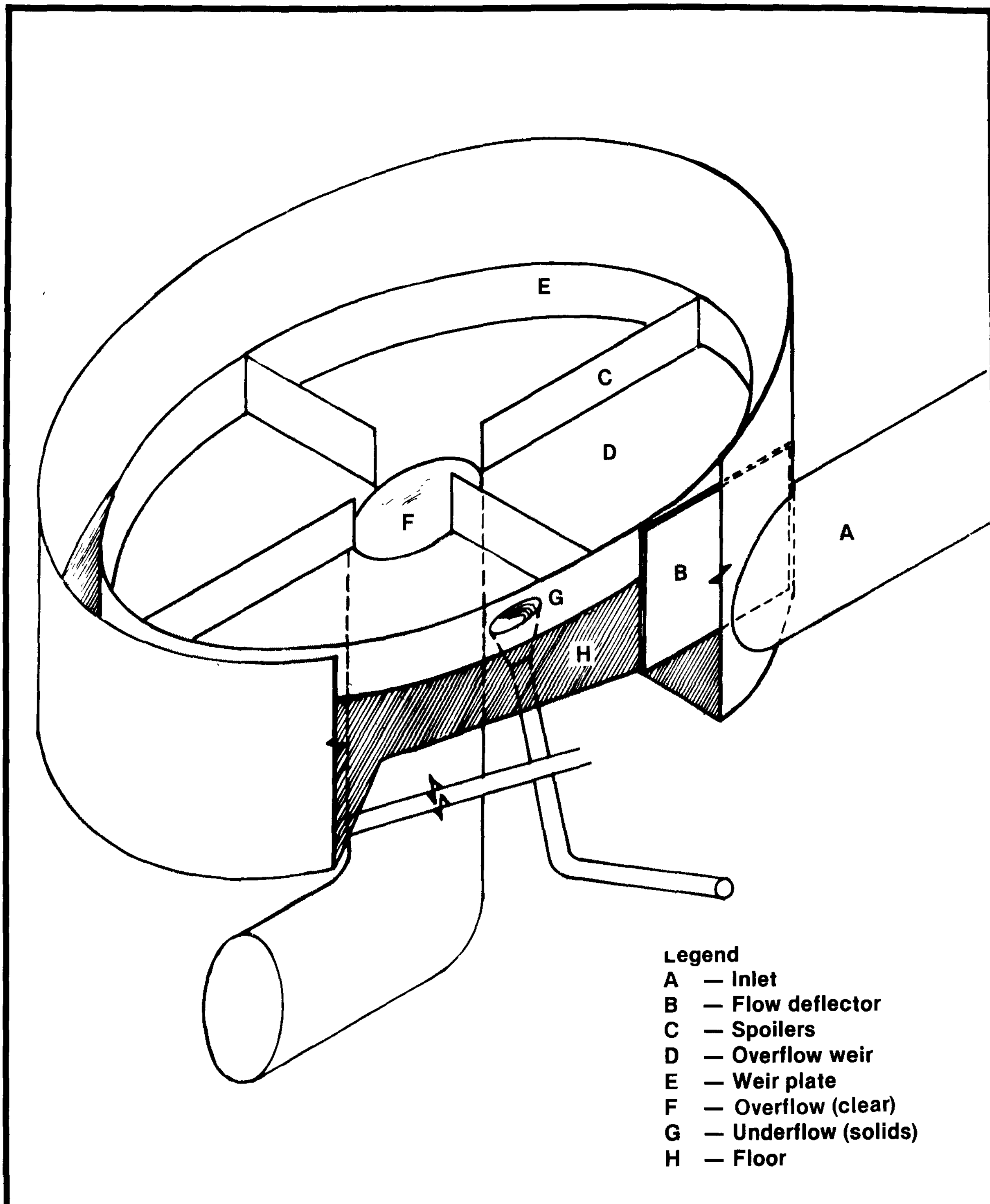


Figure 73 Schematic View, Swirl Concentrator for Erosion Runoff

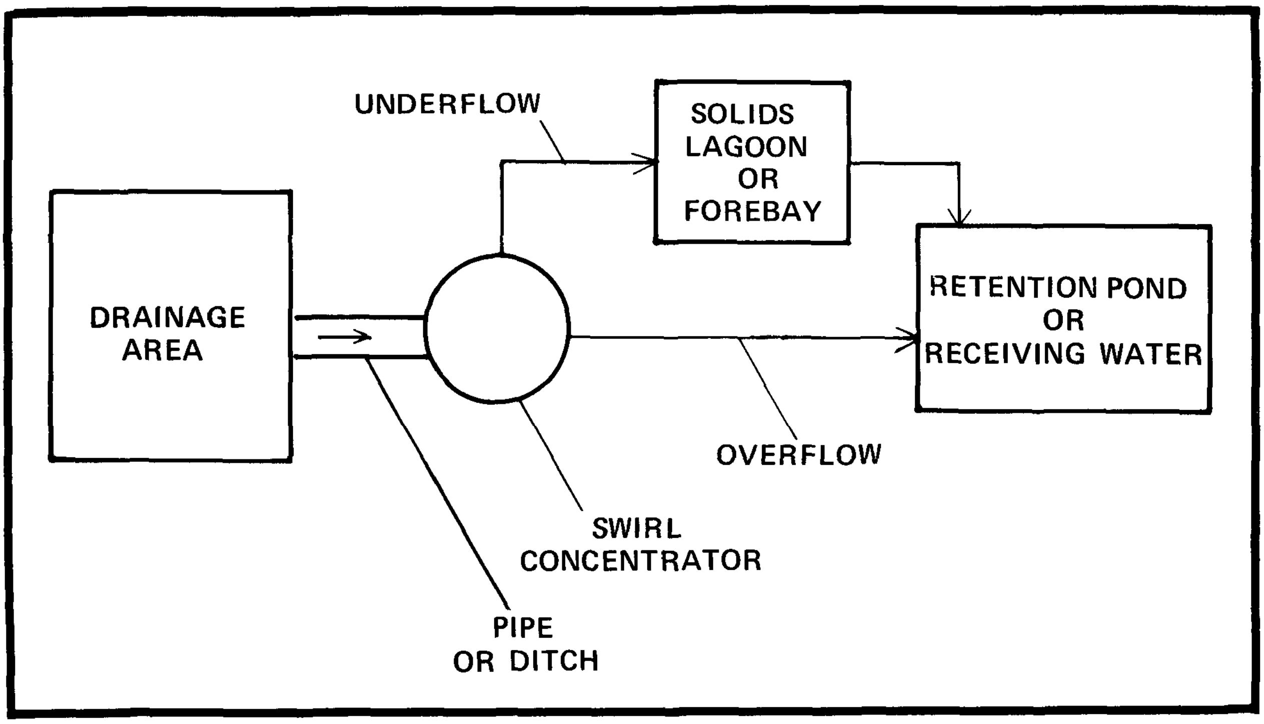


Figure 74 Typical Application, Swirl Concentrator as an Erosion Runoff Treatment Device

A second part of the hydrologic analysis required to design an erosion control facility involves an estimate of the peak volume of runoff for a given storm. This volume will be used to size the retention pond and the soil collector pond. Obviously, the high-intensity, short-duration storm may contribute a high flow rate for a short period but it would represent only a portion of the total volume of runoff that could be expected from a storm of longer duration.

The use of the Rational Method C factor will result in an estimate of a larger flow than would ordinarily be anticipated for all but the most intense storms.

Perhaps the most accurate method for determing the volume of runoff would be to integrate the area on a hydrograph determined for this watershed. For the determination of this volume, the use of a unit hydrograph would be advantangeous.

Various other methods are also available for computing the storage volume necessary to hold the total runoff.

The final hydrologic determination deals with an annual estimate of the total quantity of sediment to be expected. This volume will be used to estimate the total amount of settleable solids to be collected in the two ponds. Reference to a chart of expected annual rainfall in the project area, will provide the annual precipitation rate. It is probably not necessary to reduce these values for precipitation occurring as snowfall for the purpose of this estimate.

In summary, these three calculated quantities will be used in the following manner:

A. The peak runoff rate will be used to size the swirl concentrator erosion control device or devices, the main drainage trench conducting flow to the device and any inlet conduit that must be used.

B. The single storm volume will be used to size the solids basin.

C. The annual storm runoff volume will be used to estimate the quantity of settleable material which will accumulate in the retention basins. This represents material for which storage capacity must be provided within the solids basin, or the volume of material which must be removed.

Solids Analysis

The next step in the design procedure is to determine the quantity, type and size of material that is likely to be found in stormwater runoff. Table 31, presents an analysis of a sample of storm runoff from a construction site which was sieved and separated into groups having similar specific gravities. Such an analysis is used to determine the type and specific gravities of the material present, thus enabling a reasonable estimate of tne type and quantity of material that can be removed in a swirl erosion control unit. This example should be viewed as merely an indication of the type of investigation that should be conducted. There may be many sites for which more elaborate and complete analyses may be desirable.

Table 31
Sieve Analysis, Sample from Construction Site

Sieve Size	Size mm (in)	Material Retained gm (oz)	% Retained	SG/2.65	% Retained According to SG SG/1.20	SG/1.01
10	2.000 (.080)	4.0 (0.14)	1.14	1.04	—	0.1
20	0.840 (.030)	6.5 (0.23)	1.86	1.66	—	0.2
60	0.250 (.010)	39.0 (1.40)	11.14	8.64	2.0	0.5
100	0.149 (.006)	100.5 (3.50)	28.71	23.61	5.0	0.1
120	0.125 (.005)	77.0 (2.70)	22.00	21.00	1.0	—
200	0.074 (.003)	44.0 (1.50)	12.57	12.07	0.5	—
PAN	—	79.0 (2.80)	22.57	22.57	—	—
			TOTAL	90.59	8.5	0.9

Assuming that it is desirable to remove as much settleable material as possible in the swirl unit, the smallest particle that is predicted from the model studies to be removed is a grit particle, SG 2.65, 43 microns in diameter, having a settling velocity of 0.14 cm/sec (0.055 in./sec) as shown in Figure 75. A design incorporating the removal of this size of grit particle will also remove larger size particles of lighter specific gravity. For example, a particle of SG 2.65 and a settling velocity of 0.14 cm/sec (0.055 in./sec) is 0.043 mm (0.002 in.) in diameter. Particles having a SG of 1.20 with a diameter of 0.14 mm (0.006 in.) will settle at the same rate as particles having a SG of 1.01 (organic material) with a diameter of 0.6 mm (0.02 in.). Particle sizes larger in diameter than those quoted are expected to be removed. In the sieve analysis shown in Table 31, the material expected to be removed in part by the swirl concentrator is shown in the specific gravity columns at the right side of the table above the asterisk marks, considering that the settling velocity is 0.14 cm/sec (0.055 in./sec) for a particle having an SG of 2.65.

Hydrometer analysis using pan material, or 22.5 percent of the total sample, showed:

Percent particle size greater than 0.052 mm (0.002 in.) - 16%

Percent particle size less than 0.052 mm (0.002 in.) - 6.57%

From design data using a particle settling velocity of 0.14 cm/sec (0.055 in/sec) it was determined that the following percent of material will be subject to removal in the swirl chamber:

SG 2.65 90.59 - 6.57 = 84.02%
SG 1.20 8.5 - 1.5 = 7.0 %
SG 1.01 0.9 - 0.6 = 0.3 %

(These quantities are shown in the table as the percent in each SG column above the asterisk)

Total material subject to removal by swirl concentrator - 91.33%
Total material not subject to removal - 8.6%

The percent of removed material shown here will be multiplied by the recovery efficiencies of the chamber from the design curves as explained next.

Unit Design

This part on design makes frequent reference to the following listed Figures.

76 Prototype Particle Sizes Represented by Gilsonite - SG 1.06

77 Recovery Rates in Model as Function of Particle Settling Velocity and Discharge with 5 Percent Draw Off

78 Recovery Rates in Model as Function of Particle Settling Velocity and Discharge with 10 Percent Draw Off

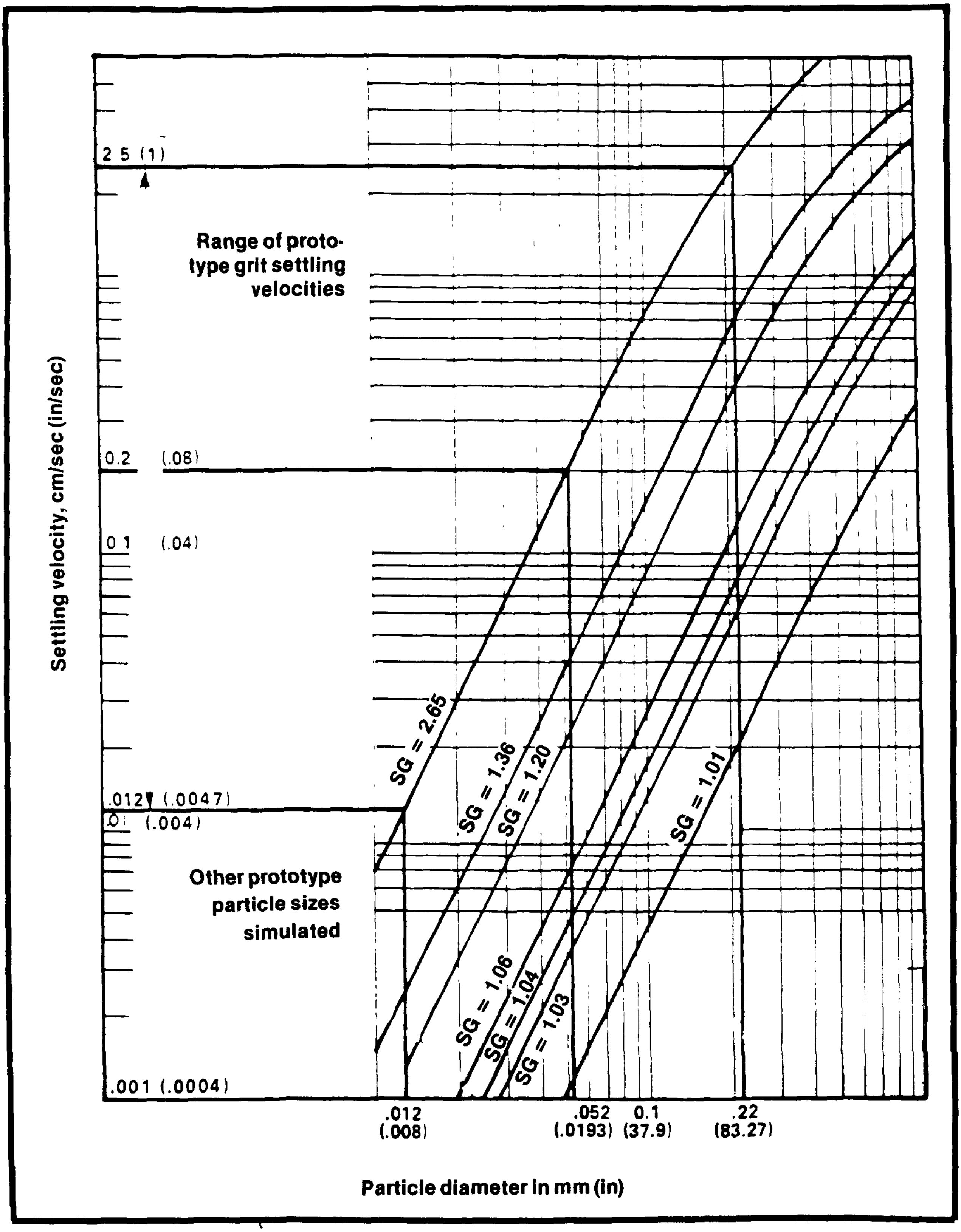


Figure 75 Prototype Particle Sizes Represented

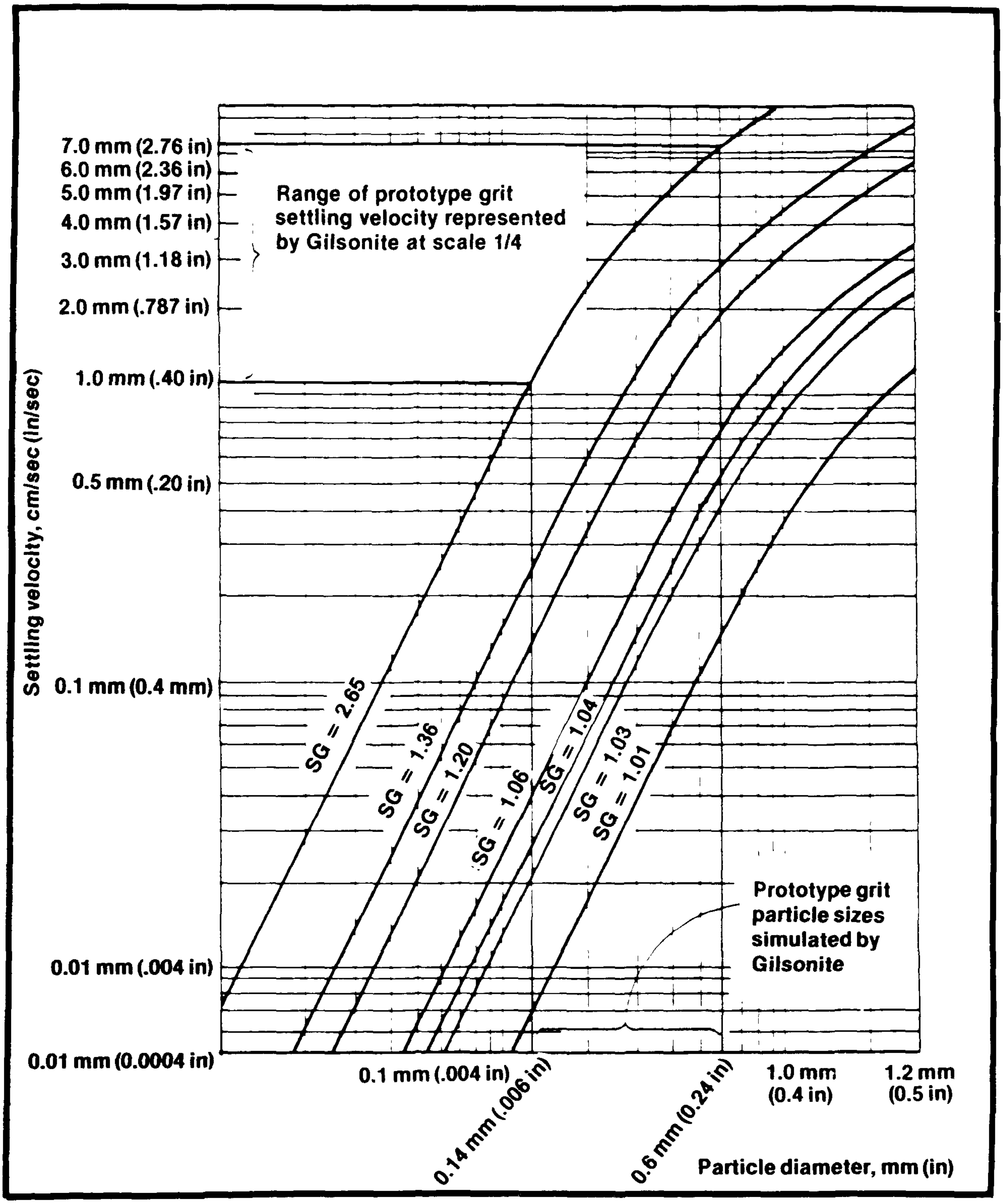


Figure 76 Prototype Particle Sizes Represented by Gilsonite-SG 1.06

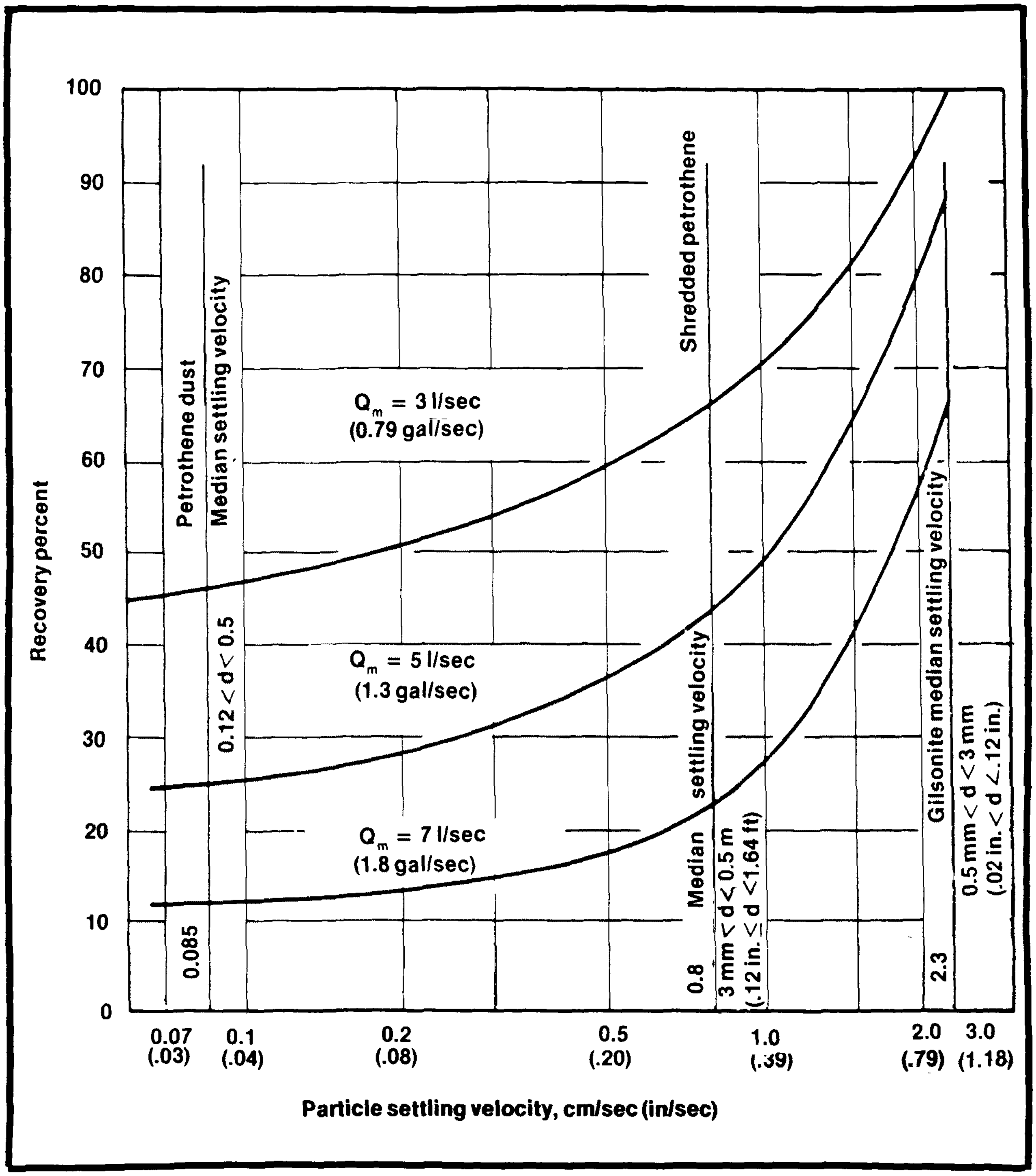


Figure 77 Recovery Rate on Model as Function of Particle Settling Velocity and Discharge with 5% Drain-Off

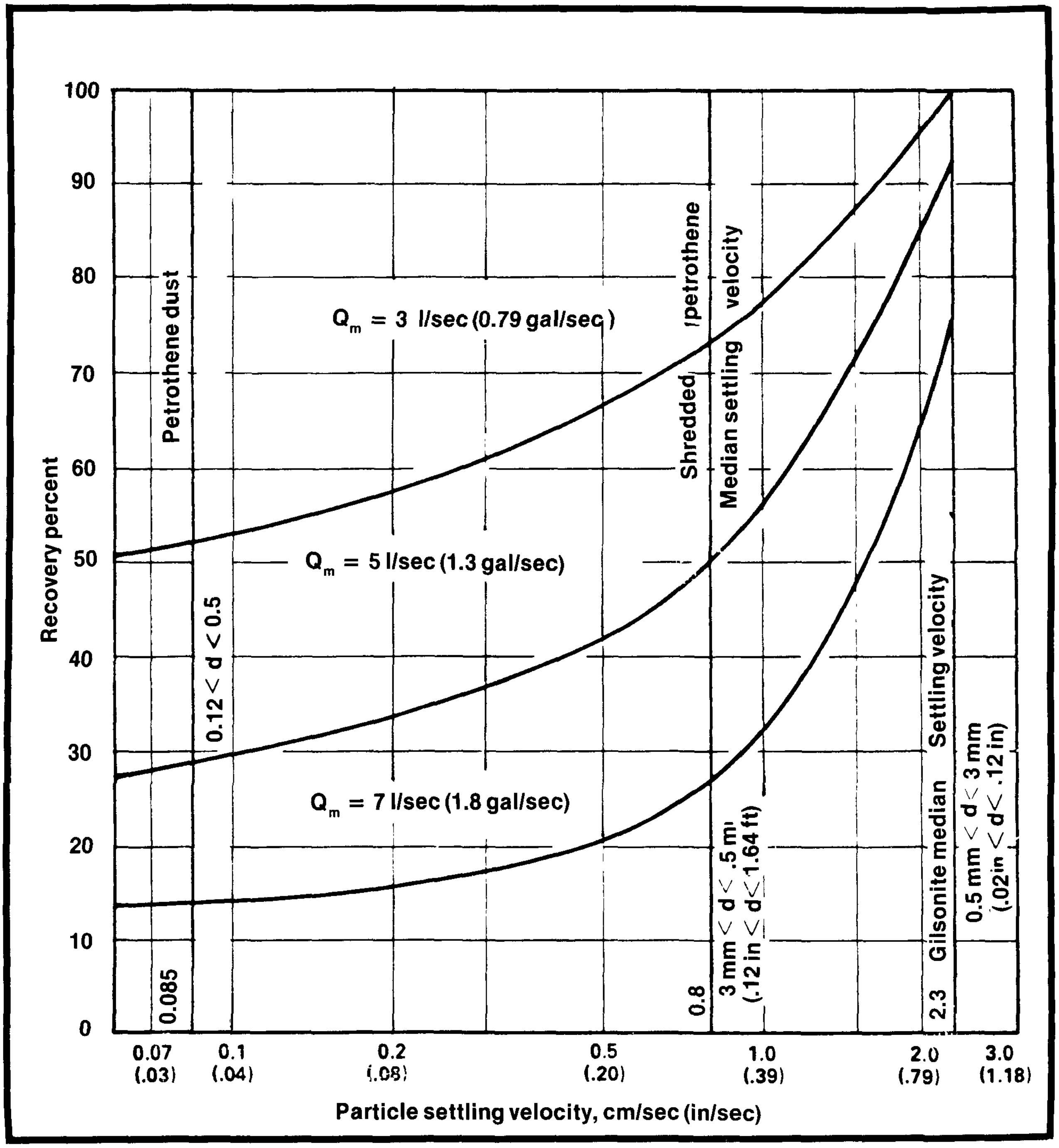


Figure 78 Recovery Rate on Model as Function of Particle Settling Velocity and Discharge with 10% Drain-Off

The procedure described in this section is relevant to a standard 3.66 m (12 ft) diameter tank as the swirl chamber. The dimensions of the structure are determined from Figure 83.

Under operating conditions it is assumed that the user has a situation in which the discharge Q_p is known as well as the particle settling velocity, V_{sp}, of the materials to be removed from the flow.

1. Enter Figure 80 (5 percent draw off) where the expected discharge appears on the abscissa

2. Move up in the graph until the given particle settling velocity curve (or particle size) is found

3. Check whether or not this intersection gives an acceptable rate of recovery

4. If the recovery is not high enough, try Figures 81 or 82 in which draw off is increased, respectively, to 10 and 14 percent of the inflow

5. If conditions are still not acceptable, even with the larger draw off rates, then reduce the expected discharge per unit by providing multiple swirl chambers

6. If this gives too many standard 3.66 m (12 ft) units, try larger chambers, making reference directly to Figures 77, 78, and 79; the recovery curves for the 0.914 m (3 ft) diameter model

7. Select an approximate new chamber diameter, D_n and divide this by the model diameter to find the new scale:

$$1/\lambda_n = 0.914/D_n \text{ m} = 3/D_n \text{ ft}$$

Where:

λ_n = scale factor

Next calculate:

new discharge scale = $1/\lambda_n^{5/2}$
new settling velocity scale = $1/\lambda_n^{1/2}$

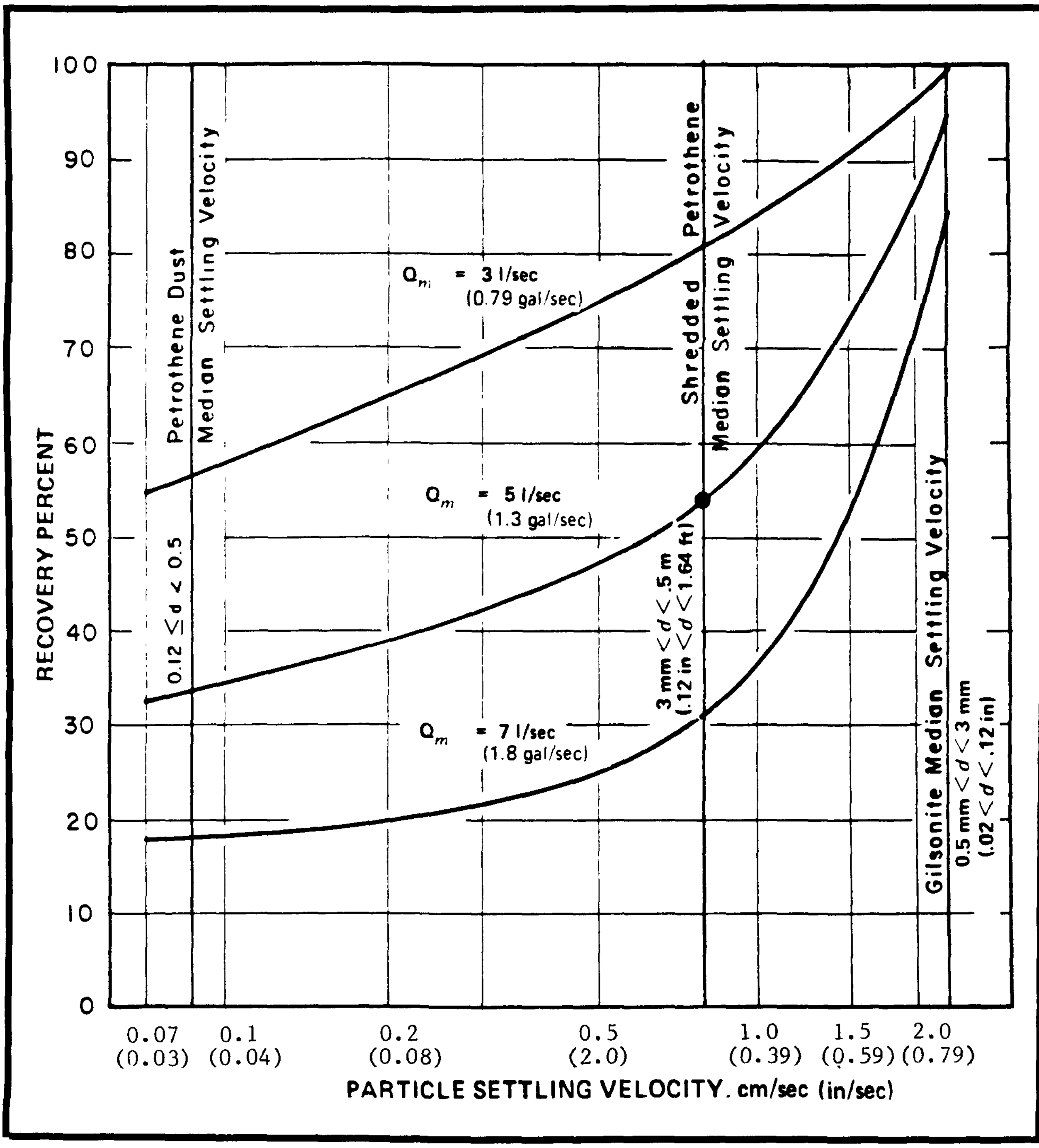


Figure 79 Recovery Rates on Model as Function of Particle Settling Velocity and Discharge with 14% Drain-Off

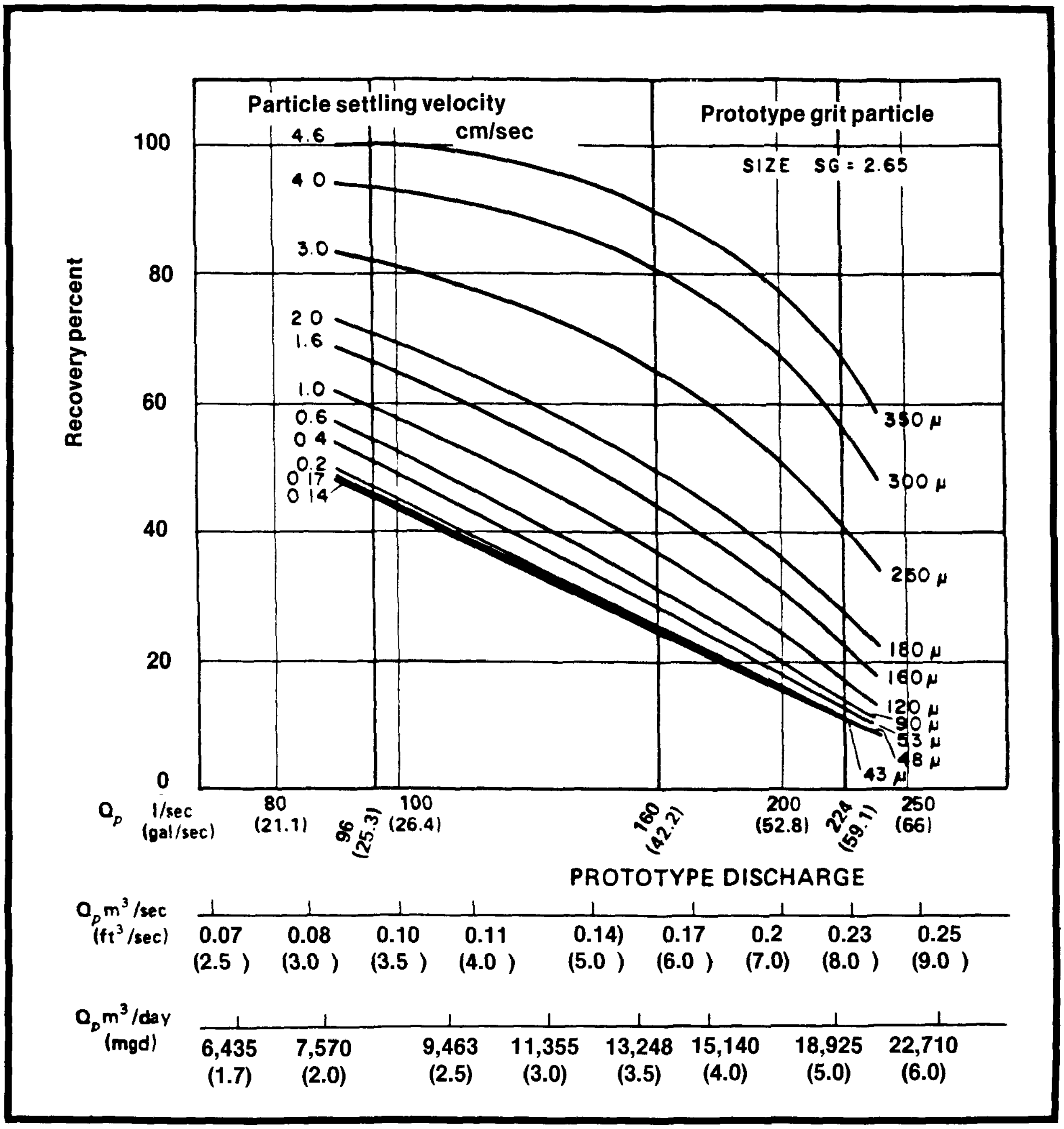


Figure 80 Predicted Prototype Recovery Rates with 5% Drain-Off

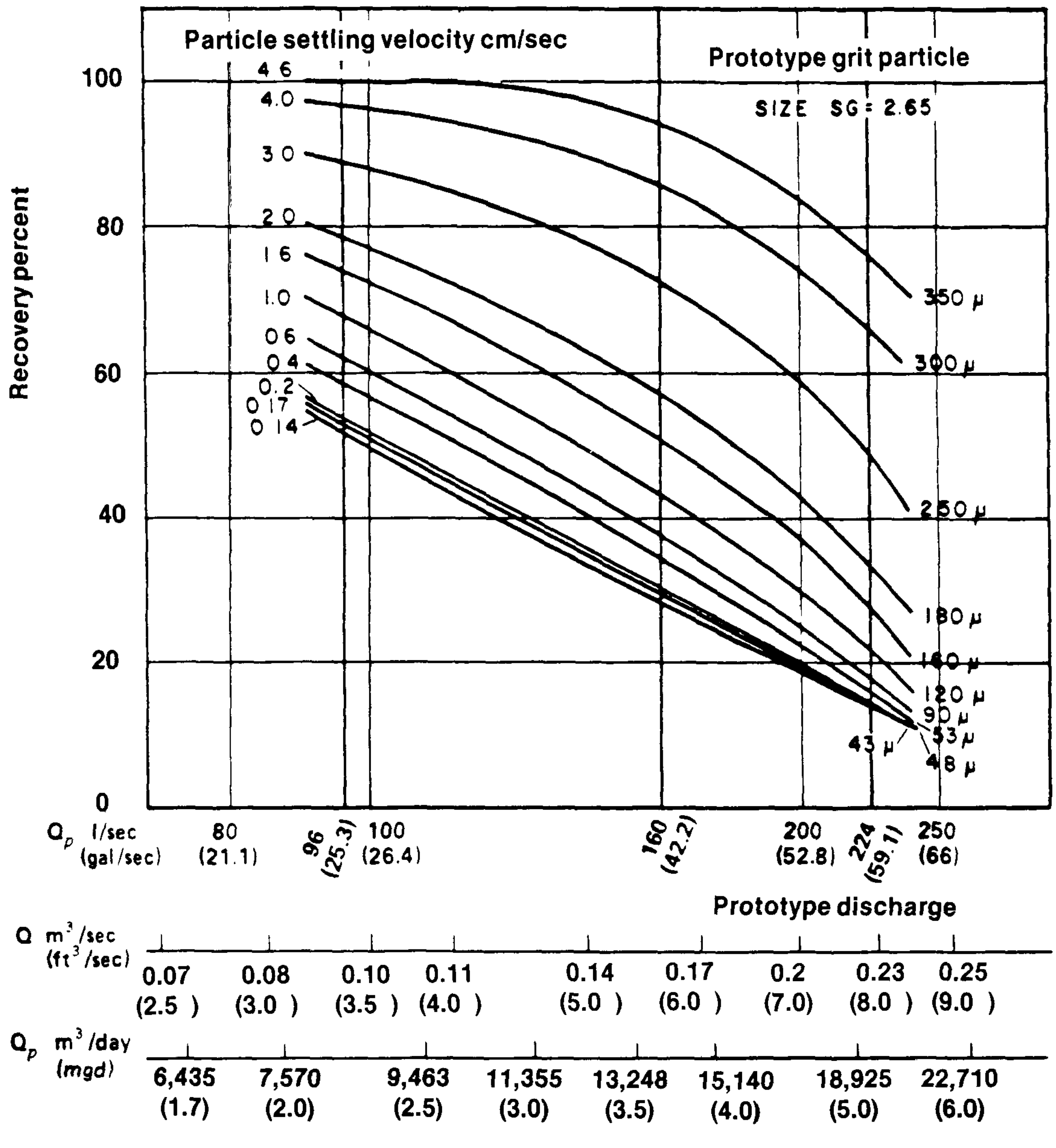


Figure 81 Predicted Prototype Recovery Rates with 10% Drain-Off

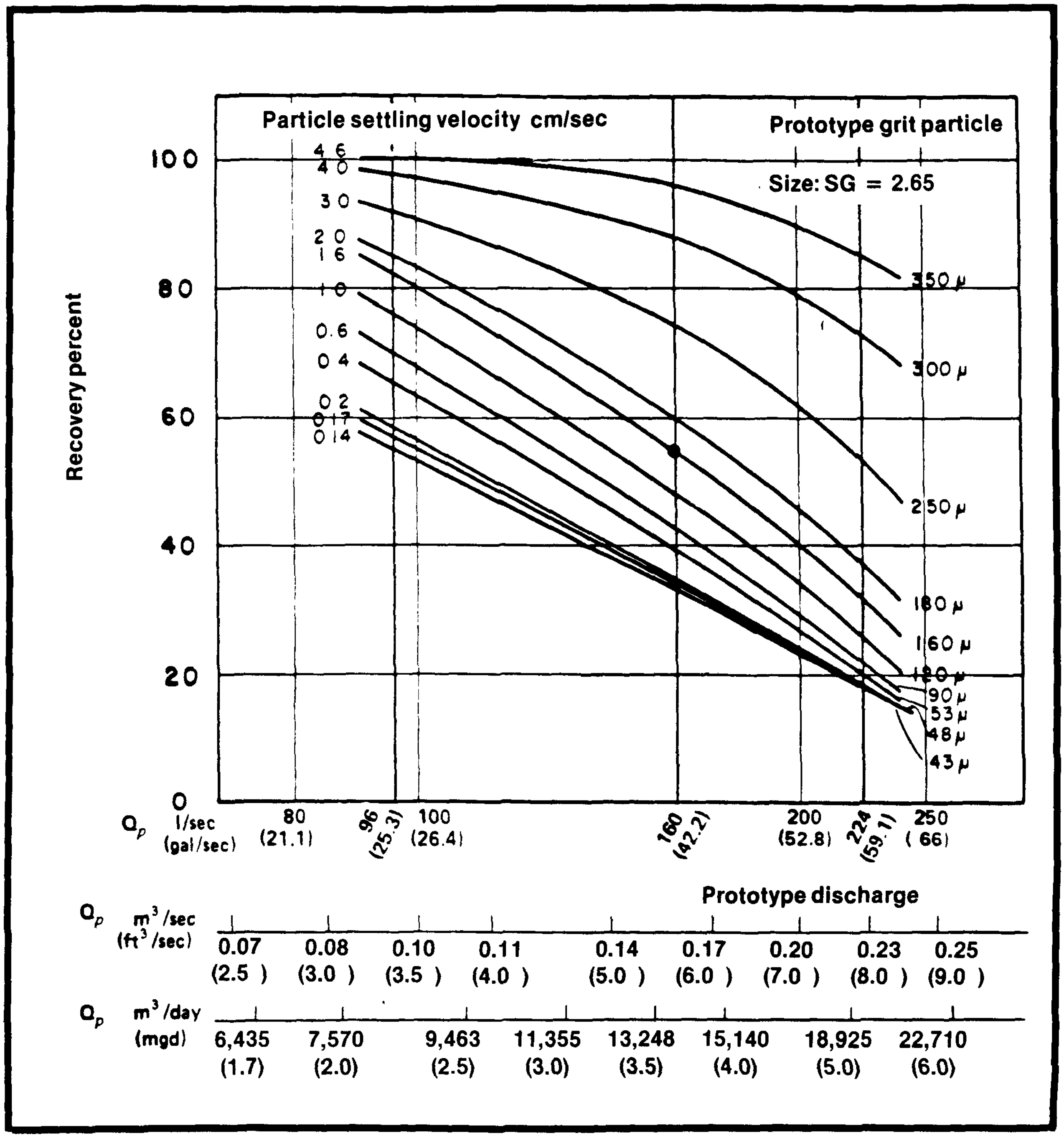


Figure 82 Predicted Prototype Recovery Rates with 14% Drain-Off

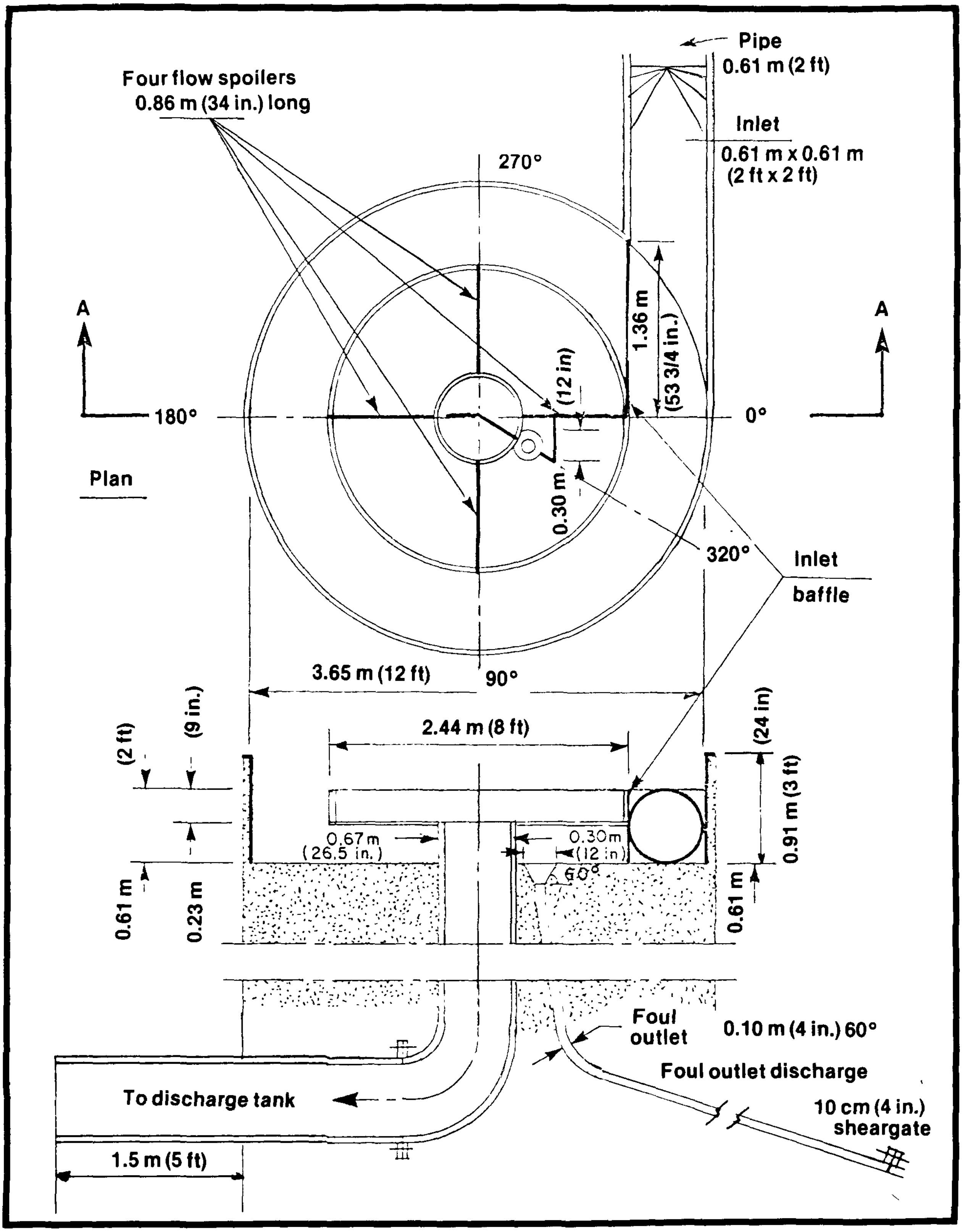


Figure 83 General Design Dimensions, Swirl Concentrator for Erosion Runoff Treatment

8. Multiply:

$$Q_p \times 1/\lambda_n^{5/2} = Q_m \text{ model discharge}$$

$$v_{sp} \times 1/\lambda_n^{\frac{1}{2}} = v_{sm} \text{ model particle settling velocity}$$

9. Go into Figures 77, 78, or 79 with these model values, interpolating as necessary between the discharge curves, to find the corresponding recovery.

10. If the recovery is too low, try progressively larger chambers, each time following the procedure in steps 7, 8, and 9 above, until a satisfactory recovery rate is obtained

11. Use Figure 83 to find the dimension of the new chamber. Since the chamber shown on the figure is the standard 3.66 m (12 ft) unit studied at scale 1:4, each dimension must be multiplied by the factor $\lambda_n/4$

12. The bottom orifice must be large enough to prevent clogging by solids which may be carried by the stormflow into the chamber.

For purposes of illustrating the procedure for the application of this swirl unit to the problem of soil erosion, two examples will be given. The first is based upon an engineering approach where a permanent facility is to be designed for a required level of efficiency. The second example is for the case where a developer must provide <u>temporary</u> facilities at a construction site.

For a permanent erosion control facility the use of the swirl concentrator may be envisioned as an auxiliary treatment device installed ahead of a stormwater retention/detention facility. The primary purpose of the unit would be to concentrate the larger soil particles in order to retard the siltation of the retention facility or downstream receiving waters. To this end, the concentrated underflow could be directed to a readily cleanable auxiliary sediment trap where conventional equipment such as a backhoe, Gradall,(R) or even a bucket loader--assuming that the area could be dewatered--could be used to remove the collected soil.

Such a facility would minimize the total maintenance cost and improve the efficiency of the major storage facility or receiving waters.

For this example, let it be assumed that a 80.9 ha (200 ac) drainage basin is selected with a time of concentration of 45 minutes. Assuming that it is desired to find the peak runoff at a time when equilibrium conditions are established for this site, the duration of the storm is taken as the time of concentration. From a duration-intensity relationship established for this site, it is determined that the intensity is 1.27 cm/hr (0.5 in./hr). Further information on the site indicates that 20 percent of the basin is occupied by buildings for which a runoff coefficient of 0.9 is selected; 15 percent is roadways with a runoff coefficient of 0.9; and the remainder is grassed yard areas for which a runoff coefficient of 0.3 is assumed. An average coefficient for this site can be calculated as:

$$C_{ave} = \frac{(40 \text{ ac} + 30 \text{ ac}) \times 0.9 + 130 \text{ ac} \times 0.3}{200}$$

$$C_{ave} = 0.51$$

For this example a simplified method of calculation of rainfall and runoff will be used. In practice, each agency should use models or methods which present a better representation of what can be expected to occur.

The peak runoff calculated for this storm, using the Rational Method, is:

$$Q = C\ i\ A$$

$$Q = 0.51\ (1.27 \text{ cm/hr})\ 80.9 \text{ ha} = 52.4 \text{ cm/ha/hr}$$

$$= 52.4 \times 27.8 = 1{,}460 \text{ l/sec}$$

$$Q = 0.51\ (0.5 \text{ in./hr})\ (200 \text{ ac}) = 51 \text{ cfs}$$

This will be the flow to the swirl treatment facility--next an estimate of the peak volume must be made to size the retention pond and solids collector pond. With reference to a set of intensity-duration curves, it was observed that for the same recurrence frequency that was used in the determination of the peak flow rates, a storm of longer duration than 4 hours would yield an intensity of 1.02 cm/hr (0.4 in/hr). The peak rate of flow for this storm can be estimated in the same manner as previously:

$$Q = 0.51\ (1.02 \text{ cm/hr})\ 80.9 \text{ ha} = 42.1 \text{ cm/ha/hr}$$

$$= 42.1 \times 27.8 = 1170 \text{ l/sec}$$

$$Q = CiA = 0.51\ (0.4 \text{ in./hr})\ 200 \text{ ac}$$

$$= 40.8 \text{ cfs}$$

Various methods are available for computing the necessary storage volume. Using one of these methods, assume the resultant volume is 8,420 cu m (297,226 cu ft). This yields a larger volume than that associated with a short-duration, higher-intensity storm.

The final determination is to estimate the annual total quantity of sediment to be expected. Charts of estimated annual rainfall in the project area should be consulted. Assume that this value is 76.2 cm (30 in.) per year. It is also assumed that the area of the retention pond(s) is small compared to the total area, although this fact may not always be true. Neglecting the reduced volume resulting from the cumulative effects of smaller storm events, the maximum runoff volume per year is then:

$$V = 0.51 \left(76.2 \frac{\text{cm}}{\text{yr}}\right) \times \frac{1\text{m}}{100 \text{ cm}} \times 80.9 \text{ ha} \times \frac{10{,}000 \text{ sq m}}{\text{ha}} = 314{,}000 \text{ cu m/yr}$$

$$V = 0.51\ (30 \text{ in/yr}) \times \frac{1 \text{ ft}}{12 \text{ in}} \times 200 \text{ ac} \times 43{,}500 \frac{\text{sq ft}}{\text{ac}}$$

$$= 11{,}100{,}000 \text{ cu ft/yr} = 411{,}000 \text{ cu yd/yr}$$

These three calculated quantities will be used in the following manner:

A. Peak runoff rate will be used to size the swirl devices and the drainage conduits to and from the facility.

B. Single storm volume will be used to size the retention basin(s), and

C. Annual storm runoff volume will be used to estimate the quantity of settleable solids in the solids basin. This material must either be stored or removed.

With reference to Figure 82 it is seen that for a 3.66 m (12 ft) diameter chamber, the highest efficiency is obtained when the flow rate does not exceed 96 l/sec (3.4 cfs). Dividing the flow by a factor of 15 would give 96 l/sec (3.4 cfs) as the design flow for each of the chambers, and this flow in Figure 82 is at the left end of the curve at the highest possible removal efficiency for this particle size. The use of 15 chambers would also mean that higher intensity storms would still be handled by these chambers with only a small reduction in efficiency. In fact, the design runoff could be more than doubled in each chamber. It should be noted that if the 14 percent draw off rate is excessive for the volume of storage desired, Figures 80 and 81 should be used with smaller draw off rates and corresponding reductions in efficiencies.

Efficiency Computation

Using the efficiencies given in Figure 82 and the percent of each size material given in Table 31, the efficiency of the 3.66 m (12 ft) diameter chamber can be determined as shown in Table 32.

For specific gravities less than 2.65 an equivalent particle size for that particle can be obtained from Figure 76. As an example with reference to Table 32, a particle for specific gravity of 1.20 is taken as 0.25 mm (0.01 in.). In Figure 76, find this size along the abscissa:

go vertically upward to the curve marked SG 1.20

then left or horizontally to the curve marked SG 2.65

then downward to the abscissa.

The values read,

settling velocity is 0.5 cm/sec (0.19 in./sec) for a SG 1.20

and

particle size 0.25 mm (0.01 in.):

an equivalent particle of SG 2.65 having this settling velocity is a particle of 0.082 mm (0.003 in.)

Refer back to Figure 82 for this size particle of 82 microns and settling velocity of 0.5 cm/sec (0.019 in./sec).

Table 32
Swirl Efficiency Analysis

Sieve Size	Col. 1 Particle Size mm(in)	Col. 2 Percent Retained SG 2.65	Col. 3 Percent Eff. from Fig. 82	Col. 4 $= \frac{1 \times 2}{100}$	Col. 5 Percent Retained SG 1.20	Col. 6 Percent Eff. from Fig. 82 & 85	Col. 7 = 4 x 5
10	2.00 (.08)	1.04	100	1.04	--	--	--
20	0.84 (.03)	1.66	100	1.66	--	--	--
60	0.25 (.01)	8.64	92	7.95	2.0	69	1.38
100	0.149(.006)	23.61	82	19.36	5.0	56	2.8
120	0.125(.005)	21.00	79	16.59	1.0	--	--
200	0.074(.003)	12.07	69	8.32	0.5	--	--
HYD	0.052(.002)	16.00	59	9.44			
				64.36			4.18

Sieve Size	Col. 8 Percent Retained SG 1.01	Col. 9 Percent Eff. from Fig. 82 & 85	Col. 10 = 7 X 8
10	0.1		0.1
20	0.2	60	0.12
60	0.5	--	--
100	0.1	--	--
120	--	--	--
200	--	--	--
HYD	--	--	--
			0.22

Total percent of removal material removed in swirl unit = 64.36% + 4.18% + 0.22%
= 68.76%

Total percent of settleable material removed by swirl concentrator
= 68.76% x 91.3% = 62.7%

For a flowrate of 96 l/sec (3.4 cfs) this yields an efficiency of 69 percent.

This procedure is continued for other particle sizes. It is seen then an efficiency of 69 percent is predicted for this material if a set of 15 swirl concentrators were used.

Alternate Chamber Design

The design discharge Q is 1,460 l/sec (51 cfs). With reference to Figure 79 the smallest particle shown is one having a settling velocity of 0.07 cm/sec (0.03 in./sec). Figures 77 and 78 could be used for 5 percent and 10 percent draw off, respectively. Figure 79 was selected since the best recovery occurred with a draw off of 14 percent. Assume that four prototype chambers will be used, each having a diameter of 6.4 m (21 ft). This sets the model scale at:

$$\lambda = L_p/L_m = 6.4 \text{ m}/0.914 \text{ m } (21 \text{ ft}/3\text{ft}) = 7$$

where L_p = diameter prototype unit and L_m = diameter of hydraulic model

From the Froude Law the velocities of settlement can be related as:

$$v_{sp}/v_{sm} = \sqrt{L_p/L_m} = \sqrt{7} = 2.65$$

where V_{sp} = settling velocity of solids in the prototype and V_{sm} = settling velocity of model solids

The model discharge is also found from the Froude Law as:

$$Q_p/Q_m = (L_p/L_m)^{5/2} = (7)^{5/2} = 129.64$$

$$Q_m = Q_p/129.64 = \frac{1,460}{4} \times \frac{1}{129.64} = 2.81 \text{ l/sec}$$

where Q_p = flow through the prototype and Q_m = flow through the model

Referring now to Figure 79, the discharge line for 2.81 l/sec (0.74 gps) must be interpolated between the 3 l/sec (0.80 gps) line and zero at 100 percent recovery. Assume it crosses the 0.07 cm/sec (0.03 in/sec) settling velocity line at about 60 percent recovery.

This model settling velocity corresponds to a prototype settling velocity of:

0.07 cm/sec x 2.65 = 0.185 cm/sec (0.073 in./sec)

From Figure 76, this gives a particle size of 0.05 mm (0.002 in.) for SG = 2.65 material.

Another approach to selecting the chamber size would be to decide to use 3 l/sec (1.106 cfs) in the required recovery curve, either Figures 77, 78, or 79. Working with Froude's Law, the scale can be found:

$$\lambda^{5/2} = Q_p/Q_m = \frac{1,460}{4} \times \frac{1}{3} = 121.7$$

$$\lambda = 6.83$$

The corresponding chamber diameter would be:

$$d = 6.83 \times 0.94 \text{ m} = 6.4 \text{ m (21 ft)}$$

The velocity scale becomes:

$$\frac{1}{\sqrt{\lambda}} = \frac{1}{\sqrt{6.83}} = \frac{1}{2.61}$$

It is now possible to prepare a new operating curve for this unit at the 365 l/sec (12.89 cfs) discharge by taking recovery rates from the 3 l/sec (0.8 gps) in either Figures 77, 78, or 79 and multiplying the corresponding settling values by 2.61 to find the settling velocities that would be recovered.

The dimensions of the individual swirl units would be $\lambda/4$, or 6.83/4, or 1.70 times for each dimension shown in Figure 83, since the dimensions shown in Figure 83 are for a model scale of 1:4.

Keeping the same scale relations, similar calculations could be carried out for the 5 and 7 l/sec (1.32 and 1.85 gps) lines on Figures 77, 78, and 79. The resulting three operating curves could then be interpolated at selected settling velocity values to yield data that could be plotted in the same manner as shown in Figures 80, 81, and 82 but for the chosen chamber size.

In addition to the settleable solids, a considerable quantity of light suspended or colloidal solids is present in storm erosion runoff. It is anticipated that none of these lighter solids would be removed in the swirl unit, but there would be almost complete removal of such solids in the second retention pond if sufficient settlement time occurred between storms.

Assessment of Retention Volumes:

The volume per storm was determined to be 8,420 cu m (297,226 cfs). Using an underflow drain-off rate of 14 percent, the volume to be handled in the solids basin is:

8,420 cu m (297,226 cu ft) x 0.14 = 1,180 cu m (41,696 cu ft) while the stormwater retention pond would be:
8,420 cu m (297,226 cu ft) - 1,180 cu m (41,696 cu ft) = 7,240 cu m (255,830 cu ft).

These pond volumes are sized to retain all of the treated runoff from the design storm. In practice, most ponds are designed to allow flow-through for the normal runoff before construction development. For the 80.9 ha (200 ac) site, with a runoff coefficient of 0.2, after full development the outflow would be 566 l/sec (20 cfs). Various methods are available for computing the required storage based on an outflow of 566 l/sec (20 cfs).

An estimate of the volume of settled material to be expected can be obtained from information provided in a study for APWA by the firm of Beak Consultants, Ltd. (8) Among figures quoted for suspended solids in stormwater, these settleable solids vary from 0 to 7,640 mg/l, with an average of 687 mg/l. The concentration of solids can vary widely and is dependent upon the character and the use of the land from which the storm flow is generated. Using an average value of 700 mg/l, an estimate of the settleable solids per storm is:

$$V = 8{,}420 \text{ cu m} \times 1{,}000 \frac{1}{\text{cu m}} \times 700 \frac{\text{mg}}{1} \times \frac{\text{kg}}{1{,}000{,}000} \times \frac{\text{cu m}}{1{,}600 \text{ kg}}$$

$$= 3.68 \text{ cu m } (130 \text{ cu ft})$$

On an annual basis the volume of settleable solids is:

$$V = 314{,}000 \text{ cu m} \times 1{,}000 \frac{1}{\text{cu m}} \times 700 \frac{\text{mg}}{1} \times \frac{\text{kg}}{1{,}000{,}000 \text{ g}} \times \frac{\text{cu m}}{1{,}600 \text{ kg}}$$

$$= 137, \text{ cu m } (4{,}841 \text{ cu ft})$$

Assume that 100 percent of all settleable solids will be retained in the ponds.

Temporary Facility at Construction Site

Another application of the swirl separator is as a temporary facility for erosion control at a construction site. For this purpose the foul sewer underflow, conveying most of the settleable solids, would discharge into a soil collector pond and the overflow would discharge into a drainage ditch or channel, or flow directly into a watercourse.

The riser pipe, shown in Figure 83 as 0.67 m (2.2 ft), could be changed to 0.61 m (2 ft) to utilize standard size pipe. The clarified overflow outlet could be attached directly to the underside of the chamber and could be made rectangular in shape. Dimensions of 0.61 m (2 ft) wide and 0.22 m (0.75 ft) high would provide a waterway having an area equivalent to the riser pipe. The underflow or foul outlet could likewise be made in rectangular or square shape, attached to the bottom of the box. The outlet should probably be at least 15 cm (6 in.) square to prevent problems with clogging. The outlet could terminate at the outside wall of the chamber, with a 15 cm (6 in.) standard pipe flange for attaching the pipe to convey flow to the solids basin.

Assume the following conditions:

Site area tributary to chamber is 3.12 ha (8 ac)
Runoff coefficient C is 0.4
Time of concentration is 15 min
Rainfall intensity is 6.35 cm/hr (2.5 in/hr)

Again, using a set of simplified calculations, for

Q = CiA:

= 0.4 x 6.35 cm/hr x 3.12 ha
= 8 cm/ha/hr
= 8 x 27.8 = 222.4 l/sec

Q = 0.4 x 2.5 in/hr x 8.0 ac
= 8 cfs

From Figures 80, 81, and 82 it is apparent that the largest allowble flow through one chamber is 222.4 l/sec (8.0 cfs). Therefore, under the above assumed conditions the largest site that can be served by one chamber is 3.12 ha (8.0 ac). The greatest recovery of solids will occur if a 14 percent draw off (Figure 82) is used rather than 10 percent (Figure 81) or 5 percent (Figure 80).

From Figure 82, the percentage of various size solids to be recovered will be as follows:

Size Solids		Percentage Recovery
mm	in.	
0.35	0.014	85
0.30	.0.012	73
0.25	0.009	53
0.18	0.007	37
0.16	0.006	31

A 14 percent draw off means that this percentage of the peak flow will pass through the underflow outlet to the soil collection pond. This amounts to 0.14 x 222.4 l/sec (8 cfs) = 32 l/sec (1.1 cfs). The head or depth of water above the underflow outlet will be 0.61 m (2 ft) when the outlet weir starts overflowing. At peak flow this head may increase to 0.76 m (2.5 ft). Approximate hydraulic computations indicate that this head is too small to permit use of a 10 cm (0.33 ft) diameter underflow outlet. If an outlet pipe 15 cm (6 in) in diameter is used, the head is sufficient to force the flow through about 15 m (50 ft) of outlet.

To meet the recovery performance shown in Figure 82 it is necessary to keep the underflow to about 32 l/sec (1.1 cfs). To prevent decreasing the rate of underflow due to backwater, the maximum water level in the soil collection pond should be below the top of the underflow pipe. The most practical way of regulating the underflow rate would be to provide a shear gate at the outlet pipe and to determine the actual setting of the gate from measurements of the volume in the collection pond during actual storm conditions.

A further design consideration is the volume of the soil erosion collection pond. Obviously the foul sewer discharge from the swirl chamber underflow will outlet into the selected drainage ditch or the designated watercourse during a storm period. However, whenever the rate of flow into the swirl chamber is not sufficient to fill the chamber to the overflow weir crest, all of the storm runoff will discharge through the foul sewer into the soil collection pond. Thus, the rate of flow into the pond will vary from 0 to 32 l/sec (1.1 cfs). Hence, if it is desired to provide storage for all underflow in a 4-hour storm the required storage would be 32 l/sec (1.1 cfs) x 4 x 60 x 60, or 447 cu m (15,800 cu ft). This would require a pond 1.2 m (4 ft) deep and 18.9 m (62 ft) square. If a 2-hour detention time is considered adequate to settle out the suspended solids, then the depth could be reduced to 0.61 m (2 ft) or the surface dimensions of the pond reduced. An overflow weir should be provided to pass 32 l/sec (1.1 cfs) when the pond becomes filled to the designed depth.

The chief advantage of such a temporary facility is that it is portable and has no mechanical parts. Thus, the chamber could be moved about on the construction site, as required, or moved to other sites. Multiple units could be used to meet requirements of larger sites or to remove higher percentages of suspended solids.

CONSTRUCTION COST

Site preparation is minimal, consisting of the leveling of about 25 sq m (24 sq yd) for each 4 m (12 ft) diameter unit

The unit will ordinarily be fabricated of steel off-site and delivered intact. This site work would consist of leveling the unit and connecting the inlet and two discharge lines.

The cost estimate is as follows:

Cost Estimate Per 4 m (12 ft) Diameter Unit

Site preparation	$ 400
Material and fabrication	6,500
Setting and field connection	1,100
Sub Total	8,000
Engineering and contingency at 25%	2,000
Total	$10,000

PROTOTYPE INSTALLATION

A prototype was tested in South Carolina at a newly constructed highway site which was known to be actively eroding and contributing abundant sediment to receiving waters adjacent to the site.

A standard unit of 3.6 m (12 ft) was constructed. The area served was 1.5 ha (2.1 ac) with an estimated peak discharge for a one in two years precipitation event of 165 l/sec (5.8 cfs).

The calculated efficiency of the unit based upon a grain size analysis of a composite of six samples from the bed material at the base of the drainway was 98.7 percent with 10 percent foul flow.

During the desired test periods, it did not rain. Tests were eventually run with tank trucks discharging water to synthesize runoff from the roadway. It appears that there were no erosion control devices to protect the swirl and thus the device received a concentrated flow of sediment with particles as large as gravel. The unit essentially failed under the test conditions. The heavier larger size particles settled rapidly on the floor of the swirl unit and the flow was of such a short duration that the solids were not moved to the foul outlet. The test procedures used indicated essentially the same particle size distribution in the overflow and the foul outlet.

It is apparent from the test results that there was a large bedload flow and that suspended solids as they entered the unit joined the bedload. Thus, in the absence of moving the bedload to the foul outlet, only minimal particle size variances were noted. The outlet was frequently clogged.

The test results are interesting and point towards the need to use the unit as a part of an erosion control system. Prior to the publication of the report on the test facility, additional tests are planned on another site.

Section VIII

CHARACTERIZATION OF SOLIDS

The efficiency of secondary flow motion devices is essentially determined by the degree that solids are removed from the influent. The devices have been considered efficient in the model studies when either standard or arbitrary amounts of the particular size and weight of material have been separated to the foul outlet. Thus for the purpose of the hydraulic model studies it was necessary to 1) select a "typical" solids composition for each type of waste flow to be treated, and 2) select an appropriate synthetic medium to represent the solids. Due to problems of scale up and handling, the use of actual waste streams was not practical.

In addition to the assumptions that were made from the published literature, the firm of Beak Consultants, Ltd. was engaged to conduct additional literature and laboratory tests to characterize solids in combined sewer overflows, stormwater and sanitary sewage (8).

In this section the basic assumptions used for each type of pollutant stream will be reviewed and compared to the basis of efficiency established for particular devices.

COMBINED SEWER OVERFLOWS

Studies by Others

In common with other non-industrial pollutional loads, combined sewer overflows have been found to vary widely in the concentrations and composition of solids and pollutants. Table 33 reports overall results and cummulative particle size distribution and results are shown in Figure 84 for comparison with some other waste streams.

Total suspended and settleable solids concentrations found by various investigators is shown in Table 34.

Table 35 presents the results of a study of the size of solids found on streets, potential solids in stormwater runoff. The average distribution is also shown in Figure 84. The solids particles are larger than found in combined sewer overflows and should be more susceptible to treatment than combined sewer overflows.

The settling velocity of the organic and grit solids for combined sewer overflows was assumed to be as represented in Figure 85.

Table 33
Particle Size Distribution of Suspended Solids in Combined Sewer Overflow

Source of Figures (reference number)	Size Range (microns)	Distribution (percent)
Envirogenics Co.[24] San Francisco, Cal	>3,327	5.1
	991-3,327	8.8
	295-991	15.9
	74-295	21.8
	<74	48.3
Meridian Engineers [25*] Lancaster, Pa.	>9,525	1.77
	4,760-9,525	1.06
	2,000-4,760	1.40
	1,190-2,000	1.88
	590-1,190	3.10
	420-590	2.78
	210-420	7.01
	149-210	5.19
	74-149	20.1
	44-74	23.8
	< 44	31.91

* The material tested represents those solids retained in a catch basin. Sampling took place the week following the storm the week following the storm event. Thus, results are not directly *applicable to all solids in combined sewer overflows.* The particle sizes could be higher than in the actual flow as some fractions of the smaller size ranges could have been carried through the basin.

Table 34
Solids Concentrations in Combined Sewer Overflows

Source of Figures (Reference Number)	Settleable Solids ml/l			Settleable Solids mg/l			Total Suspended Solids mg/l			Volatile Suspended Solids mg/l		
	Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.	Max.	Min.	Avg.	Max.	Min.
Envirogenics Company[24]	2.58	4.0	0.05				67.6	426	4	52.2	373	4
Rex Chainbelt, Inc.[26]												
a) Extended overflows							166 ± 26			90 ± 14		
b) First flushes							522 ±			308 ±		
(95% confidence level for a & b)							150			83		
Hydrotechnic Corporation[27]												
a) Spring storms (1971)	6.98	14.0	1.5				411	976	177			
b) Summer and fall storms (1970)	5.26	19.0	0.2				234	1.560	28			
Envirogenics Company[28]												
Winter 19681969												
a) Start of storm				178.2	488	28	230.5	502	56	166.2	311	51
b) 3 hours after start				77.3	142	0	106.3	186	47	91.7	186	26
c) 12-18 hours after start				112.2	210	28	145.5	241	30	99.5	221	26
Symposium on Storm and Combined Sewer Overflows[29]												
Portland, Oregon	3.1	5.0	1.5				146	325	70	90	166	57
Milwaukee, Wisconsin												
a) Extended overflows							133			58		
							174			87		
b) First flushes							330			221		
(95% confidence level)							848			495		
Detroit Michigan												
a) 1968 average of daily grab samples — 59 locations								1.350	53			
b) 1969 average of daily grab samples — 59 locations								1.005	70			
Bucyrus, Ohio — 3 sewer locations[25]							533	2.440	20	182	440	70
							430	990	90	238	570	80
							477	1.050	120	228	640	70
Engineering Science, Inc.[30]												
San Francisco, Selby Street		145.0	<0.3		1.067	27		1.260	24		886	4
Laguna Street		40.0	2.0					483	53		264	28
Benzie and Courchaine[31]												
Detroit, Michigan (1964)							150	1.398	23			
Burm et al[32]												
Detroit, Michigan (1965)				238	656		274	804		117	452	
Dunbar and Henry[33]												
Buffalo, New York								1.220	172			
Buffalo, New York								544	158			
Buffalo, New York								436	126			
Detroit, Michigan							250					
Toronto, Ontario								930	130			
Toronto, Ontario								580	17			
Welland, Ontario								426	168			
Weibel et al[34]												
Cincinnati, Ohio (1962-1963)							210	1.200	5	53	290	1

Table 35
Particle Size Distribution of Solids — Selected City Composites

Particle Size Range (microns)	Distribution (Percent by Weight) Milwaukee	Bucyrus	Baltimore	Atlanta	Tulsa
>4,800	12.0	—	17.4	—	—
2,000-4,800	12.1	10.1	4.6	14.8	37.1
840-2,000	40.8	7.3	6.0	6.6	9.4
246-840	20.8	20.9	22.3	30.9	16.7
104-246	5.5	15.5	20.3	29.5	17.1
43.104	1.3	20.3	11.5	10.1	12.0
30-43	4.2	13.3	10.1	5.1	3.7
14.30	2.0	7.9	4.4	1.8	3.0
4.14	1.2	4.7	2.6	0.9	0.9
<4	0.5	—	0.9	0.3	0.1

Note Columns may not total 100% due to rounding
Source URS Research Company (35)

Assumptions for Swirl Regulator and Helical Bend Separators

On the basis of available data, the concentrations, by size and specific gravity shown in Table 36, were selected. This selection is shown graphically in Figure 86.

A design objective for both separator designs was for the capture of all grit particles of 0.35 mm diameter. Capture of this amount will result in the separation of various amounts of other sized particles. From Figures 85 and 86 it can be readily determined that with the 100 percent capture of 0.35 mm grit, 82 percent of the grit and 68 percent of the organics will be captured at 100 percent efficiency.

SANITARY SEWAGE

Various studies have indicated a rather wide range of particle sizes in sanitary sewage. Table 37 indicates the results of five studies. In Table 38 the classification of solids is shown by total and volatile suspended solids as determined by a variety of investigators.

Settling velocity tests were made at the Northeast Water Pollution Control Plant in Philadelphia, Pennsylvania. Figure 87 indicates the average values found for the three samples. The percent settleable solids ranged from 63 to 84 percent and the median settling velocity observed was 0.054 cm/sec (0.0017 ft/sec).

The conventional method for establishing efficiency of primary settling facilities has been to set the overflow rate, i.e., the liters per day per square meter (gal/day/sq ft) with a minimum depth. Although performance requirement varies widely, 60 percent suspended solids removal is assumed as normal.

Available data on the mechanical analysis of grit removed from representative wastewater treatment plants were compared to establish criteria for grit sizes for this study.

Table 36
Specific Gravity, Size, and Concentration of Settleable Solids for Combined Sewer Overflows

Material	Specific Gravity	Concentration (mg/l)	Particle Size	Particle Size Distributed					
1) Settleable excluding grit	1.05—1.2	200—1550	0.2—5 mm	Particle size (mm)	0.2	0.5	1.0	2.5	5.0
				% by weight	10	10	15	25	40
2) Grit	2.65	20—360	0.2—2 mm	Particle size (mm)	0.2	0.5	1.0	1.5	2.0
				% by weight	10	10	15	25	40
3) Floatable solids	0.9—0.998	10—80	5—25 mm	Particle size (mm)	5	10	15	20	25
				% by weight	10	10	20	20	40

Table 37
Particle Size Distribution of Suspended Solids in Sanitary Sewage

Source of Figures	Particle Size Range (microns)	Distribution (percent)
Hunter & Heukelekian[36] (average of two studies) a) Winter-spring 1959 b) Fall-winter 1959-1960	>100 (Settleable)	49.4
	1 — 100 (supracolloidal)	31.4
	0.2 — 1.0 (colloidal)	19.2
Huekelekian & Balmat[37]	>100	47.0
	1 — 100	34.0
	0.2 — 1.0	19.0
Meridian Engineers[25]*	>1,190 (0.047 in.)	4.42
	590 — 1,190	1.38
	420 — 590	3.46
	210 — 420	3.09
	<149	86.9
Painter, Viney & Bywaters[38]	>100	37.1
	1 — 100	44.8
	0.2 — 1.0	18.1

*Note Remainder passed No 200 mesh

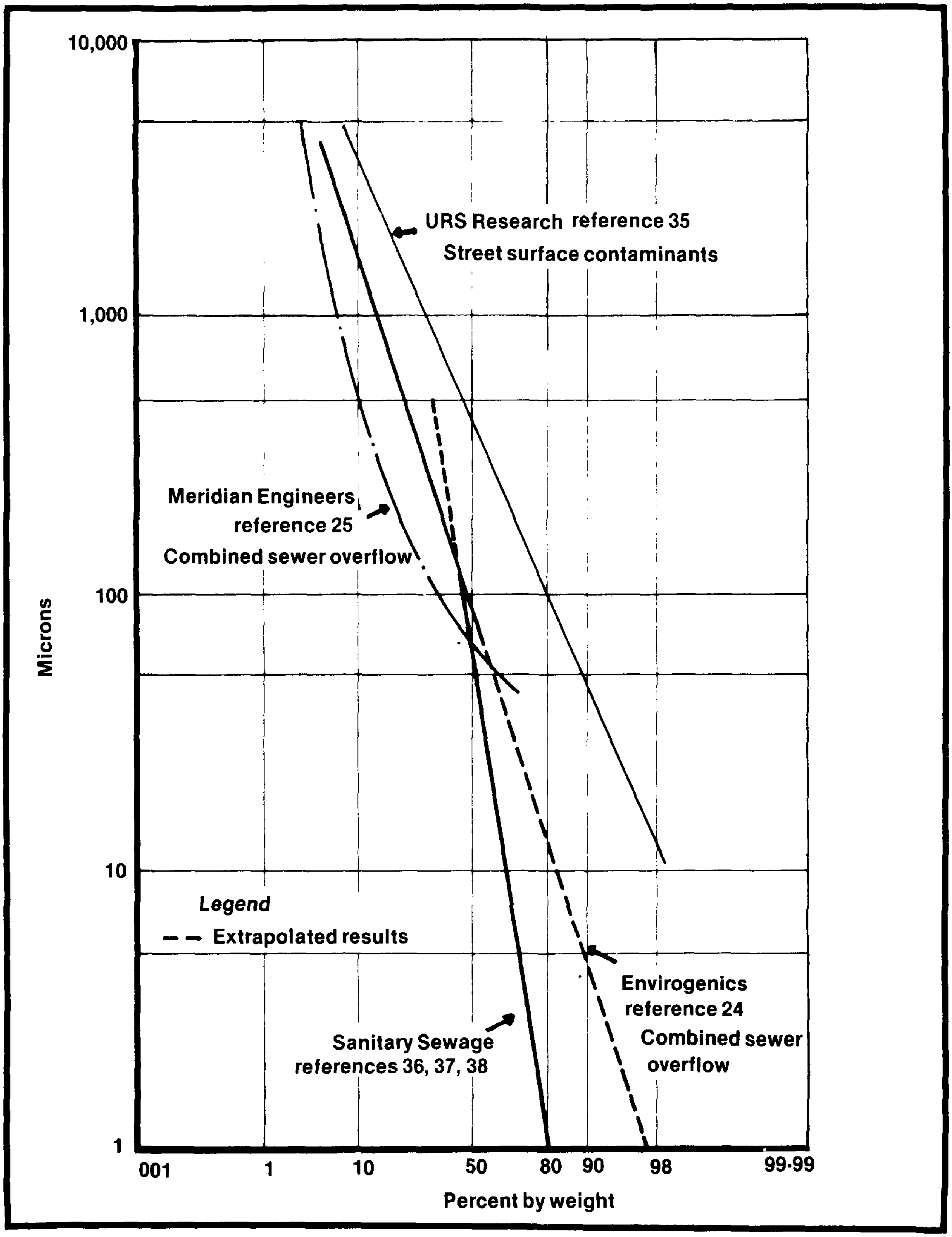


Figure 84 Particle Size Distributions of Some Waste Stream Solids

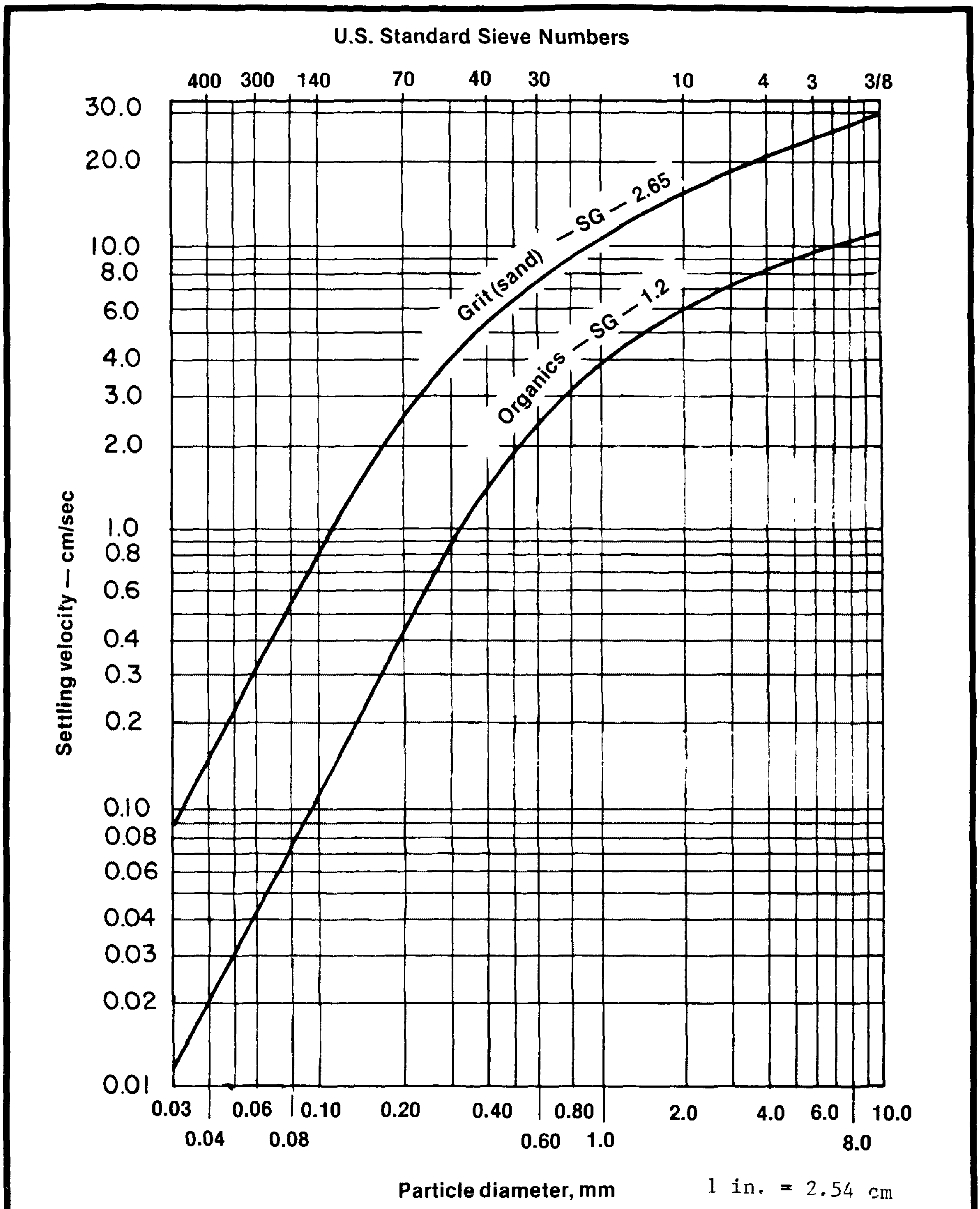
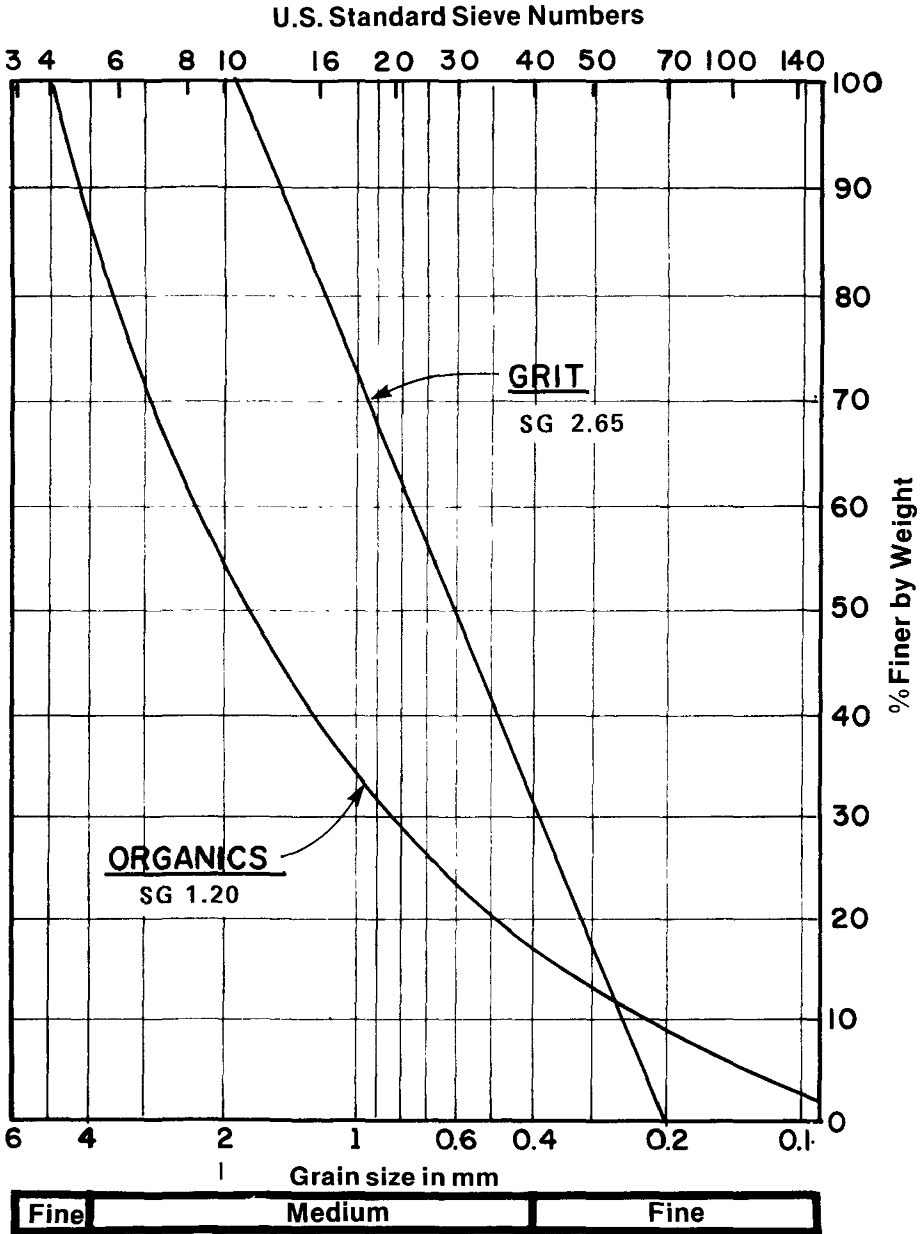


Figure 85 Particle Settling Velocities for Grit and Organic Material in Still Water



U.S. SIEVE SIZE	SIZE mm	SIZE in.	% FINER BY WEIGHT GRIT	% FINER BY WEIGHT ORGANICS
4	5.0	(0.020)	100	100
10	2.0	(0.08)	100	53
20	0.84	(0.034)	63	31
40	0.42	(0.017)	31	17
50	0.30	(0.012)	18	14
70	0.20	(0.008)	0	10

Figure 86 Typical Gradation for Grit and Organic Material

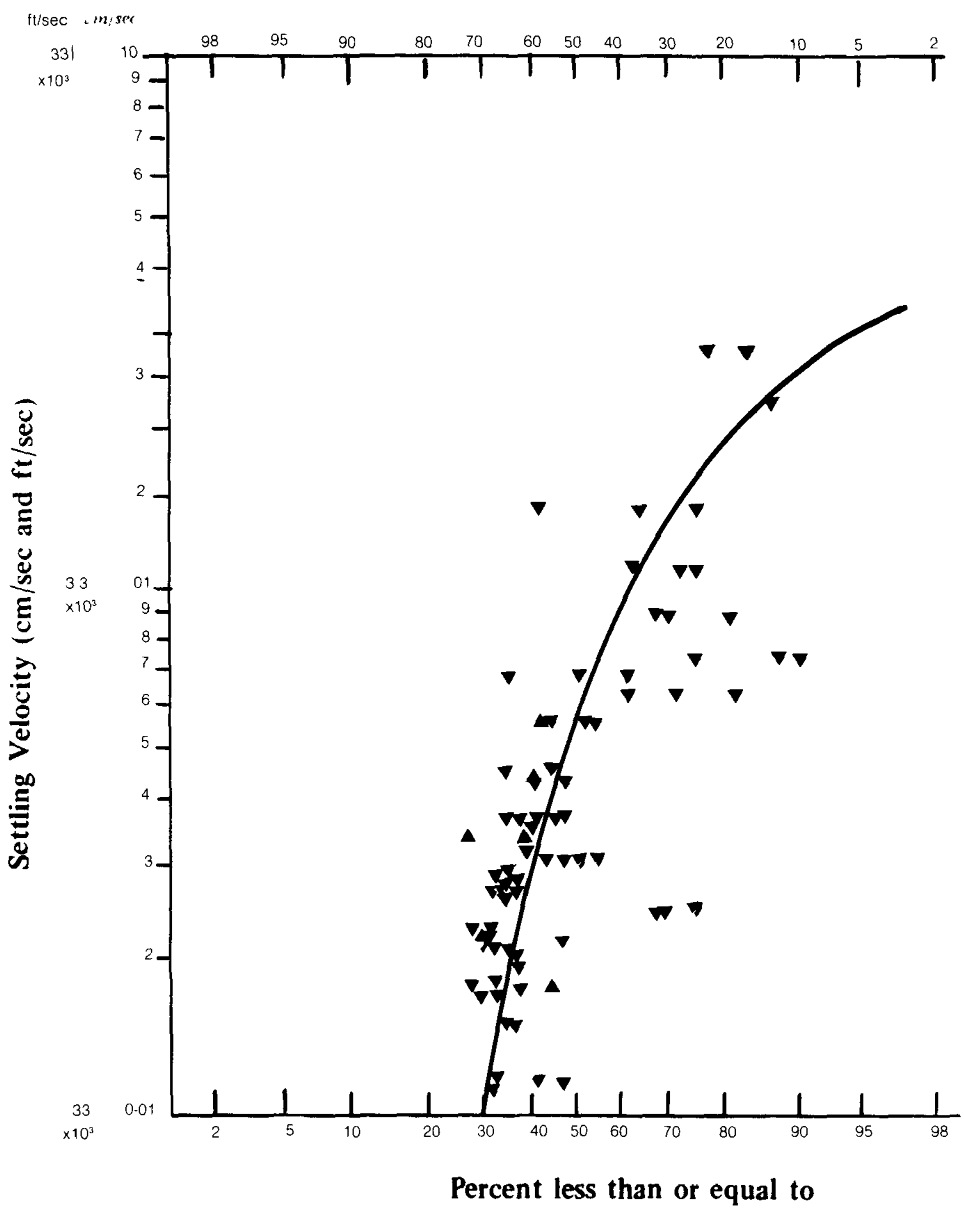


Figure 87 Settling Velocity Distribution of Solids in Sanitary Sewers

Table 38
Sieve Analysis of Samples from Grit Chambers

Sieve Designation		Percentage Finer by Weight							
		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
		Green Bay	Kenosha	Tampa	St. Paul	St. Paul	Winnipeg	Winnipeg	Denver
	U.S.	Wis.	Wis.	Fla	Minn.	Minn.	Manitoba	Manitoba	Colo
mm	Sieve No.	1/	1/	1/	1/	1/	2/	2/	2/
6.3	1.2 mm (0.5 in.)								94.9
4.75	4				99.0	93.0		77.1	
3.35	6								89.2
2.36	8				95.0	80.0		46.3	
2.00	10	96.3	88.0				96.9	38.9	75.2
0.850	20	90.9			88.0	47.0	83.2	14.7	
0.600	30								6.7
0.425	40	80.2	30.0				44.3	6.3	
0.300	50	70.4		97.7	80.0	33.0	19.2	3.5	
0.212	70	48.3					4.4	1.3	
0.180	80		5.0						
0.150	100	21.8		40.7	3.0	0.1			0.7
0.075	200	3.9		0.5					

Notes:

1/ adapted from data in ASCE Manual No 36, 1959 edition
2-8/all data adapted from correspondence, 1973
(4) upper range
(5) lower range
(6) inlet end
(7) outlet end

Data from eight existing plants located in the United States and Canada are tabulated in Table 38.

The original data were adjusted to correspond with the U.S. sieve numbers and to indicate percent of weight finer than given sieve sizes. These sieve analyses are shown graphically in Figure 88.

Most of the grit particles in the samples are larger than 0.2 mm. This may be explained by the fact that most grit chambers are designed to remove only grit greater than 0.2 mm size. A notable exception is the sample from Tampa where 65 percent of the sample is finer than 0.2 mm.

Based on the foregoing, a "typical grit" for purposes of this study was assumed to range in size from 0.2 mm to 2.0 mm, with a gradation corresponding to a straight line on a mechanical analysis graph.

The assumed gradation is given in Table 39 and shown graphically in Figure 89.

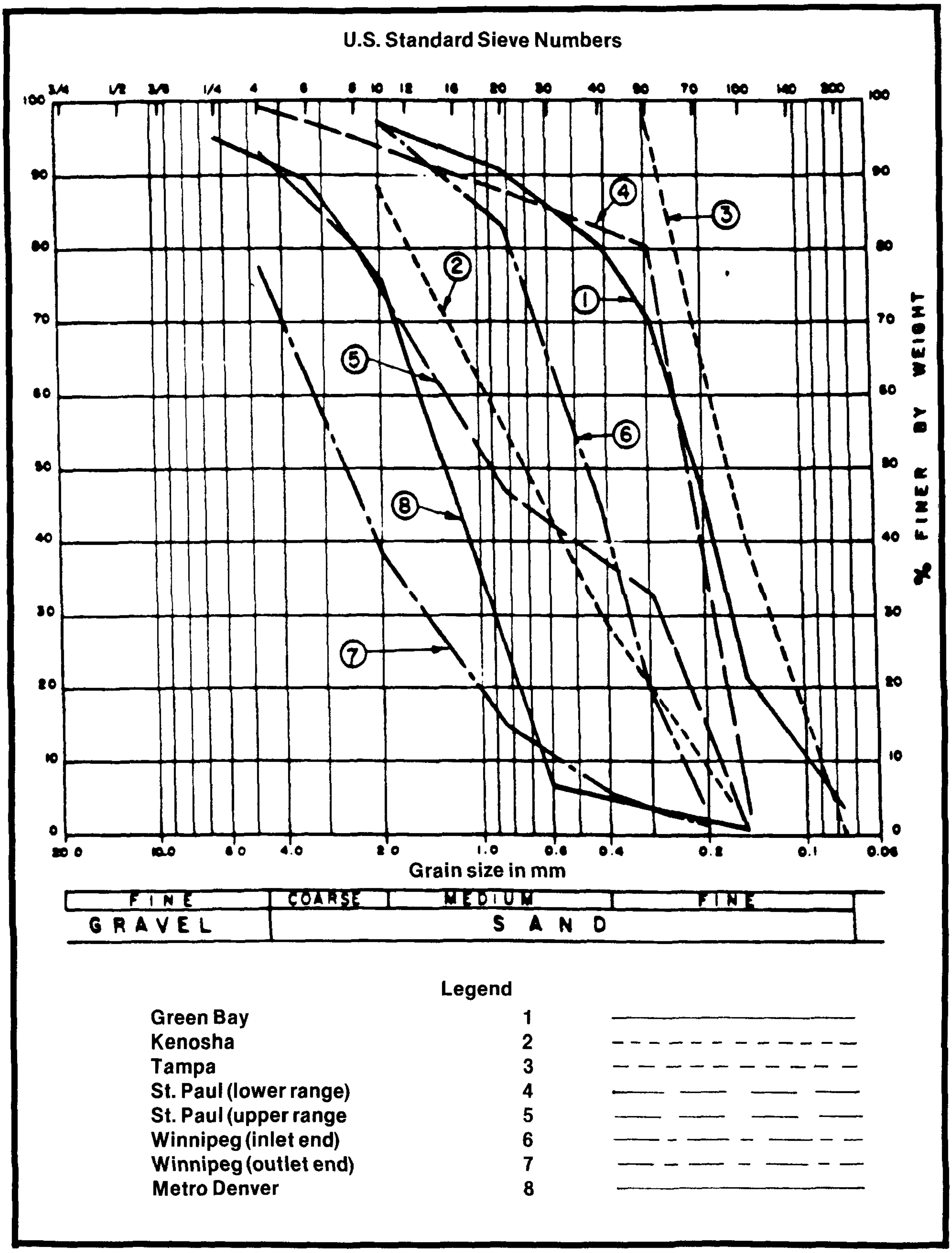


Figure 88 Gradation Curves of Samples from Grit Chamber

Table 39
Typical Grit Gradation

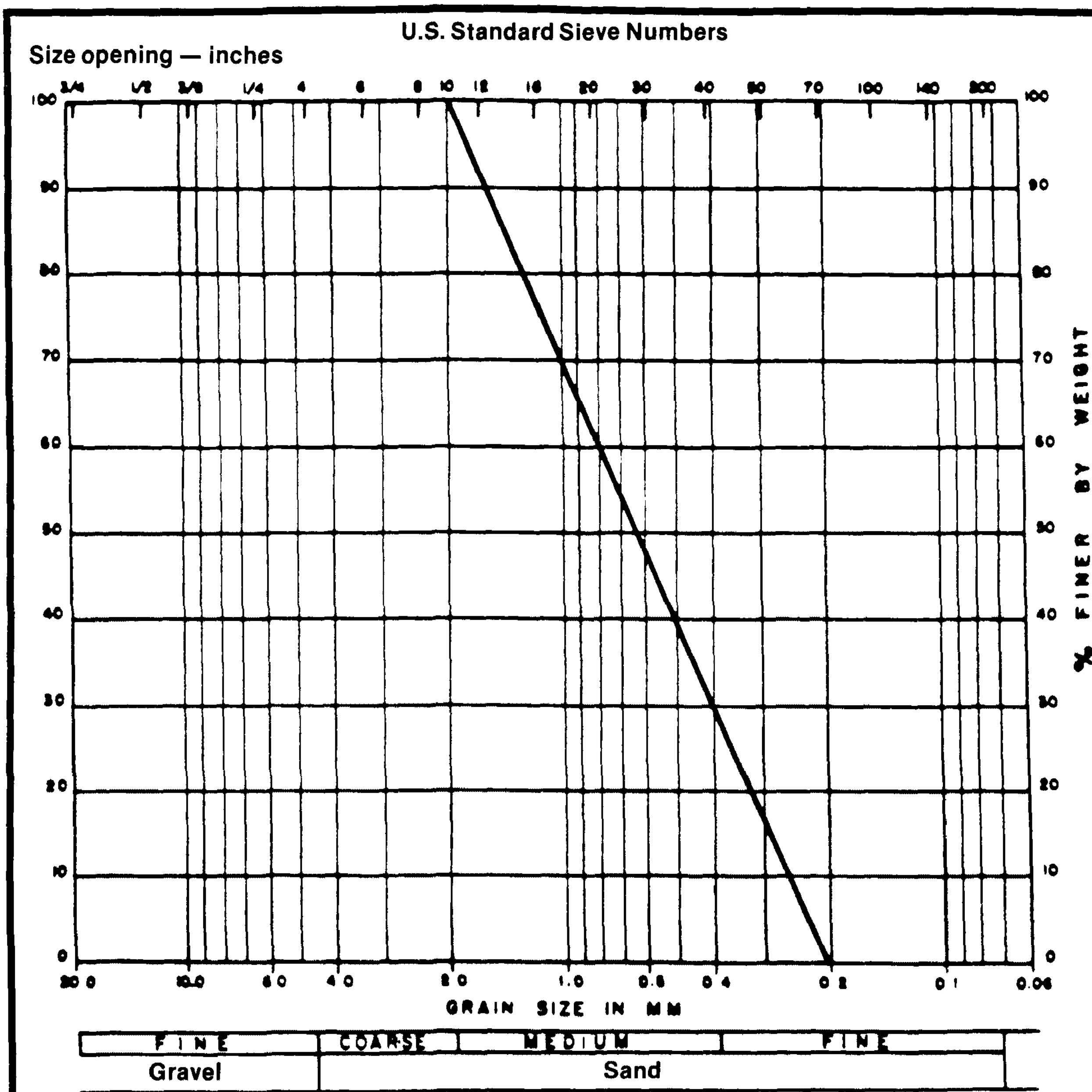
Size mm	Size in.	U.S Sieve No.	% Finer by Weight
2.000	0.080	10	100
0.850	0.034	20	63
0.420	0.017	40	31
0.300	0.012	50	18
0.212	0.008	70	0

Specific gravity of the typical grit is assumed to be 2.65

EROSION PRODUCTS

The types of solids found in construction site stormwater runoff is even more site-specific than the other pollution streams which have been discussed. Hazen (19) has reported that settling velocities of soil materials can range from 0.015 cm/sec (0.0005 ft/sec) for silt 10 microns in size to 10 cm/sec (0.33 ft/sec) for coarse sand, 1,000 microns in size. The smallest size particles that the swirl unit can remove at its design flow is 43 microns in diameter with a Specific Gravity of 2.65 and a settling velocity of 0.14 cm/sec (0.0046 ft/sec). Larger particles of lighter specific gravity will also be removed as described in Section VII.

A standard of efficiency for the swirl unit is not feasible. Rather, the amount of polishing of flow can be determined based upon an analysis of the soil particles which will reach the unit.



Size mm	Gradation U.S. Sieve No.	% Finer by Weight
2.000	10	100
0.850	20	63
0.425	40	31
0.300	50	18
0.212	70	0

Figure 89 Gradation Curve of Typical Grit

Section IX

GLOSSARY

Combined Sewer -- A pipe or conduit which collects and transports sanitary sewage, with its component commercial and industrial wastes and infiltration and inflow during dry-weather conditions, and which, in addition, serves as the collector and conveyor of stormwater runoff flows from streets and other sources during precipitation and thaw periods, thus handling all of these types of wastewaters in a "combined" facility.

Concentrate -- The portion of the inflow directed to the interceptor sewer which carries the bulk of the settleable solids.

Concentrate Outlet -- The outlet in the floor of the chamber in which the concentrate enters the foul sewer. Also see Foul Sewer.

Concentric Skirt -- A vertical sheet or panel, constructed in circular form concentric with the outer diameter and the overflow downshaft pipe in a swirl chamber for the purpose of separating flow zones and acting as a suppressant of any short-circuiting of flow patterns or the overflow of floating solids with the effluent.

Deflector -- A plate or plane structure which diverts and directs flows in a swirl separator chamber into desired patterns and thus prevent flow kinetic conditions which would interfere with optimum swirl motion.

Depth of Chamber -- The vertical distance between the floor level of the swirl separator and the crest of the overflow weir.

Diameter of Swirl Chamber -- The internal diameter of the separator chamber.

Dip Plate -- A vertical plate or baffle which is partially immersed in flowing liquid in a manner that will prevent the discharge of surface or floating materials over an outlet weir in the swirl and helical bend regulator/separator.

Dry-Weather Flow -- The flow in the combined sewer during periods without precipitation, normally sewage and groundwater infiltration.

Erosion -- The washing or scouring action of stormwater on the land, resulting in the displacement and movement of grit, silt and other indigenous solids with the wastewater flow; the type of solids which are intended to be removed from the flow by the swirl separator chamber.

Exterior Liquid Mass -- The liquid induced to flow in the outer zone of the circular swirl separator chamber, by use of the skirt, wall structural configuration or other built-in devices, where the higher velocities of flow produce a longer liquid trajectory which allows adequate time for heavier solids to settle to the floor of the chamber.

Floatables -- Solid and liquid matter which is lighter than water and float on the surface of the wastewater flowing in the swirl and helical regulator/separator.

Floatables Trap -- A device or structural configuration in a swirl separator chamber which intercepts floatable solids, prevents them from overflowing from the chamber with clarified wastewater, and retains these materials at a desired location until removed and disposed of by predetermined means.

Flow Spoiler -- Vertical energy dissipating baffle or plate installed on the weir disc or elsewhere in the swirl separator chamber for the purpose of preventing excessive flow disturbances and dampening the development of free vortex flow patterns and other undesired flow conditions in the chamber.

Foul Sewer -- A sewer line, from the bottom of the swirl separator chamber to some point of discharge to an interceptor sewer, a catchment basin or other point of solids disposal, installed for the purpose of drawing off the solids concentrate flow from the swirl chamber due to the recovery efficiency of the device.

Grit -- Solids, predominantly mineral in character, in the combined sewer flow which are larger than 0.2 mm (0.008 in.) and with a specific gravity 2.65.

Gutter -- A structural configuration in the floor of the swirl separator which provides a channel for the desired flow of sanitary wastewater during dry-weather conditions from the chamber inlet to the foul sewer (concentrate) outlet, and during wet-weather for conducting the foul concentrate to the foul sewer.

Helical Bend -- A physical configuration of a pipe or open channel which results in a bend or radius through which a liquid flow occurs in a manner that produces helical, or secondary flow phenomena, inducing the rapid separation of solids from the liquid and the deposition of the solids along the inner diameter of the radius; in the study, the total helical bend combined sewer overflow regulator/separator consists of a transition section, a straight section, and the bend section.

Helical Flow -- The pattern of liquid flow induced by the helical bend combined sewer overflow regulator/separator characterized by a helical configuration, or secondary motion, created in the liquid flow.

Hydraulic Head Loss -- The lowering of the hydraulic grade line through a pipeline, device, chamber or other facility, due to dynamic conditions which produce friction, turbulence or other conditions that are translated into loss of pressure, or head, or free water gradient surface level.

Inlet Baffle -- A structural plate installed from the inlet to the overflow weir for the purpose of producing or inducing the desired flow pattern in a swirl chamber; a device to serve as a guide for the incoming flow and to place it in circulatory action to take full advantage of the swirl secondary flow pattern in the chamber.

Inlet Size -- The diameter or square dimensions of the sewer which enters the swirl separator at its floor level and thereby, serves to create the flow pattern which produces the solids-liquid separation which the chamber is intended to induce.

Interior Liquid Mass -- The liquid induced to flow in the inner zone of the circular swirl separator chamber--by use of the weir skirt, wall structural configuration or other built-in devices which induce exterior liquid mass flows--where the lower velocity permits lighter solids to settle out of the wastewater flow and to deposit on the chamber floor and to be drawn to the foul sewer outlet. The principle of the swirl separator is to organize the flow patterns and cause the liquid mass to pass through the exterior and interior liquid mass zones to optimize solids separation and removal.

Long-Flow Pattern -- The swirl flow pattern through the swirl separator, induced by proper baffling which causes the liquid to traverse the circular chamber more than once, and prevents the incoming flow from being diverted or short-circuited directly to the overflow weir, thereby inducing the solids to discharge into the foul gutter and foul sewer outlet.

Organic Solids -- Solids of a non-grit, or lighter weight, contained in the combined sewer flow, which can decompose and become oxygen-demanding in receiving waters.

Overflow Weir -- The structural member of the swirl chamber, which is built as a central circular wall with a proper form of overflow edge over which the clarified wastewater can discharge to the downshaft outlet leading to receiving waters or to holding or treatment facilities.

Regulator -- A device or apparatus for controlling the quantity of sewage and stormwater admitted from a combined sewer collector sewer into an interceptor sewer, pumping or treatment facility. The secondary flow motion regulators described in this manual also improve the quality of the overflow to receiving waters.

Scaling -- The principle of ascertaining dimensions and capacities of hydraulic model test units and mathematical analysis systems to evaluate the performance of swirl chambers, and to scale up such sizes to provide actual field design and construction criteria or parameters.

Scum Ring -- A circular plate or baffle encircling the overflow weir, located at a predetermined distance from the weir and at a depth that will cause it to retain floatables and scum and prevent them from passing over the weir crest with the clarified liquid.

Settleable Solids -- That portion of the solids contained in the wastewater flow into a swirl separator chamber which will subside and be collected in the chamber due to gravity and other liquid-solids kinetic conditions induced by the controlled swirl flow pattern. (Note: Not all suspended solids are settleable solids, nor are so-called colloidal solids or other finely dispersed solids settleable solids.)

Spoiler (Energy Dissipating Baffle) -- A plate or structural plane constructed from the scum ring to the downshaft on the weir plate in a swirl separator chamber for the purpose of preventing or dampening the development of free vortex flow conditions, minimizing agitation and rotational flow over the discharge weir, and increasing the capacity of the downshaft.

Static Regulator -- A regulator device which has no moving parts, or has movable parts which are insensitive to hydraulic conditions at the point of installation and which are not capable of adjusting themselves to meet varying flow or level conditions in the regulator-overflow structure.

Storm Frequency -- The time interval between storms for which storm sewers and combined sewers, and such appurtenant structures as swirl separator chambers, are designed to handle or treat without flooding and or for desired treatment efficiency.

Straight Section -- The part of the helical bend combined sewer overflow regulator/separator structure which precedes the bend section and delivers the flow uniformly and without velocity interferences into the helical section. In the studies of the helical bend principle, it was determined that the straight section having a length of five times the diameter of the sewer pipe will be required for effective solids recovery in the helical bend section.

Spillway Channel -- The channel or conduit which receives the overflow effluent from the helical bend weir section and delivers it to a pipe or conduit leading to receiving waters, or facilities for the retention and/or treatment of the clarified wastewater discharge.

Suspended Solids -- 1) The quantity of material deposited when a quantity of water, sewage, or other liquid is filtered through an asbestos mat in a Gooch crucible or a 0.35-0.45 micron millipore fiberglas filter. (39) 2) Solids that either float on the surface of, or are in suspension, in water, wastewater, or other liquids, and which are removable by laboratory filtering as described above.

Swirl Chamber -- A cylindrical tank or chamber, in which the shape, method of inflow and overflow, and internal appurtenant structures induce a secondary motion flow pattern which produces the desired separation of solids from the liquid flow.

Swirl Combined Sewer Overflow Regulator/Separator -- In the context involved in this study and report, a chamber with necessary appurtenant structural configurations which will kinetically induce a rotary motion to the entering wastewater flow from a combined sewer, resulting in secondary motion phenomena which will cause a concentration of solid pollutional materials at a predetermined location, from which it can be diverted into the foul sewer, thereby producing a partially clarified waste for decantation or overflow into receiving or storm overflow treatment facilities.

Transition Section -- That portion of the helical bend combined sewer overflow regulator/separator which carries the combined sewer flow from the entering sewer pipe section and delivers it to the straight section and thence to the bend section; the transition section in the studies had a length of at least fifteen times the inlet sewer diameter and expanded the flow cross section to three times the inlet diameter.

Underflow -- The concentrate, containing the recovered solids, which is withdrawn from the bottom of the swirl concentrator for erosion runoff treatment; the converse of the clarified overflow.

Weir Plate -- A plate or surface constructed contiguous with the outlet overflow weir of a swirl chamber. In the swirl combined sewer overflow regulator/separator, a weir skirt hanging below the weir traps floatables and holds them until released for removal from the chamber.

Weir Skirt -- A plate hanging below the weir plate, to assist in retaining floatable solids under the weir plate and in inducing the shearing of the chamber flow into an exterior liquid mass and an interior liquid mass, thereby optimizing the solids separation effectiveness of the swirl concentrator principle.

WWF (Wet-Weather Flow) -- The flow in the combined sewer caused by rainfall or snow melt and the dry-weather flow.

SECTION X
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SECTION XI

APPENDIX - OTHER APPLICATIONS AND DEVICES

The fundamental work which was used to develop the family of secondary flow solids separation devices fostered interest by other researchers interested in treating other wastewater streams. Such devices are in various stages of development and testing. Reference is made to them in this manual to indicate other applications of the flow principle.

The following listed devices will be highlighted:

1. An advanced primary treatment unit,
2. A device to use when bypassing excess sewer flow, and
3. A treatment unit for aquaculture wastes.

In addition, a short description of the Hydro-Brake(R) will be given. This unit, while not used for treatment, uses secondary motion, and has no moving parts and acts as an effective foul sewer flow control.

ADVANCED PRIMARY TREATMENT

Bernard Smisson, the developer of the swirl principle for combined sewer overflow regulations and treatment has continued his work to other applications including primary treatment. Following the hydraulic model studies which led to the design of the unit described in Section VI of this manual, Smisson introduced moving scrapers to hasten sludge collection and reduce the overall size of the unit. Laboratory results need to be confirmed with a larger scale unit and closer control of the solids used to represent wastewater solids before a prototype unit is constructed. However, the general approach appears promising.

SEWER BYPASS POLLUTION CONTROL

Russell Allen White, while a graduate student of the University of Wisconsin, Milwaukee, was concerned with the pollution problems resulting from the discharge from pumps when overloaded sewers were relieved to minimize basement flooding. He developed, through hydraulic model tests, a small unit through which the pump could discharge. (22) The foul flow was then returned to the sewer, and only the clarified effluent discharged to national waterways.

Such a device represents a practical interim measure to treat part of the flow until such time as the system has infiltration and inflow removed or sufficient transport capacity established.

TREATMENT FOR AQUACULTURE WASTES

Andrew B. Buch, while a graduate student at Clemson University extensively studied the methods and problems associated with treatment of fish rearing wastes (39) Many forms of aquaculture are being advanced as a means of providing protein to the world population. Such facilities use a large flow-through volume of water which must be treated before discharge. A variation of the swirl separator was adapted to allow partial treatment of the flow prior to recycling to minimize total quantities needed. The wastes of interest are solids, BOD_5, and ammonia. A primary finding of a comparative study of several treatment systems found that "the swirl primary unit requires one-third the volume of normal retention basins with more effective treatment. Since reduced construction and maintenance costs also make this unit attractive, it should be seriously considered for aquaculture pollution abatement."

HYDRO-BRAKE(R)

A Hydro-Brake is a patented flow controller made of stainless steel. It is self-regulating and has no moving parts. It requires no power, but uses the static head of stored water to operate its own "energy" to retard the flow. The movement of water through a Hydro-Brake involves a swirl action, dissipating energy to control the rate of discharge. Although the function of a Hydro-Brake is somewhat similar to an orifice, it has certain important advantages:

1. It permits a much larger opening for passage of the same amount of water. This is particularly important where clogging is a possibility, such as, for instance, in catch basins. It is also important where sanitary or combined sewage flows are being regulated.
2. The flow rate of a Hydro-Brake is not significantly affected by a variation in head. This is important where it is desirable to maintain a relatively large passage for the water, yet also maintain a fixed maximum rate of flow during peak conditions.
3. The outflow from a Hydro-Brake does not create a high velocity jet stream as an orifice will, thus avoiding scouring inside sewer pipe.

The Hydro-Brake was invented in Denmark about 15 years ago and has, since 1975, been marketed in North America by Hydro Storm Sewage Corporation of New York.

The flow control device has been used successfully on units at Boston, Massachusetts and Lancaster, Pennsylvania. The Hydro-Brake must be designed for the specific application and flow condition. The design is patented and units are available only through the Hydro Storm Sewage Corporation, New York City. Units may be very small, say for a 10 cm (4 in.) diameter pipe, or several meters in diameter.

OTHER WORK

A vortex type solid-liquid separator was tested in 1972 by S. Giray Veliglu while a graduate student at Bogazici University, Istanbul, Turkey. (40) The unit was designed to be used as an intermediate unit in water and wastewater treatment. Available data does not allow direct comparisons to the units described in this manual.

The Turkish report concluded, however, that the unit met their design objectives.

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