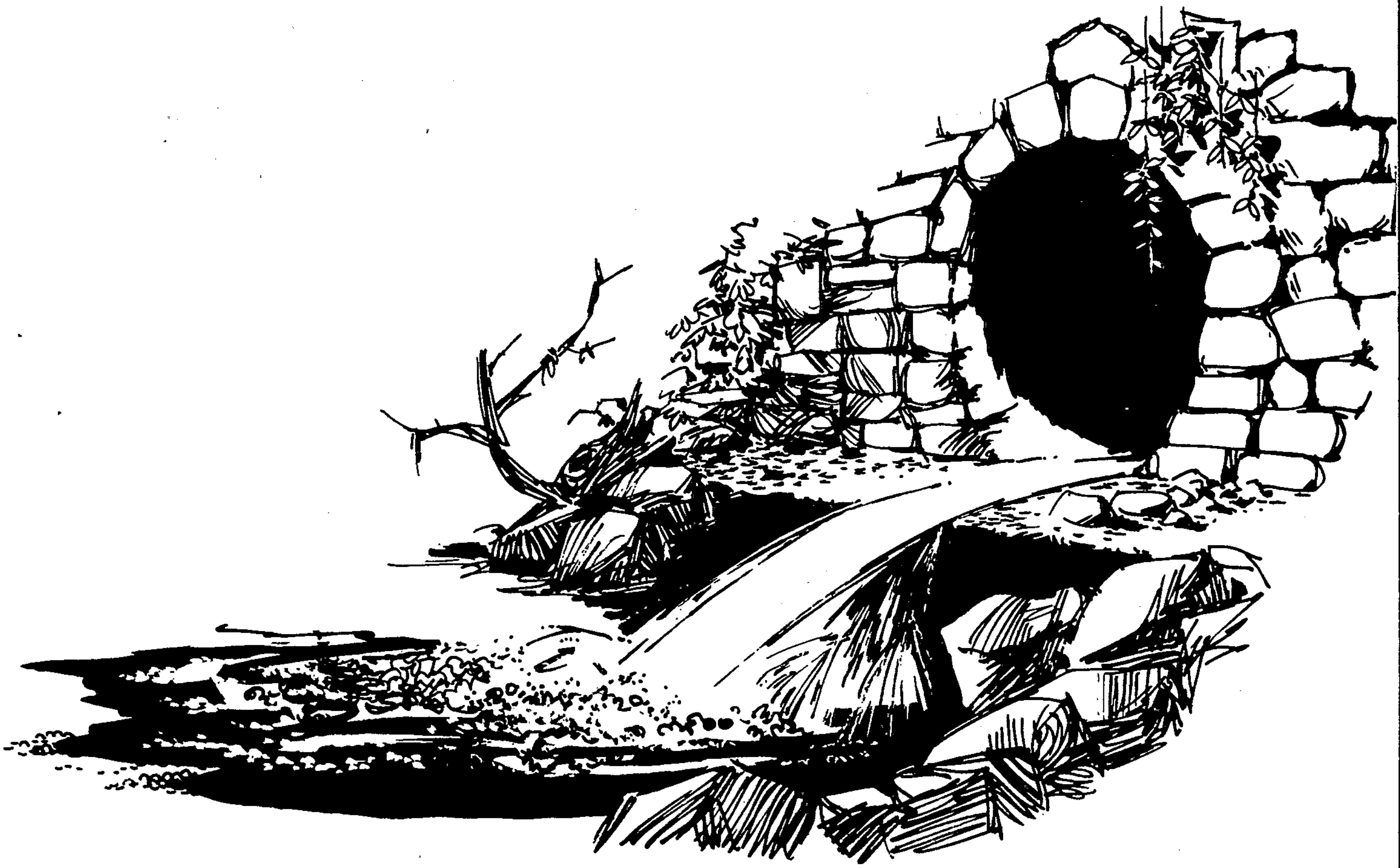
WATER POLLUTION CONTROL RESEARCH SERIES • 11022 DMU 08/70

Combined Sewer Regulation and Management

A Manual of Practice



U.S. DEPARTMENT OF THE INTERIOR • FEDERAL WATER QUALITY ADMINISTRATION

WATER POLLUTION CONTROL RESEARCH SERIES

The Water Pollution Control Research Reports describe the results and progress in the control and abatement of pollution of our Nation's Waters. They provide a central source of information on the research, development and demonstration activities of the Federal Water Quality Administration, Department of the Interior, through in-house research and grants and contracts with the Federal, State, and local agencies, research institutions, and industrial organizations.

Triplicate tear-out abstract cards are placed inside the back cover to facilitate information retrieval. Space is provided on the card for the user's accession number and for additional key words. The abstracts utilize the WRSIC system.

Water Pollution Control Research Reports will be distributed to requesters as supplies permit. Requests should be sent to the Project Reports System, Office of Research and Development, Department of the Interior, Federal Water Quality Administration, Washington, D. C. 20242.

Previously issued reports on the Storm & Combined Sewer Pollution Control Program:

11020	–	12/67	Problems of Combined Sewer Facilities and Overflows, 1967. (WP-20-11)
11030	DNS	01/69	Water Pollution Aspects of Urban Runoff. (WP-20-15)
11020	EXV	07/69	Strainer/Filter Treatment of Combined Sewer Overflows. (WP-20-16)
11020	FKI	01/70	Dissolved Air Flotation Treatment of Combined Sewer Overflows. (WP-20-17)
11020	DIH	06/69	Improved Sealants for Infiltration Control. (WP-20-18)
11020	DES	06/69	Selected Urban Storm Water Runoff Abstracts. (WP-20-21)
11020	DIG	08/69	Polymers for Sewer Flow Control. (WP-20-22)
11020	EKO	10/69	Combined Sewer Separation Using Pressure Sewers. (ORD-4)
11020	–	10/69	Crazed Resin Filtration of Combined Sewer Overflows. (DAST-4)
11023	FDD	03/70	Rotary Vibratory Fine Screening of Combined Sewer Overflows. (DAST-5)
11020	–	06/69	Sewer Infiltration Reduction by Zone Pumping. (DAST-9)
11020	DGZ	10/69	Design of a Combined Sewer Fluidic Regulator. (DAST-13)
11023	DPI	08/69	Rapid-Flow Filter for Sewer Overflows.
11020	–	03/70	Combined Sewer Overflow Seminar Papers.
11020	DWF	12/69	Control of Pollution by Underwater Storage.
11024	–	06/70	Combined Sewer Overflow Abatement Technology.
11034	FKL	07/70	Storm Water Pollution.
11024	FKN	11/69	Storm Pollution & Abatement from Combined Sewer Overflows–Bucyrus, Ohio. (DAST-32)
11024	DMS	05/70	Engineering Investigation of Sewer Overflow Problem–Roanoke, Virginia.
11000	–	01/70	Storm & Combined Sewer Demonstration Projects–January 1970.

c. 2

COMBINED SEWER REGULATION AND MANAGEMENT

A MANUAL OF PRACTICE

by the

American Public Works Association

for the

FEDERAL WATER QUALITY ADMINISTRATION

DEPARTMENT OF THE INTERIOR

and

TWENTY-FIVE LOCAL GOVERNMENTAL JURISDICTIONS

Program No. 11022 DMU c. 2

Contract 14-12-456

July 1970

For sale by the Superintendent of Documents, U. S. Government Printing Office
Washington, D.C., 20402 - Price $1.50

SPONSORING LOCAL AGENCIES

City of Akron, Ohio
City of Alexandria, Virginia
City of Atlanta, Georgia
City of Boston, Massachusetts
Metropolitan District Commission, Boston, Massachusetts
City of Charlottetown, PEI, Canada
Metropolitan Sanitary District of Greater Chicago, Illinois
City of Cleveland, Ohio
City of Eugene, Oregon
City of Fort Wayne, Indiana
City of Kansas City, Missouri
City of Middletown, Ohio
City of Montreal, Quebec, Canada
City of Muncie, Indiana
Metropolitan Government of Nashville and Davidson County, Tennessee
City of Omaha, Nebraska
City of Owensboro, Kentucky
Allegheny County Sanitary Authority, Pittsburgh, Pennsylvania
City of Richmond, Virginia
Metropolitan St. Louis Sewer District, Missouri
City of St. Paul, Minnesota
Municipality of Metropolitan Seattle, Washington
City of Syracuse, New York
City of Toronto, Ontario, Canada
Washington, District of Columbia

FWQA Review Notice

This report has been reviewed by the Federal Water Quality Administration and approved for publication. Approval does not signify that the contents necessarily reflect the views and policies of the Federal Water Quality Administration.

ABSTRACT

Design application, operation and maintenance of combined sewer overflow regulator facilities are detailed in this Manual of Practice, developed in conjunction with a report prepared on combined sewer overflow regulators.

Design calculations are given for various types of regulators and tide gates. A sample regulator facility control program is given to illustrate the development of a control system. Operation and maintenance guidelines are also given. Thirty-eight sketches and photographs are included.

This manual and accompanying report were submitted in fulfillment of Contract 14-12-456 between the Federal Water Quality Administration, twenty-five local jurisdictions and the American Public Works Research Foundation.

APWA RESEARCH FOUNDATION
Project 68-1b

STEERING COMMITTEE

Peter F. Mattei, Chairman

Arthur D. Caster	Walter A. Hurtley
William Dobbins	Ed Susong
George T. Gray	Harvey Wilke
Carmen Guarino	

Richard H. Sullivan, Project Director
J. Peter Coombes, Principal Investigator

SPECIAL CONSULTANTS

James J. Anderson
Dr. Morris M. Cohn
Morris H. Klegerman
Ray E. Lawrence
M. D. R. Riddell

APWA Staff*

R. D. Bugher	Violet Perlman
R. H. Ball	Katherine Manolis
Leo Weaver	Ellen M. Pillar
Lois V. Borton	Maxine Coop
Marilyn L. Boyd	Mary J. Webb
Kathryn D. Priestley	Sheila J. Chasseur
Patricia Twist	Oleta Ward

*Personnel utilized on a full-time or part-time basis on this project

CONTENTS

FIGURES

TABLES

IN EXPLANATION: THE PURPOSE OF A MANUAL OF PRACTICE ON COMBINED SEWER REGULATION AND MANAGEMENT

The American Public Works Association has completed a research study covering a national in-depth investigation of combined sewer practices of representative local jurisdictions. The study covered the design, choice and application of regulator devices, use of tide or backwater gate facilities, performance of regulator-overflow installations, and operation and maintenance methods. Efforts were made to obtain reliable information on the cost of construction, equipment, and maintenance of regulators and their appurtenant facilities. The report of this investigation covers the methods utilized in the survey, an evaluation of the research data, an enumeration of the findings and a list of recommendations. The recommendations when adopted should result in better engineering technologies and the utilization of more sophisticated methods and mechanisms in future regulator practice.

The investigation disclosed the inadequacies of many of the present regulator-overflow devices, and methods of their operation. Wet-weather overflows were, as expected, common to all installations; however, these overflows were more frequent and more extended in duration than necessary to protect upstream collector sewers, and downstream interceptors and treatment works. Even instances of dry-weather overflows were reported. Little or no attempt has been made to improve the quality of overflow waste waters by the use of supplementary protective devices.

The major deficiencies, unreliability and inadequacy of regulator-overflow installations, in the main, were the result of failure to apply recognized engineering and construction methods. These malpractices included such matters as: Lack of overall planning of combined sewer systems and of recognition that overflow regulation is a total systems problem; the use of an excessive number of overflow points to protect local sewer systems from surcharging and backflooding, without considering the impact of any single regulator-overflow station on a total system network and on the pollution of receiving waters; inexact design criteria for such facilities; inappropriate choice and application of particular types of regulators to the specific control functions; and ineffective operation and maintenance procedures.

It is apparent that many of these positive factors resulted from cut-and-try design, and operation techniques utilized during the years when combined sewer flows were discharged to receiving waters without treatment. The advent of sewage treatment, coupled with higher standards of pollution control of receiving waters, now make it mandatory to utilize improved practices in combined sewer regulation.

These improvements in regulator technologies are of practical importance. The engineering profession and government officials must face the responsibility of providing for the control of combined sewer overflows at the lowest possible cost commensurate with dependable performance and the reduction in pollution discharged to receiving waters.

These purposes can be enhanced by establishing construction, equipment, and maintenance criteria, which will serve as functional guidelines for designers of combined sewer systems; owners of such sewer facilities; manufacturers who provide the equipment for regulator and sewer system management procedures; and operation and maintenance personnel who must get the best possible service from the best possible facilities.

It is the purpose of this Manual of Practice on Combined Sewer Regulation and Management to present guidelines for:

1. Applicability of types of regulators to meet specific control needs (Section 1)
2. The design and layout of regulator structures and regulator devices and controls (Section 2)
3. Design of tide or backwater gate devices and structures (Section 3)
4. The application of instrumentation and control facilities for the purpose of achieving maximum performance from each individual regulator station and of the integration of all regulator stations into a total controlled systems management program (Section 4)
5. Improved operation and maintenance practices (Section 5).

This Manual of Practice is not intended to be a "cookbook." It offers guidance to the design engineering profession; to manufacturers and suppliers of products and processes of primary and secondary significance to the regulator fields; and to governmental agencies and officials who are responsible for the administration and operation of combined sewer systems. The Manual is not a substitute for knowledge and experience. It is a tool for the use of properly trained and experienced professionals.

SECTION 1

TYPES OF REGULATORS; BASIC PRINCIPLES; APPLICABILITY; GUIDELINES ON SELECTION

CONTENTS

STATIC REGULATORS

1.1 Manually Operated Gates

1.1.1 Basic Principle

The chief element of this regulator is a manually operated vertical gate mounted on a vertical orifice, through which passes the intercepted sewage which is diverted to the treatment plant. Low flows pass through the gage, a channel with either subcritical or critical depth. At higher flows the regulator may act as a simple vertical orifice with free discharge or as a submerged orifice.

The regulator structure consists of an overflow dam constructed across the channel of the combined sewer which diverts dry-weather flow through the gate into an orifice chamber. From this chamber the flow is conveyed by a branch interceptor to the main interceptor. In wet weather, excess flows discharge over the dam to the receiving waters.

1.1.2 Application

This device may be used for diverting wet-weather flows of less than 4 cfs. since dynamic regulators rarely are justified economically for such small flows. Manually operated gates are considered more effective as regulators than other types of static devices. The gate opening is adjustable so that the quantity diverted to the interceptor may be varied. Operation and maintenance costs, as well as construction costs, are less for this type than for automatic regulators. It is most applicable where further regulation of the diverted flow will occur downstream, either at another regulator or at the treatment plant.

For flows of less than 4 cfs the choice of regulator would appear to be between the manually operated gate and the tipping gate described in 1.8. However, the latter device diverts a lower ratio of wet-weather to dry-weather flow.

1.2 Fixed Orifices–Vertical

1.2.1 Basic Principle

The main element is a fixed vertical opening in the combined sewer through which the diverted flow passes. The principle is as described for the manually operated gate.

1.2.2 Application

This device is generally used for wet-weather flows of less than 2 cfs. It is specifically applicable where additional regulation of the diverted flow will occur further downstream, either at another regulator or at the treatment plant. When not regulated downstream, the intercepted flow during a storm may exceed design quantity. When used for small flows, such excess interception may not be serious in a large sewer system where only a small percentage of the total flow is diverted to the plant.

The use of a manually operated gate is preferred over a fixed orifice, although the gate represents an additional capital cost.

While provision can be made in design for varying the size of the fixed orifice by the use of removable plates, the changing of such plates may be difficult after the regulator has been in service for some time. The fixed orifice does not require lubrication as does the manually operated gate.

1.3 Fixed Orifices–Horizontal (The Drop Inlet)

1.3.1 Basic Principle

The device consists of a horizontal opening constructed either in the bottom of the combined sewer or the bottom of a separate chamber of the combined sewer. In the former case it is usually covered with a grate. When constructed in a separate chamber a diversion dam must be constructed in the combined sewer to divert the dry-weather flow through a vertical opening into the orifice chamber. It is also preferable to use such a dam when the orifice is placed in the combined sewer. In this way the opening acts as a horizontal orifice under all conditions of flow. The flow not diverted into the interceptor through the orifice passes over the dam to the receiving waters.

1.3.2 Application

This device is generally used for diverting wet-weather flows of less than 2 cfs. Present practice is to replace this type with more effective regulators.

During dry-weather periods, in spite of daily maintenance, clogging of the grates frequently causes excessive overflow to the receiving waters. During storm periods, clogging of the grates, or "jumping across" the orifice, causes a large portion of the total flow to discharge to overflow.

Maintenance is difficult as it is impossible to shut off all flow to the interceptor unless the horizontal orifice is placed in a separate chamber. When a separate chamber is used, a vertical orifice will result in less variation in intercepted flows during storm periods than is the case in a horizontal orifice.

The only advantages of this type of regulator are its low initial capital cost and that, at most, only one structure is required.

The horizontal orifice does not regulate combined sewer flows effectively and is expensive to maintain.

1.4 Leaping Weirs

1.4.1 Basic Principle

This regulator consists of an invert opening in the combined sewer dimensioned to permit the dry-weather flow to fall through the opening and to be conveyed through a branch interceptor to the main interceptor and treatment plant. During wet-weather periods the increase in velocity and depth in the combined sewer causes all or most of the flow to pass over or leap over the opening and continue on to receiving waters.

1.4.2 Application

Leaping weirs generally have been used for intercepting low volume flows. While used to a considerable extent in the past, recent practice is to replace existing leaping weirs with other types of regulators.

The disadvantages of this regulator are:

1. It cannot be used when a tide gate is required since the backwater effect will prevent the leaping action and the device will act as a horizontal orifice.
2. During storm periods all the flow may leap over the opening and part of the flow will not be diverted to the treatment plant.
3. It is difficult, if not impossible to temporarily shut off all flow to the interceptor and treatment plant.
4. The opening may become clogged or bridged with floating material, causing spillage of dry-weather flow into the receiving waters.

Its main advantage is that only one structure is required, which may be desirable where space is limited or where economy is essential. The leaping weir is not considered an effective regulator.

1.5 Side-Spill Weirs

1.5.1 Basic Principle

The side-spill weir is constructed parallel to, or at a slight angle to the axis of the combined sewer, with the crest set at an elevation above the peak dry-weather flow line. During wet-weather periods flows in excess of the peak dry-weather flow will discharge over the weir into the outfall sewer. The excess flow may be further regulated downstream or may discharge directly into the receiving waters.

1.5.2 Application

Theoretically, the side-spill weir may be used for the overflow of any quantity of excess wet-weather flow. However, since the length of the weir is proportional to the quantity of overflow, the structure becomes larger and more costly as the volume of overflow increases. Side-spill weirs are frequently used for low flows, and where the overflows are regulated further downstream.

The major advantage of this type of regulator is that maintenance costs are generally lower than for other types. The major disadvantage is that the regulator cannot be adjusted after construction except by reconstruction of the weir or by manual adjustment of the weir crest.

When close regulation of the flow to the plant is desired it is preferable to use other types of regulators, the design of which is based on accepted hydraulic principles. Little or no field data have been published regarding the operation of these weirs that conform with the theoretical values for larger flows. Where such regulation farther downstream does not take place the intercepted flow in times of storm may exceed interceptor design capacity. This should be checked in design to insure that such excess interception does not result in unnecessary spills at outfall locations downstream or cause surcharging at the treatment plant.

1.6 Internal Self-Priming Siphons

1.6.1 Basic Principle

A siphon may be defined as a closed conduit which lifts a liquid to an elevation higher than its free surface and discharges it at a lower elevation. When a closed conduit rises above the hydraulic grade line, negative pressure (i.e., pressures less than atmospheric pressure) develops at the summit which is equal to the vertical distance between the hydraulic grade line and the center line of flow at the summit. To initiate operation the siphon must be primed, i.e., sufficient negative pressure must be developed at the summit to raise the water in the uptake branch of the siphon until flow is established. Priming is effected by removal of air from the summit either by mechanical means, such as a vacuum pump, or by use of the hydraulic energy inherent in the differential in elevation between the upper and lower water surfaces of the intake and discharge levels of the siphon. This difference in levels is the operating power which must overcome all energy losses within the siphon, including friction, entry, bends, and discharge; it generates the required velocity to maintain the design flow rate. Under maximum vacuum conditions in a siphon, a water column could rise to a theoretical height of approximately 34 feet at mean sea level. Practically, a water column height of only 75 percent of the theoretical maximum can be obtained.

1.6.2 Application

The internal self-priming siphon may be used to divert excess storm flows to receiving waters and not to the interceptor as the normal regulator practice dictates. Such flows should exceed about 5 cfs so that the throat section will be large enough to prevent clogging. It may be used to replace an overflow weir in any regulator, provided there is an adequate difference in water levels above and below the siphon to allow it to function as designed. In some cases the siphon may be more economical than a weir due to the smaller structure required. This type siphon also can be used to control the water levels in a conduit leading to a treatment plant or pumping station in order to prevent excessive flows to these installations. The siphon will maintain the water levels within a narrow range by discharging excess flows to receiving waters. For this purpose it may be desirable to use two or three siphons, with each succeeding one set to operate at a slightly higher water level.

DYNAMIC REGULATORS—SEMI-AUTOMATIC

1.7 Float-Operated Gates

1.7.1 Basic Principle

This type regulator consists of a regulating gate, a float, and an interconnecting device arranged so that variations in the water level either in the combined sewer or interceptor will move the float and actuate the gate. Operation of the gate does not require either hydraulic pressure or electric power. A typical layout is shown in Figure 2.7.1.

The regulator consists of an overflow dam or weir constructed across the channel of the combined sewer to divert dry-weather flow through the regulating gate into the regulator chamber and thence into the branch interceptor. The branch interceptor discharges the diverted flow into an interceptor which conveys the flow to the treatment plant. In wet weather excess flows discharge over the dam and continue through the storm sewer to the receiving waters.

1.7.2 Application

Theoretically, this type device can be used for diversion of any volume. Generally, however, its use will not be economically justified for diverting flows of less than 4 cfs. Its major advantage is that no outside source of energy is required for operation. Regulation is controlled by movements of the float. In the larger sizes, the float diameter may be as much as 5 feet. This requires a large size floatwell which may trap grit that creates a maintenance problem. Accumulation of floating material on the float may cause malfunctioning of the system. Since the entire system is in fine balance, proper operation requires at least biweekly maintenance.

1.8. Tipping Gate

1.8.1 Basic Principle

This regulator consists of a rectangular metal plate mounted on a horizontal pivot located below the center of gravity of the plate. The plate is mounted in a casting in such a manner that the flow diverted to interceptor must pass under it. During dry-weather flows the pressure on the upstream side of the gate is below the pivot and the gate rests in the open position permitting all flow to pass into the interceptor. During periods of storm flow the water level in the combined sewer rises and the resultant pressure on the upstream side of the gate above the pivot causes the gate to partially close, thus reducing the orifice area and limiting the quantity of flow to the interceptor. The remainder of the storm flow discharges over a diversion dam into the outfall sewer and into the receiving waters.

1.8.2 Application

Tipping gates can be used to divert a wide range of flow volumes. These gates will intercept less flow in wet-weather periods than the fixed orifice or manually operated gate due to the partial closing or "tipping" of the gate by the upstream water pressure. The device can be adjusted in the field to revise either the maximum or the minimum opening, thus altering the flow to be intercepted. The discharge through a 12-inch gate under a head of one foot will vary from about 1 to 6 cfs depending on the opening height. The head required to close the gate will vary from 0.3 to 1.5 feet, depending upon the downstream water level.

1.9 Cylindrical Gates

1.9.1 Basic Principle

This device consists of a horizontal circular orifice located in a chamber adjacent to the combined sewer. The regulator is a cylindrical gate balanced by a counterweight and hung from an articulated frame directly over the orifice.

The rising of the water surface, either in the collector or in the interceptor, controls directly the closing of the orifice by the cylinder without the use of floats and transmissions.

1.9.2 Application

This new type regulator has been used in only one city for sewage diversion. Operation and maintenance problems are still being worked out. Further performance records are needed for accurate evaluation. This gate is hydraulically activated by sewage flow and, hence, no outside source of energy is required. According to the manufacturer this device is suitable for diverted flows of from less than 10 cfs to 200 cfs.

1.10 Motor-Operated Gate

1.10.1 Basic Principle

This regulator functions in similar fashion to the cylinder-operated gate (1.11) except that the gate is operated directly by a motor rather than a pneumatic or hydraulic cylinder.

1.10.2 Application

The motor-operated gate can be used for any volume of flow where automatic or remote control of the diverted flow is desired. Its use is not generally considered economically feasible for design flows of less than 4 cfs. Electric power must be available for operation of the motor. The motor is mounted on a floor stand directly above the gate.

If the sewer is deep enough the motor can be housed in an underground chamber; otherwise the motor will require housing above the ground. The latter alternative is preferable in any case since corrosion is less in an above-grade site. If an underground chamber is used, dehumidification and heating equipment may be provided or special equipment may be provided to handle these difficult conditions.

1.11 Cylinder-Operated Gate

1.11.1 Basic Principle

The chief element of this regulator is a cylinder-operated gate and orifice through which the intercepted flow passes to the treatment facility. The gate is operated by a hydraulic or pneumatic cylinder which responds to the sewage level as measured by a sensing device located either upstream or downstream of the gate. Operation of the cylinder may be by water, oil or air pressure. The sensor usually is either a float or compressed-air bubbler tube. The gate also may be operated by remote control which overrides the sensing device.

The regulator consists of an overflow dam constructed across the channel of the combined sewer so as to divert maximum dry-weather flow through a sluice gate into a regulator chamber. From this chamber the flow is conveyed by a branch interceptor to the main interceptor which leads to the treatment plant. Excess flows during storm periods will overflow the dam and continue in the combined sewer to the receiving waters.

1.11.2 Application

The cylinder-operated gate is suitable for flows over 4 cfs where automatic regulation of the diverted flow is desired. While this type can be used on smaller flows, it is not generally considered economical.

The water-cylinder type can be used where a water supply is available which will produce a minimum cylinder pressure of 25 psi. Because of the low cylinder pressure the size of sluice gate is generally limited to 9 to 12 square feet. Multiple gates are used where the opening exceeds 9 to 12 square feet. The hazard of cross-connections between the cylinder system and the public water supply must be considered. This is an important design requirement.

The oil-cylinder type requires an electric power source to operate the oil pump and the air compressor. To protect this equipment from the effects of the sewer atmosphere a separate chamber must be constructed to house the electrical equipment. Oil pressure of about 750 psi is preferred. The gate is not restricted as to size, as in the case of the water-cylinder type.

The air-cylinder type also requires a source of electric power to operate the air compressor. Air pressures of 90 to 200 psi have been used.

In jurisdictions that have tried both types, the oil-cylinder is preferred. The principal advantage of the oil-cylinder or air-cylinder type is that the flow can be monitored and regulated from a remote point thus making full use of the interceptor system, and its storage capacity, while protecting downstream treatment facilities and reducing the frequency and volume of overflows.

In general, cylinder-operated gates are considered an effective type of regulator currently in use in North America.

1.12 Current Developments–New Devices

1.12.1 General

This subsection includes regulators of recent design on which experimental work has been done and which, in some cases, have been installed for actual use.

1.12.2 Fluidics

Fluidics is defined as "the use of devices that have no moving parts, and that use a fluid medium for control of other devices, or that directly achieve an objective such as logic, computation or amplification". (Engineer, Jan.-Feb., 1969).

Fluidic devices of two general types have applicability, depending on the type of fluid-flow interaction that takes place within them. These categories are: (1) wall attachment, and (2) vortex amplifier.

Wall attachment devices form the largest group of fluidic components. In these devices, a high-velocity jet of fluid, emitted between two walls, attaches itself to one of them, attracted there by an area of lower pressure next to the wall caused by air entrainment.

The jet remains stable in this position unless it is disturbed by a pressure pulse or by continuous pressure from a central port. The basic configuration is shown in Figure 1.12.2.1.

The vortex amplifier consists of a cylindrical chamber as shown in Figure 1.12.2.2, an axially oriented outlet, a radially located supply inlet and a tangentially directed control inlet. When the flow is not being controlled, the inflow proceeds directly through the chamber to the outlet. When the flow is controlled, the momentum exchange between the inflow and control flow establishes a resultant spiral flow path to the outlet. This centripetal acceleration can provide significant impedance to the flow and the variation is essentially proportional to the control flow in maximum/minimum ratios up to ten. The device, therefore, operates in analog fashion. This produces quantity control and, as explained in 1.12.3, the secondary velocities imparted in simple spiral motion may induce solids separation in the flow. This offers opportunity for the control of the quality of overflow wastes.

Figure 1.12.2.3 shows a schematic arrangement for a fluidic regulator. The combined sewer splits into two branches. The first, or branch interceptor, conveys the flow to the treatment plant and the second, the storm sewer, conveys additional flow to receiving waters. In dry-weather periods a low dam in the storm sewer diverts all the flow to the branch interceptor and thence to the treatment plant. In wet-weather periods the portion of the flow diverted to each branch can be regulated by the amount of air pressure or vacuum supplied to slots A and B. These slots extend the full height of the sewer. The air pressure or vacuum is self-induced by the flow in the sewer by the use of various pneumatec devices.

Flow in excess of design cannot be passed through the regulator device. Excess flow can be passed over the unit into the overflow channel.

1.12.3 Vortices

The vortex regulator (in England called a vortex overflow or rotary vortex overflow) consists of a circular channel in which rotary motion of the sewage is induced by the kinetic energy of the sewage entering the tank. Flow to the treatment plant is deflected and discharges through a pipe at the bottom and near the center of the channel. Excess flow in storm periods discharges over a circular weir around the center of the tank and is conveyed to receiving waters. It is claimed that the rotary motion causes the sewage to follow a long path through the channel. In this period secondary innovational flow patterns are established, setting up an interface between the fluid sludge mass and the clearer liquid. In effect, it is claimed the device acts as a quality separator. The flow containing the concentration of solids is directed to the interceptor.

Research has been carried out on hydraulic models and two full-sized regulators of this type have been built in Bristol, England, (Reference 1). Details of these two regulators are shown in Figure 1.12.3. Design factors for these regulators are as follows:

	Whiteladies Road	Alma Road
Regulator diameter (ft)	18	18
Overflow diameter (ft)	9	9
Av. dry-weather flow (cfs)	0.18	0.92
Wet-weather flow diverted to plant (cfs)	2.58	5.84
Ratio WWF:DWF–design	14	6
Storm flow–once a year (cfs)	44.0	54.7
Size inlet (ft)	3	4 x 3

The ratio of wet-weather flow to dry-weather flow of 6:1 used in the Alma Road regulator conforms with British practice. In the Whiteladies Road regulator, provision was made for reducing the ratio if it becomes necessary.

During dry-weather periods sewage enters the chamber and flows into the branch interceptor near the center. In storm periods excess sewage discharges over the center weir and flows out the storm sewer. The baffle and weir crest configurations prevent floating material from flowing over the weir.

The depth of the chamber from the weir level to the invert is dependent on the available head, since the plant outlet is operating under a hydraulic grade from the weir level to the point where the sewer flows free. The storm sewer outlet must pass under the chamber and, if necessary, the entrance to the pipe can be surcharged. The design of the overflow weir follows accepted hydraulic practice and its level will normally be set so that at maximum design flow the inlet sewer is full.

Model studies at Bristol using synthetic sewage solids indicated a higher removal of solids in the flow to the bottom of the regulator than in the flow over the weir. Pilot studies are now underway at Bristol using a vortex regulator as a primary clarifier for raw sewage.

However, another series of experiments elsewhere on a model vortex regulator using raw sewage indicated poor performance in removing screenable solids. Under certain conditions the concentration of screenings in the sewage over the weir was greater than in the sewage passing to the bottom of the regulator (Reference 2).

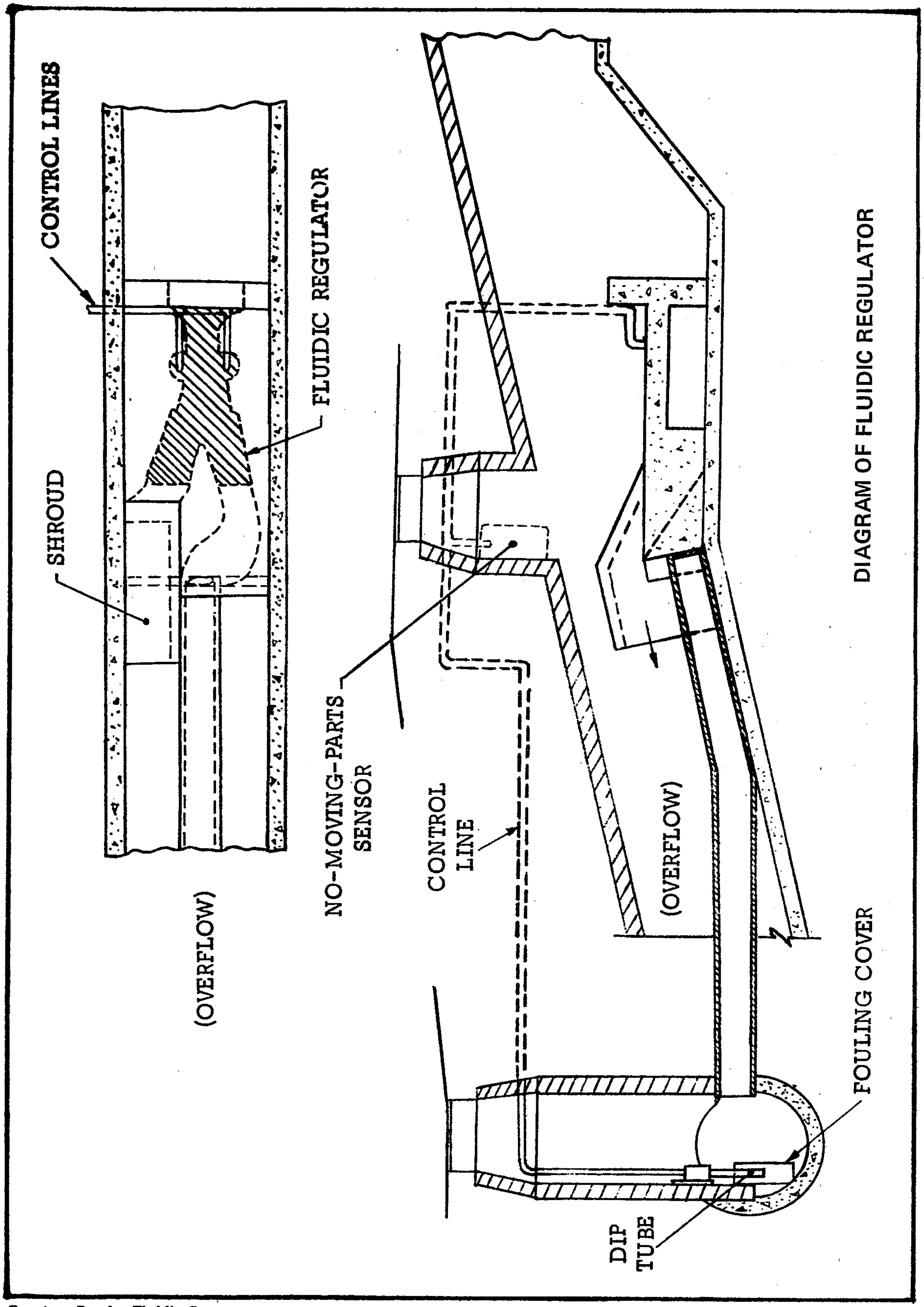
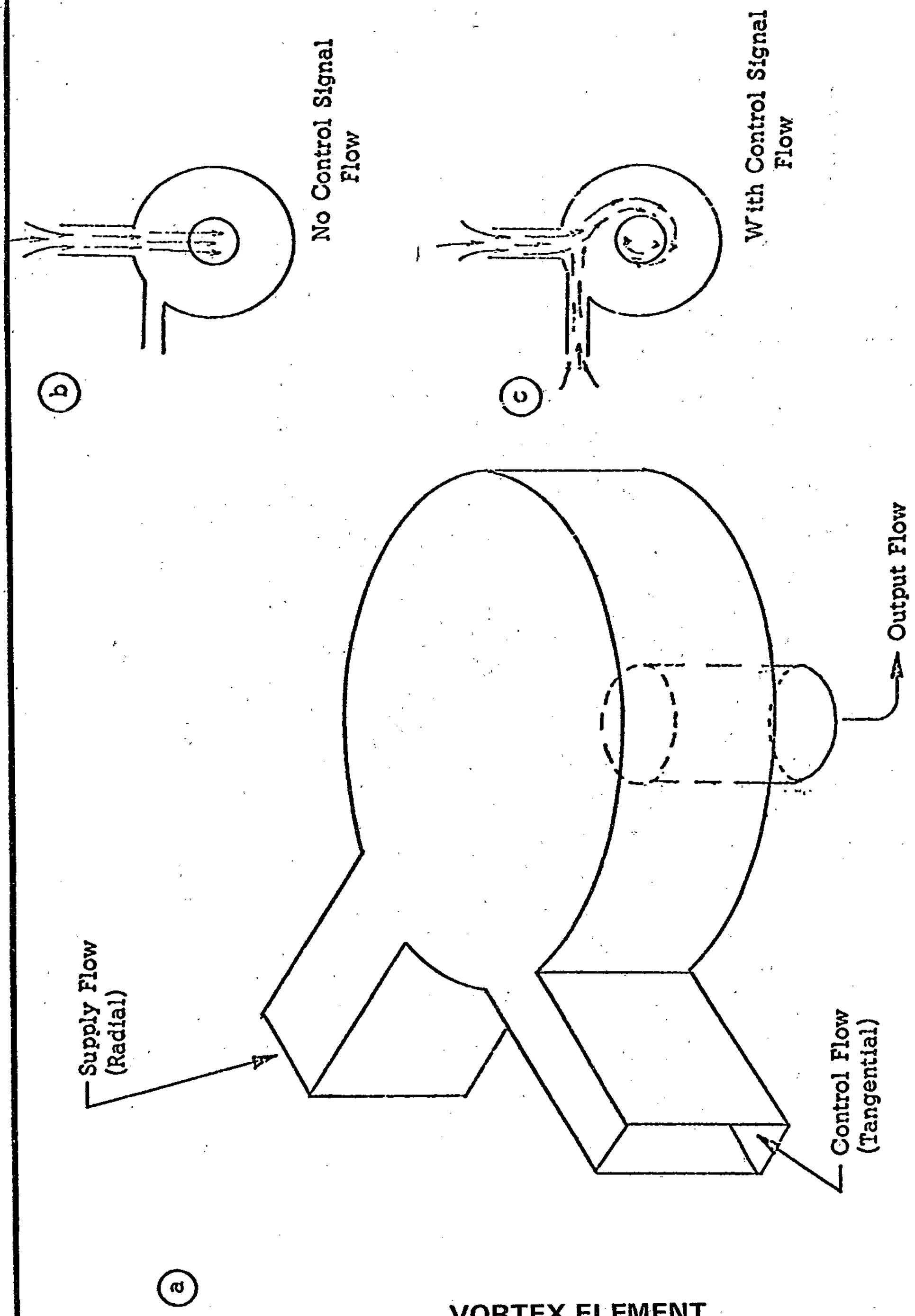


DIAGRAM OF FLUIDIC REGULATOR

Courtesy Bowles Fluidic Corp.

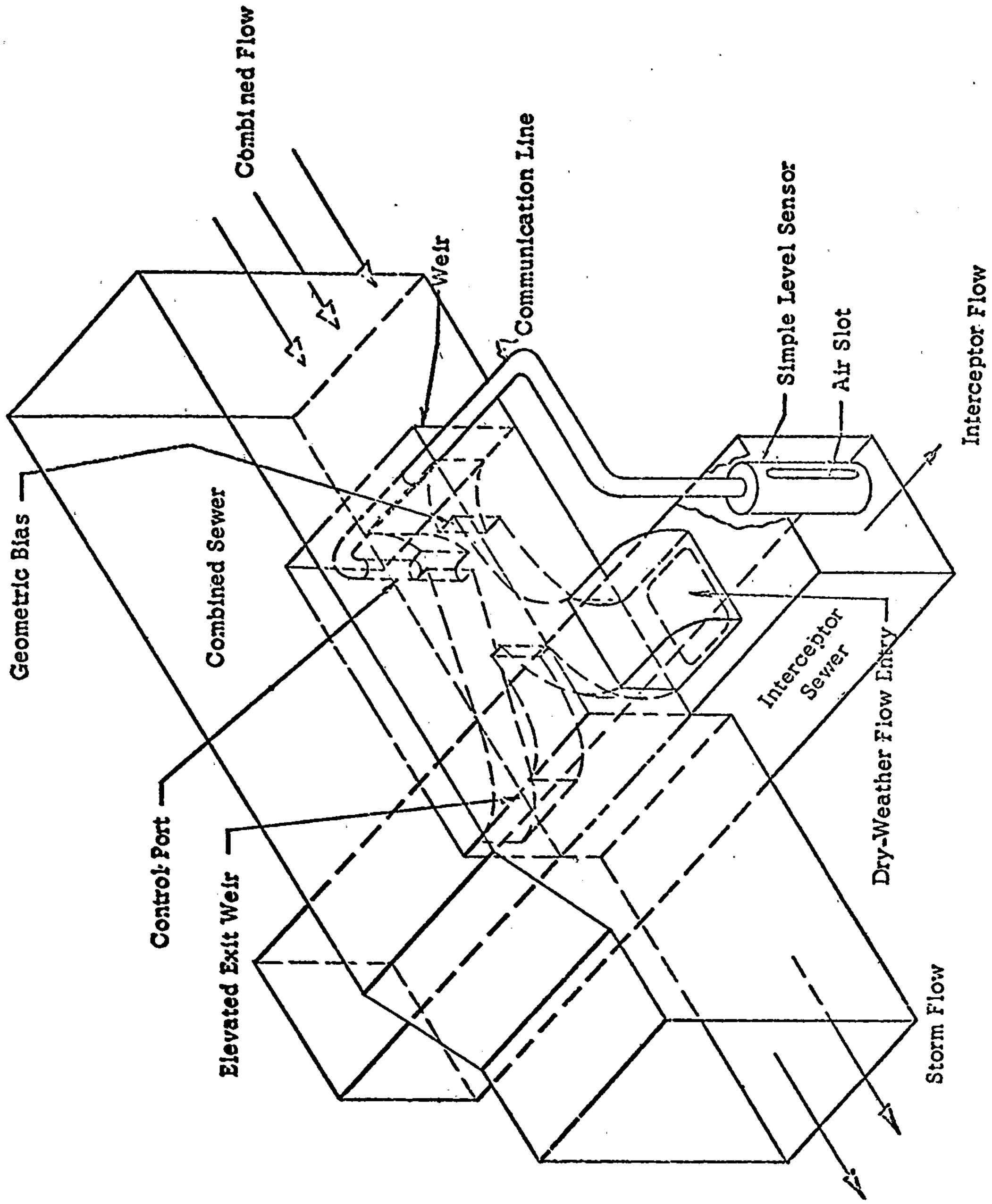
FIGURE 1.12.2.2



VORTEX ELEMENT

Courtesy Bowles Fluidic Corp.

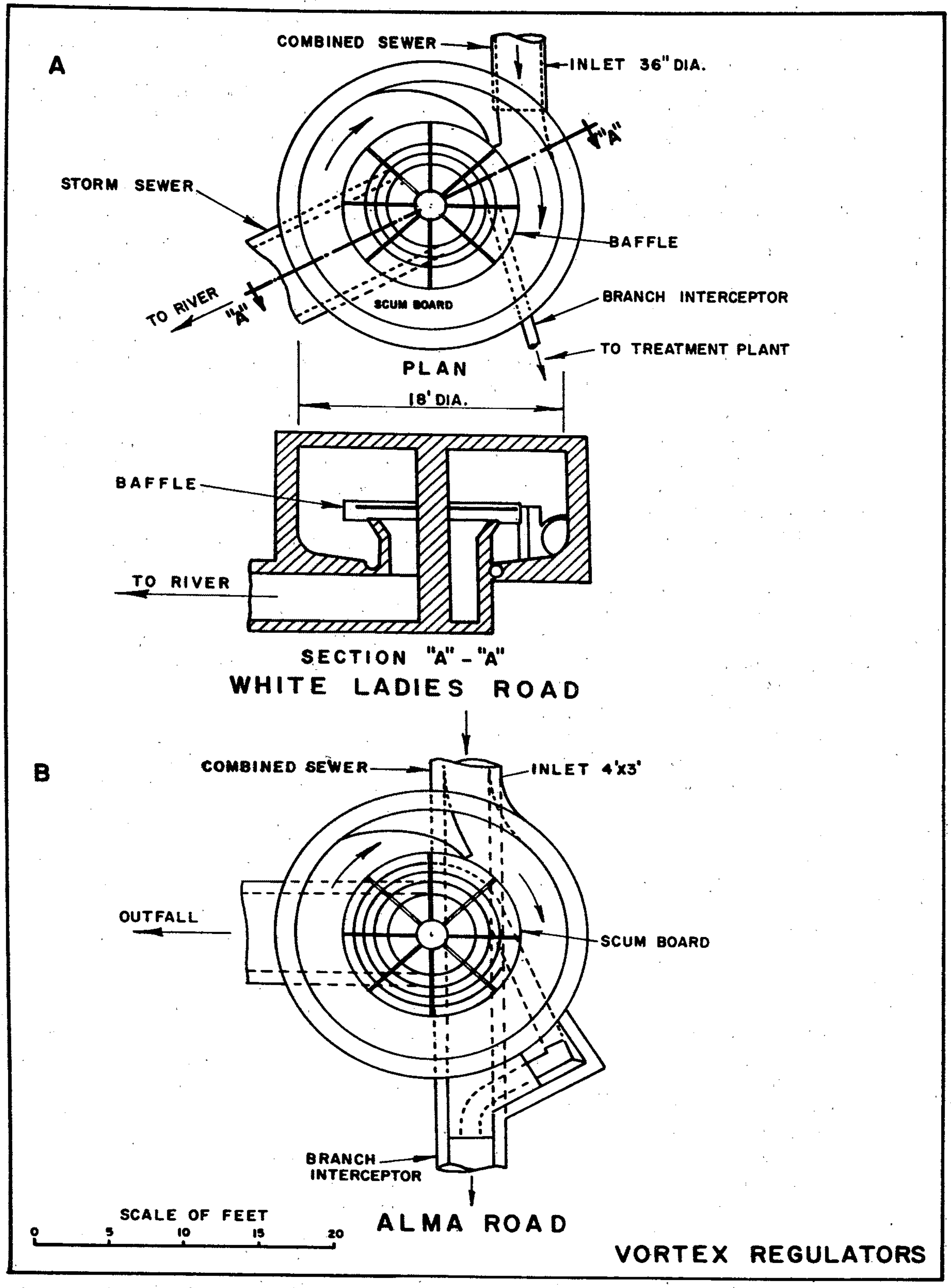
FIGURE 1.12.2.3



SCHEMATIC ARRANGEMENT – FLUIDIC INTERCEPTOR SEWER FLOW CONTROL – SINGLE SENSOR

Courtesy Bowles Fluidic Corp.

FIGURE 1.12.3



Courtesy Institution of Civil Engineers

1.12.4 Spiral Flow Separators

The spiral-flow regulator (in England the proposed name for the device is "storm sewage spiral-flow separator") is based on the concept of using secondary helical motions which exist in the bends of conduits, to establish a boundary layer between concentrated solids-bearing flow and clearer liquid, thus effectively separating the more heavily polluted mass for discharge to the interceptor.

Basically, in a bend of a conduit the direction of flow paths is not circumferential but follows a helical or spiral pattern. At the bend the more concentrated liquid mixture is nearer to the bed of the channel and tends to concentrate along the inner wall of the bend and the clearer liquid mixture tends to flow out towards the outer wall. With an overflow positioned along the outer wall of the bend it is possible to draw off the less polluted effluent. A bend with a total angle between 60 and 90 degrees is employed.

Model studies of this device were carried out at the University of Surrey, England (Reference 3). These investigations are by no means complete but they indicate that it is feasible, by one short bend or a series of short bends, to separate the heavily polluted sewage from the clearer overflowing liquid.

The simplest form of regulator suggested by the model studies is shown in Figure 1.12.4. A short bend of approximately 60 degrees is used as a separator. The heavily polluted sewage is drawn to the inner wall. It then passes to a semi-circular channel situated at a lower level leading to the treatment plant. The proportion of the drawn-off discharge will depend on the particular design. The side weir, with properly designed baffles, starts 10 to 15 degrees from the beginning of the bend at the outer wall. Its length will depend on the design, but it could become a double weir downstream of the end of the bend, i.e., after the heavily polluted sewage is decanted or drawn off. Surface debris collects at the end of the chamber and passes over a short flume to join the sewer conveying the flow to the treatment plant.

The authors of the model study report that even the simplest application of the spiral-flow separator will produce a cheap regulator which will be superior to many existing types. They also stated that further research is necessary in order to define the variables, the limits of applications, and the actual limitations of the spiral-flow regulator.

The principal advantage of this device is that two relatively flat, reverse curves, produce effective helical motion which may provide quality separation characteristics. This application may be economically significant in existing space-limited combined sewer interceptor junction locations.

1.12.5 Stilling Ponds

The stilling pond regulator as used in England comprises a short length of widened channel which acts as a stilling basin, from the bottom of which the flow to the plant is discharged. The flow to the plant is controlled either by use of an orifice on the outlet in the chamber or by use of a "throttle pipe,"—i.e., an outlet pipe designed so that it will be surcharged in wet-weather periods. Its discharge will depend on the sewage level in the regulator. Excess flows during storms discharge over a transverse weir and are conveyed to the river. The use of the stilling pond provides time for the solids to settle out when the first flush of storm water arrives at the regulator before discharge over the weir begins. In England it is generally assumed that the first flush will carry the greatest concentration of solids. This first flush concept is not universally accepted in North America.

The performance of this regulator was investigated in England (Reference 2). The experimental structure is shown in Figure 1.12.5. The size of this structure was considered suitable for a domestic population of 2,000. Discharge to treatment was 0.9 cfs at first spill over the weir. At maximum inflow to the regulator of 7.4 cfs the flow to treatment was 1.06 cfs. Tests were made with no scum board and with a scum board set 6 and 18 inches from the weir. The best results were obtained with the scum board set 6 inches from the weir when the ratio of screenings in the overflow to the screenings in the flow to the plant was 0.69.

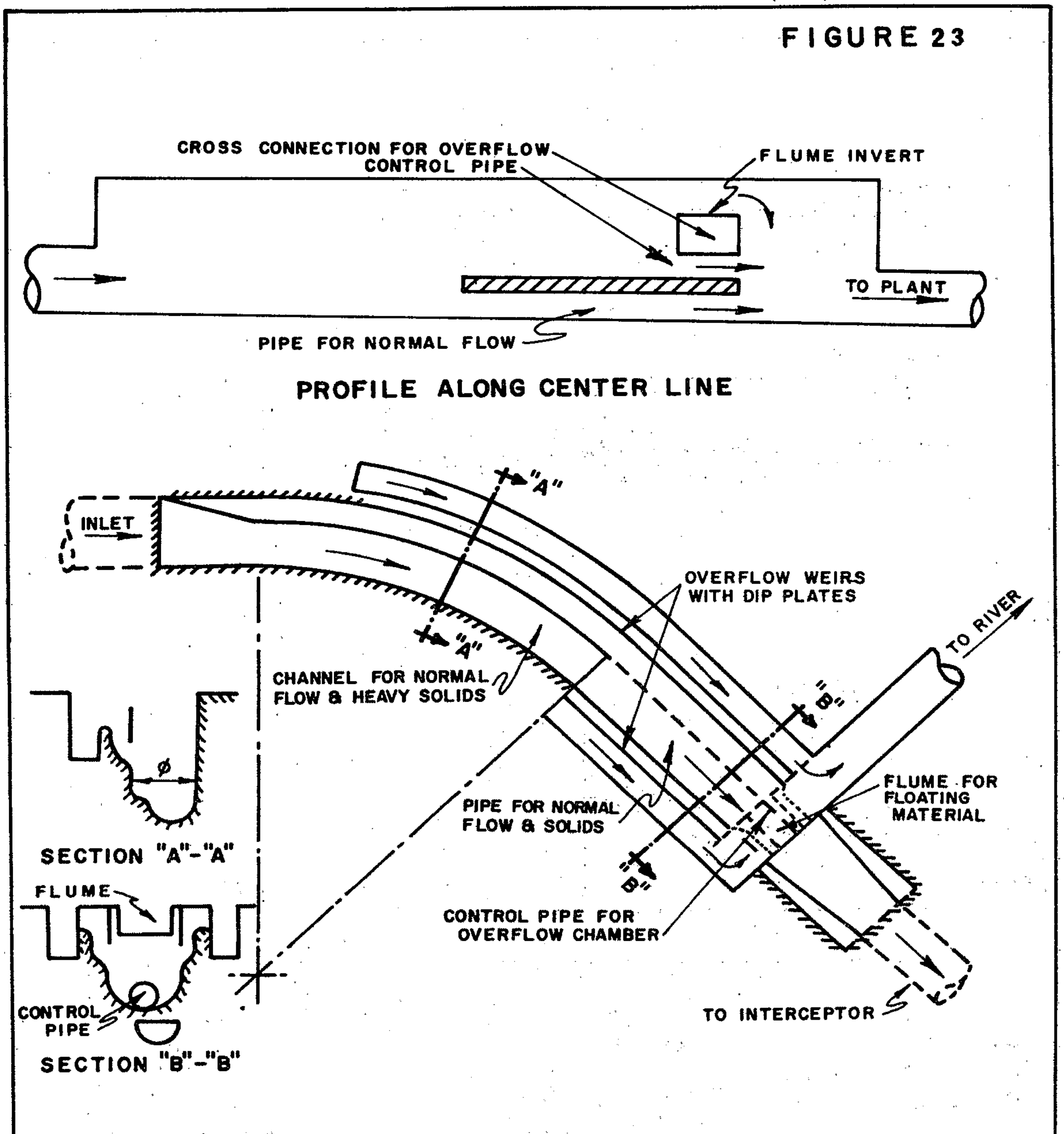
A possible application of this type regulator is shown in Figure 1.12.5 when it is desired to construct the chamber in an existing combined sewer.

This type regulator is considered suitable for overflows up to 30 cfs (Reference 4). If the stilling pond is to be successful in separating solids it is suggested that not less than a 3-minute retention be provided at the maximum rate of flow (Reference 4).

1.12.6 High Side-Spill Weirs

Unsatisfactory experience with side-spill weirs in England has led to the development of the high side-spill weir, referred to there as the high double side-weir overflow. The weirs are made shorter and higher than would be required for the normal side-spill weir. The rate of flow to treatment may be controlled by use of a throttle pipe or a mechanical gate controlled by a float.

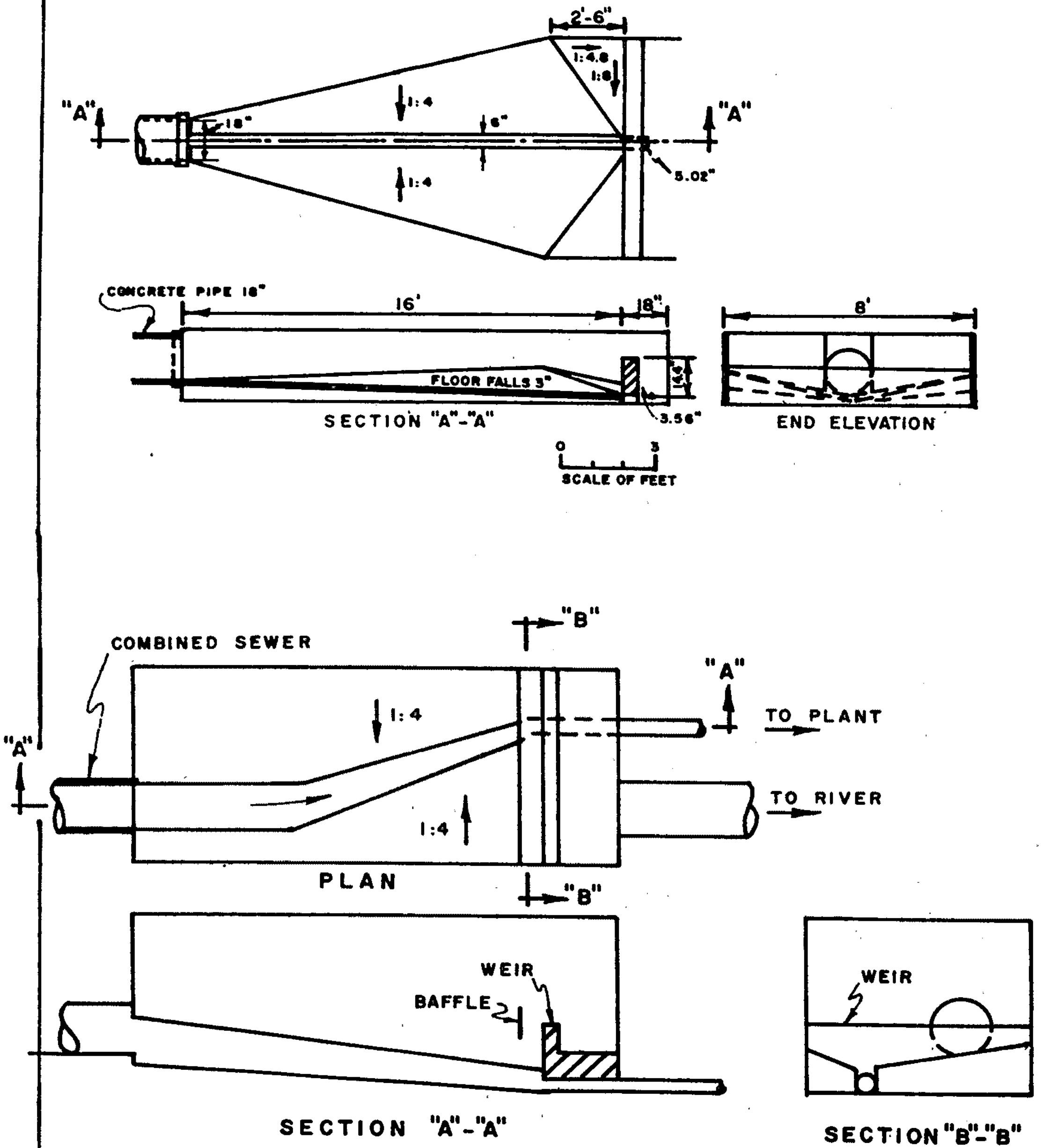
The performance of this type of regulator was investigated in England (Reference 2). The experimental structure is shown in Figure 1.12.6. The structure was sized for a population of roughly 2,000. Discharge to treatment was 0.94 cfs at first discharge over weir and this discharge increased to 1.12 cfs



SPIRAL FLOW (HELICAL) REGULATOR

Courtesy Institution of Civil Engineers

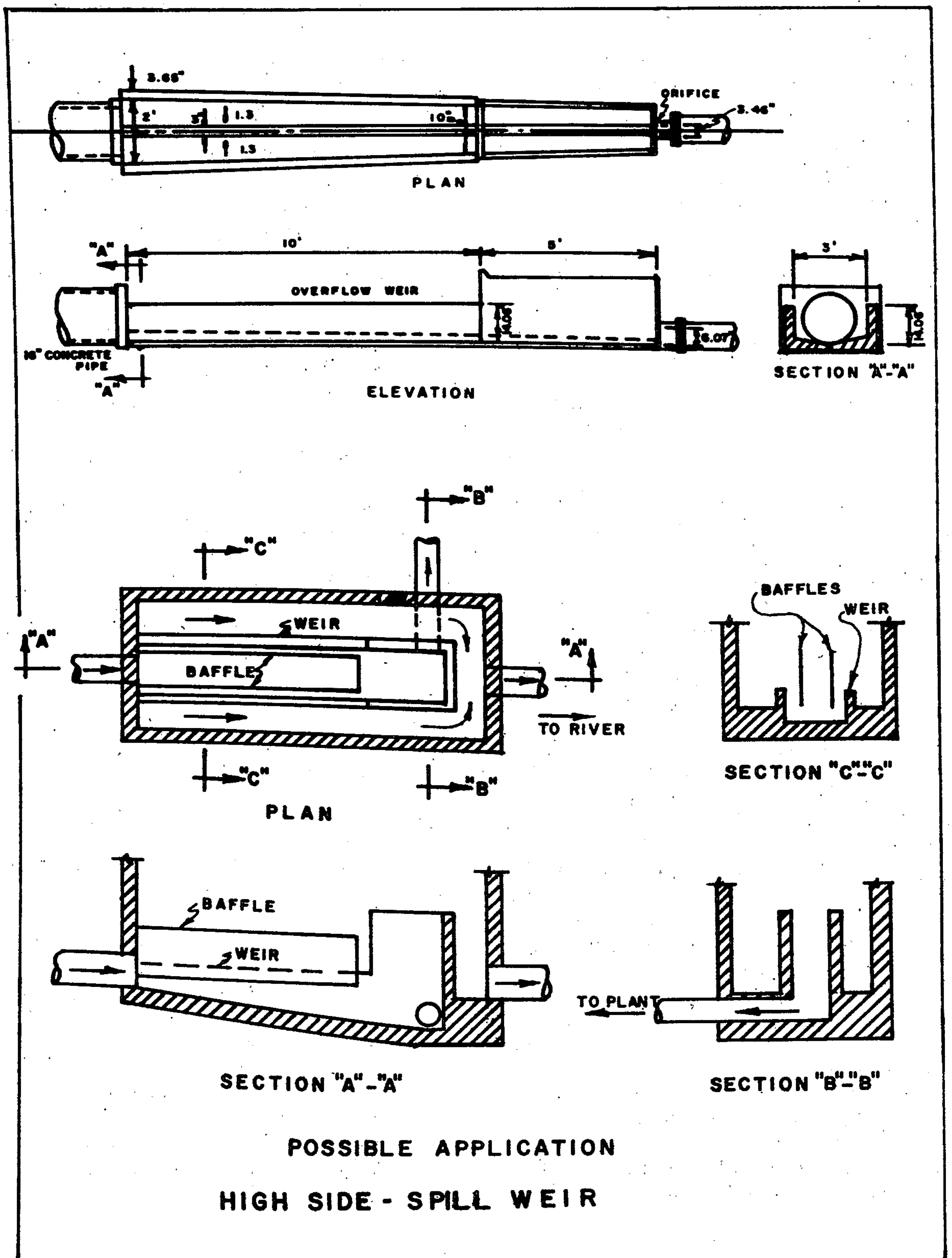
FIGURE 1.12.5



POSSIBLE APPLICATION
STILLING POND REGULATOR

Courtesy Institution of Civil Engineers

FIGURE 1.12.6



Courtesy Institute of Civil Engineers

when total inflow to the regulator was 7.0 cfs. The orifice on the pipe to the interceptor was 6 inches wide by 3 1/2 inches high. The ratio of screenings in the overflow to screenings in the flow to treatment was 0.5, the lowest of the four types tested. This device has the best general performance when compared to the stilling pond, the vortex and the low side-overflow weir.

1.12.7 Broadcrested Inflatable Fabric Dams

The broadcrested inflatable fabric dam is a variably-controlled gating structure manufactured from reinforced rubberized fabric. This reinforced fabric is shaped into a sealed tube, capable of being pressurized with either water, air or a combination of air and water. Each inflatable dam is adapted to and designed to be readily installed and operated on irregular, flat or curved foundation surfaces without affecting the inflatable dam's design discharge characteristics and capabilities. The inflatable dam is installed in a deflated state and therefore assumes the shape and contour of the foundation surfaces. Flow in the combined sewer can be regulated and sewage or storm flow can be diverted to the interceptor by the operation of the inflatable dam. Flow through the interceptor to the treatment plant can also be controlled by inflatable dams or by some other gating device if flow control can be regulated.

Overflow can be regulated by simply increasing the elevation of the inflatable dam by either automatic, semi-automatic or manually operated controls. Only when the capacity of the interceptor has been reached and the level of the admixture of sewage and storm water reaches the storage capacity of the combined sewer system will overflow be allowed to occur. The inflatable dam is a fail-safe gating structure which will not allow clogging and jamming during peak storm periods. The inflatable dam can be controlled so that hydraulic pressure provided by the upstream water level in the sewer conduit will activate a positive deflation mechanism, allowing excess effluent to run off. Then when flows subside and overflow pressures are reduced, the inflation control valves open and the inflatable dam re-inflates until the designed pressure and dam height is reestablished.

Crest control for inflatable dams used to regulate flows in a waterway is based on the relative head between upstream water level and dam inflation pressure. When water is used as the inflation medium, an inverted "U" tube siphon pipe installed in the drain line provides a fail-safe and positive deflation mechanism. The height of the siphon apex is adjustable so that flexibility in settings is possible and deflation can correspond, as desired, to various upstream water levels and flow rates. An air vent is connected to the top of the siphon. With the valve closed, the siphon will prime whenever dam inflation pressure, as increased by rising upstream water level, exceeds the siphon height setting, and continuous complete deflation then takes place. With the apex valve open, the siphon acts as a standpipe and dam deflation will be partial and intermittent, depending upon the rates at which flows build up and subside. Positive deflation control is thus assured under high flow conditions, whether the air valve is open or closed. The siphon serves a secondary purpose in preventing over-inflation during the filling operations.

As flows subside, overflow pressure reduces, the inflation float valve opens, and the dam gradually re-inflates until ultimately the upstream pool returns to normal level and the dam is again at nominal inflated height.

Air-inflated dams operate under the same fail-safe principle as water-inflated dams except that air-actuated instrumentation and controls excite the deflation cycle system. The decision as to whether to use water, air, or a combination of air and water as the best inflation medium for inflatable dams depends upon operating requirements. The best crest control is achieved with water inflation. The use of air, however, usually results in less cost for fabric and control equipment, especially when inflation-deflation cycle time limits must be quite rapid. In addition, air inflation is dictated whenever a dam must remain fully operational during freezing winter conditions.

Figure 1.12.7 is an artist's conception of a regulator facility utilizing the inflatable fabric dam.

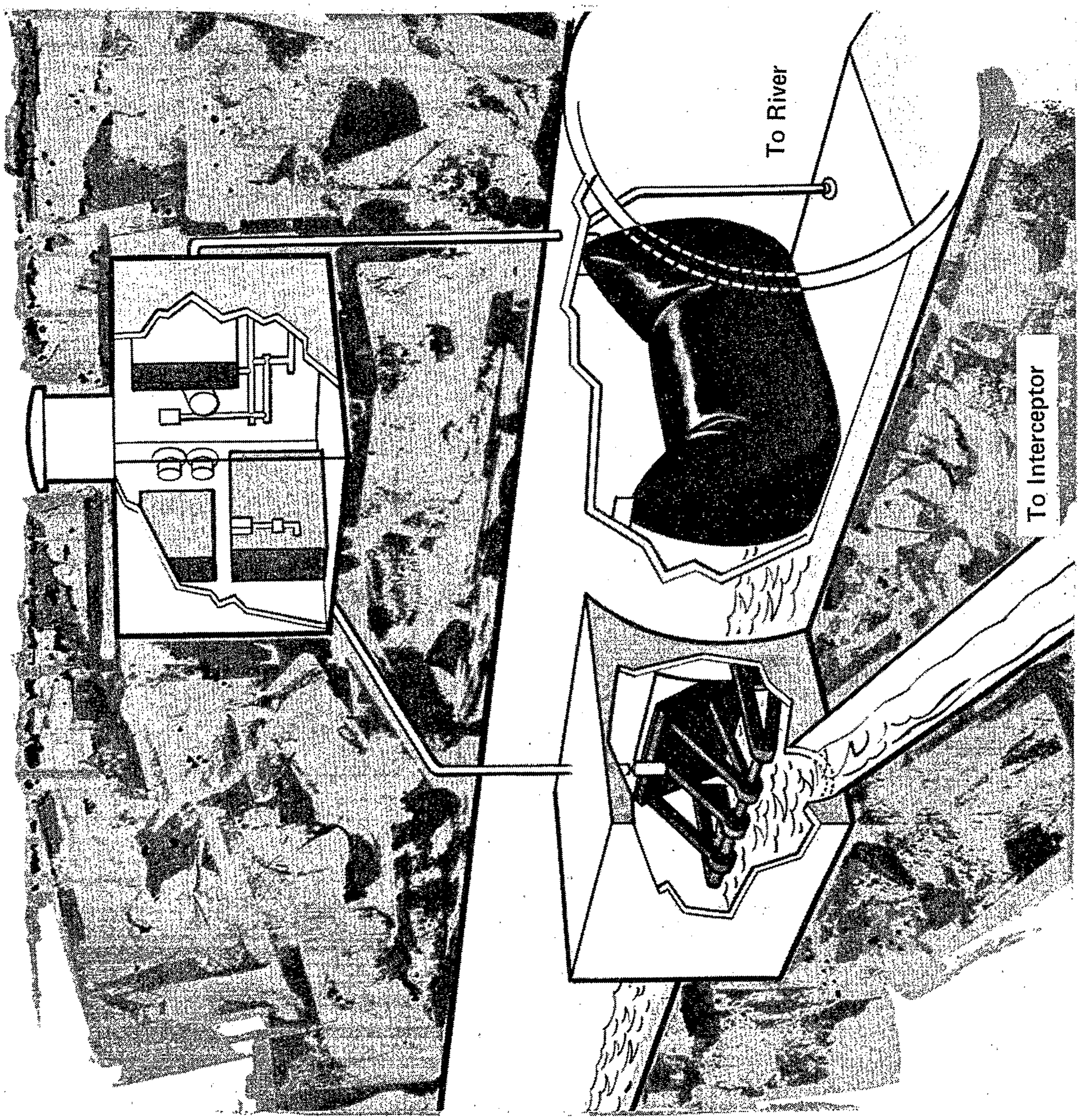
1.12.8 References

All references listed below are from the Symposium on Storm Sewage Overflows, May 4, 1967, sponsored by the Institution of Civil Engineers.

1. Smisson, B., "Design, Construction and Performance of Vortex Overflows".
2. Ackers, P. et al., "Storm Overflow Performance Studies Using Crude Sewage".
3. Prus-Chacinski, T. M. et al., "Secondary Motion Applied to Storm Sewage Overflows".
4. Oakley, H. R., "Practical Design of Storm Sewage Overflows".

FIGURE 1.12.7

ARTISTS CONCEPTION – INFLATABLE FABRIC DAM



Courtesy Firestone Coated Fabrics, Co.

SECTION 2

DESIGN GUIDELINES FOR REGULATORS, THEIR CHAMBERS AND CONTROL FACILITIES

CONTENTS

STATIC REGULATORS

2.1 Manually Operated Gates

2.1.1 Description

The regulator may consist of three chambers: (1) A diversion chamber, (2) orifice chamber, and (3) tide-gate chamber. The last chamber may be omitted when a tide gate is not required. A typical regulator is shown in Figure 2.1.1.

The diversion chamber contains a dam to deflect the dry weather flow at right angles to the sewer axis into the orifice chamber. The diversion dam is usually set at a maximum height of six inches above the invert of the combined sewer to minimize backwater effects upstream in the combined sewer during storm flows. The diversion channel invert is set so that peak DWF can be diverted without overflowing the dam. Excess storm flow will pass over the dam into the flap-gate chamber and continue to the receiving waters.

The gate is set in the orifice chamber on the common wall between the two chambers. The opening is manually adjustable. The minimum dimension of the opening should be four inches to reduce clogging tendencies.

The gate usually consists of a square sluice gate or circular shear gate. The use of the gate has these advantages: (1) The size opening can be adjusted, (2) the gate can be readily opened to clear it of debris; and (3) the gate can be readily closed to stop all flow to the orifice chamber when maintenance is required.

A square or rectangular sluice gate is preferable to a circular one. When a circular gate is partially closed the opening is crescent shaped. This form of opening is more subject to clogging than a square or rectangular opening because material may become wedged in the acute angles at the ends of the crescent.

There is some difference of opinion among designers as to whether the diversion chamber should be constructed with or without a channel. If the channel is used the DWF is conveyed into the orifice with little or no reduction of velocity and there will be no deposition of material in the diversion chamber between storms. If the diversion dam is only six inches above the invert of the combined sewer it will cause little impediment to large solids or floating material during storm flows even if the face of the dam is vertical. Other designers prefer to omit the channel and provide a flat slope on the face of the diversion dam so that storm flows will readily sweep any deposition on the invert up and over the dam. However, during low DWF the pool upstream of the dam will act as a stilling basin and cause grit and solids to accumulate in the diversion chamber. Odors may then become a problem. Since the purpose of the regulator is to convey all sanitary flow including grit and solids to the treatment plant it would appear that the use of the channel is preferable.

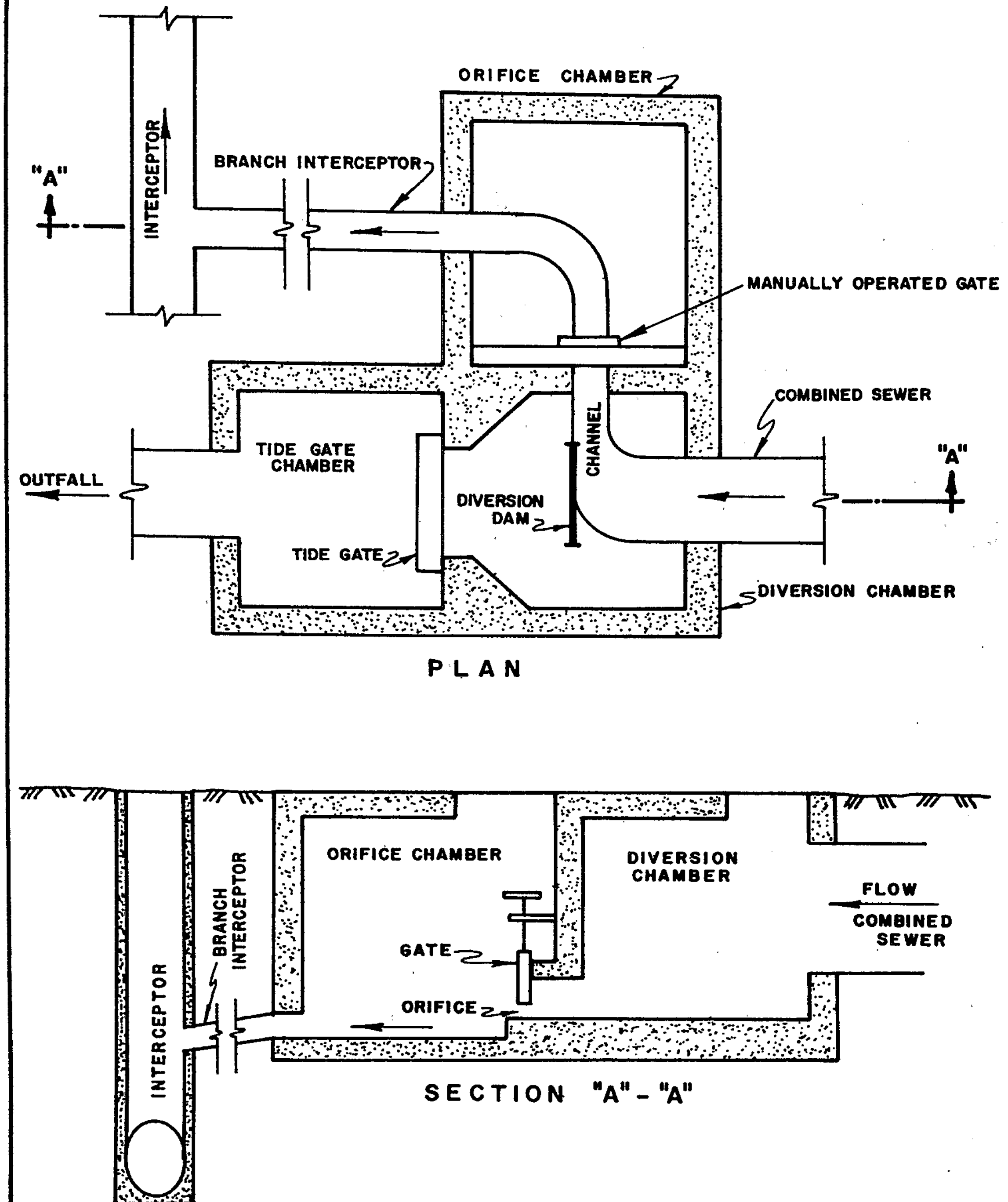
2.1.2 Design Guidelines

A typical layout for this device is shown in Figure 2.1.1. For design, obtain all pertinent data for combined sewer at the proposed location of the regulator including, diameter, invert elevation, slope, average and peak DWF and peak storm flow. The regulator design flow is considered herein to be the peak DWF. Obtain similar data for the interceptor at the proposed junction with branch interceptor. If the interceptor is being designed in conjunction with the regulator assume the elevation for interceptor and adjust as found necessary by subsequent computations.

The gate may be either square, rectangular or circular. The design computations which follow are based on the use of a rectangular orifice. Discharge through the orifice is proportional to the square root of the head on the orifice. Hence a four-fold increase in the head will result in a two-fold increase in the discharge. During low flow periods, discharge will be determined on the basis of critical depth through the orifice, providing the water surface downstream of the orifice is lower. As the depth of flow increases and rises above the top of the orifice, discharge will be based on the difference between the heads on the two sides of the orifice. The orifice chamber and branch interceptor must be designed so that there will be no backwater in the orifice chamber to affect the discharge from the orifice at design flow. The branch interceptor should be designed to carry the peak dry-weather sanitary flow. As the diverted flow exceeds the peak DWF the hydraulic gradient of the branch interceptor will increase, thus raising the water surface in the orifice chamber downstream of the orifice. The resultant submergence of the orifice will reduce the head on the orifice and this will decrease the discharge through it during storm periods.

Thus, the quantity of storm flow diverted to the interceptor is affected by two factors: (1) The effect of the increased depth of flow in the combined sewer on discharge through the orifice is lessened because the discharge is proportional to the square root of the head on the orifice; and (2) the increase in intercepted flow may exceed the capacity at normal

FIGURE 2.1.1



TYPICAL LAYOUT
MANUALLY OPERATED GATE

flow depths of the branch interceptor, thus raising the hydraulic grade line in the orifice chamber and reducing the effective head on the orifice. The second factor becomes more pronounced as the branch interceptor length is increased. It is, accordingly, desirable to locate the regulator some distance from the interceptor—at least 100 feet. Further, to not cause backwater from the branch interceptor in the orifice chamber it is desirable that the flow in the branch be subcritical at design flow.

If field conditions are such that the branch interceptor cannot be designed to meet the foregoing criteria it may be necessary to provide some flow control device in the orifice chamber. This could be a vertical stop log control, as used for the cylinder-operated gate, or an orifice either in the channel or on the outlet pipe from the orifice chamber.

The hydraulic gradient and energy lines for peak DWF should be computed starting with the water surface at the interceptor and proceeding upstream along the branch interceptor to the orifice chamber. The elevation of the control dam in the diversion chamber is usually set 0.5 feet maximum above invert of the combined sewer. The size of the orifice should be selected and the required head to pass the design discharge should be determined. The invert of the orifice should be set at the required elevation and water surface downstream of orifice in orifice chamber should be determined. The water surface should be compared with required hydraulic grade for the branch interceptor. Elevations of the latter and the size of the orifice must be adjusted as required.

The quantity diverted to the interceptor during storm periods is determined by the trial and error method.

In the initial computation, the hydraulic computations should start at the water surface in the interceptor at peak DWF. 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.

Sample computations are given in paragraph 2.1.3 of this manual.

2.1.3 Sample Computation
Manually Operated Gate

Pertinent data

A = Cross Sectional area in sq. ft.
D = Diameter
V = Velocity
d = Depth of flow
Q = Quantity of discharge
b = Width of opening

H_m = Minimum specific energy
HGL = Hydraulic grade line
d_c = Critical depth of flow
g = Acceleration of gravity
C = Coefficient
W. S. = Water surface elevation

Interceptor

D = 36", Invert el. = 10.0, W.S. = 12.4

Combined Sewer

D = 54", Invert el. = 16.00, s = 0.0026
Manning n = 0.013, V (full) = 6.4 fps

Flow	Q cfs	d ft.	V fps	W.S. el.
DWF = Dry-weather flow - av	0.5	0.3	1.8	16.3
Dry-weather - peak	2.0	0.5	2.5	16.5
1-year storm	60.0(1)	2.5	6.7	18.5
10-year storm	100.0(1)	3.6	7.2	19.6

(1) includes peak dry-weather flow

Distance from interceptor to regulator = 100'
HGL = hydraulic grade line
EL = energy line

2.1.3 Manually Operated Gate

	ELEVATION Invert	HGL	EL
Design Q = 2.0 cfs			
See figure 2.1.3.1			
Interceptor - peak dry-weather flow	10.0	12.40	

Determine lowest profile of branch interceptor

Branch interceptor

L = 100′ n = 0.013 Q = 2cfs
D = 10′ S = 0.008
V (full) = 3.7 fps
d/D = 0.8 V = 1.14 x 3.7 – 4.2 fps
$V^2/2g$ = 0.27

	Invert	HGL	EL
Lowest possible invert			
12.40 – 0.8 (0.83)	11.74	12.40	
Pipe outlet loss 1.0 (0.27 – 0) = 0.27			12.67
Friction head 100 x 0.008 = 0.8			

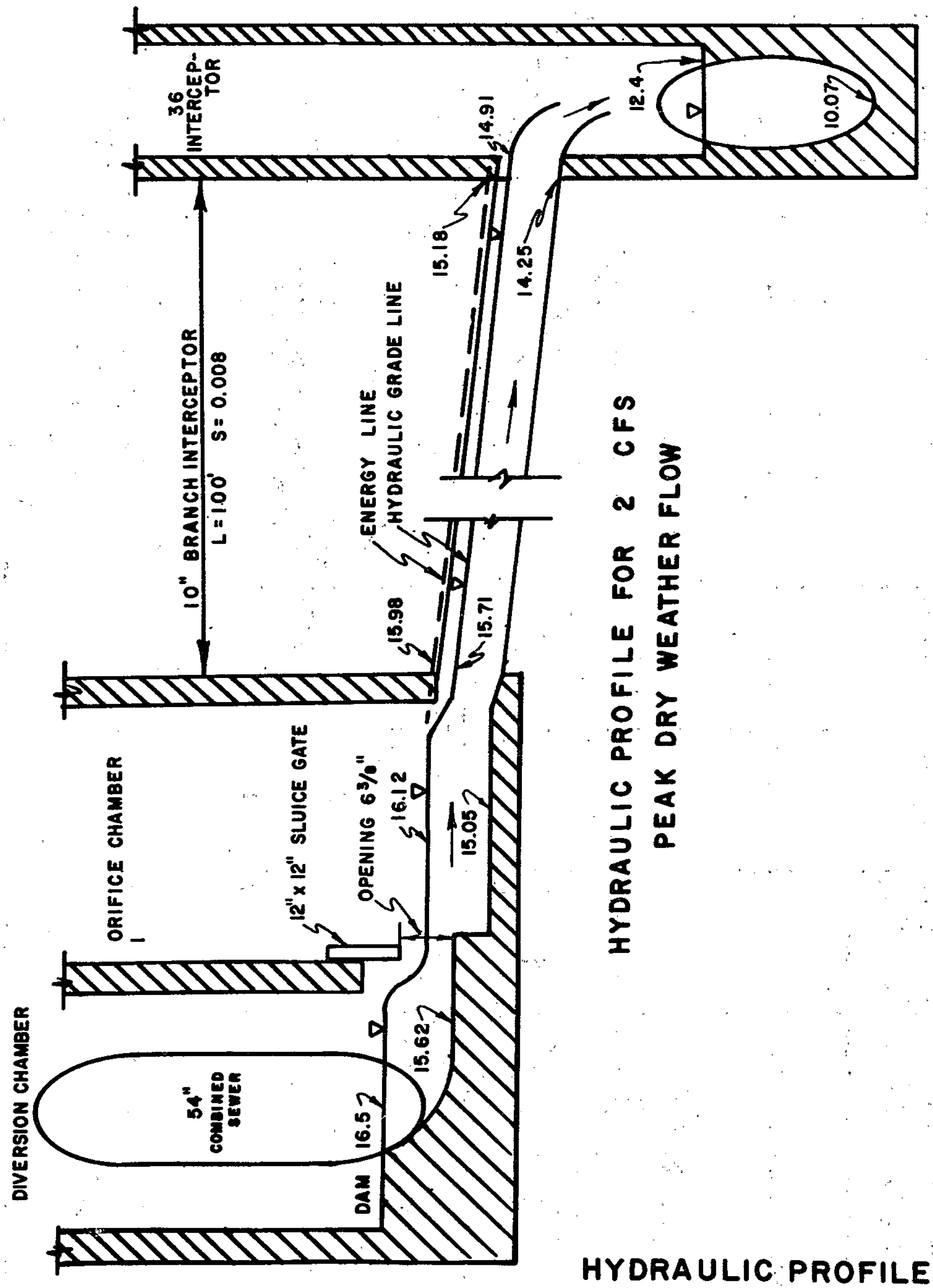
Branch Interceptor	ELEVATION Invert	HGL	EL
Upstream end	12.54	13.20	13.47
At orifice chamber			.14
Pipe inlet loss 0.5 (.27) = 0.14			13.61
Assume complete loss of velocity in orifice discharge. Water surface in orifice chamber same as EL.		13.61	
Diversion Chamber			
Dam = 16.00 + 0.50 = 16.50		16.50	

Try 12″ x 12″ sluice gate
Determine invert elevation

$Q = 3.09\, b\, H_m^{3/2}$
$2.0 = 3.09 \times 1 \times H_m^{3/2}$
$H_m = 0.75$
$d_c = 2/3\ 0.75 = 0.50$
$V_c = \frac{Q}{b\,dc} = \frac{2.0}{1x0.5} = 4.0$ fps
$d = H_m + 0.5 \frac{V_c^2}{2g}$
$= 0.75 + 0.5 \frac{16}{64.4}$
$d = 0.88^1$

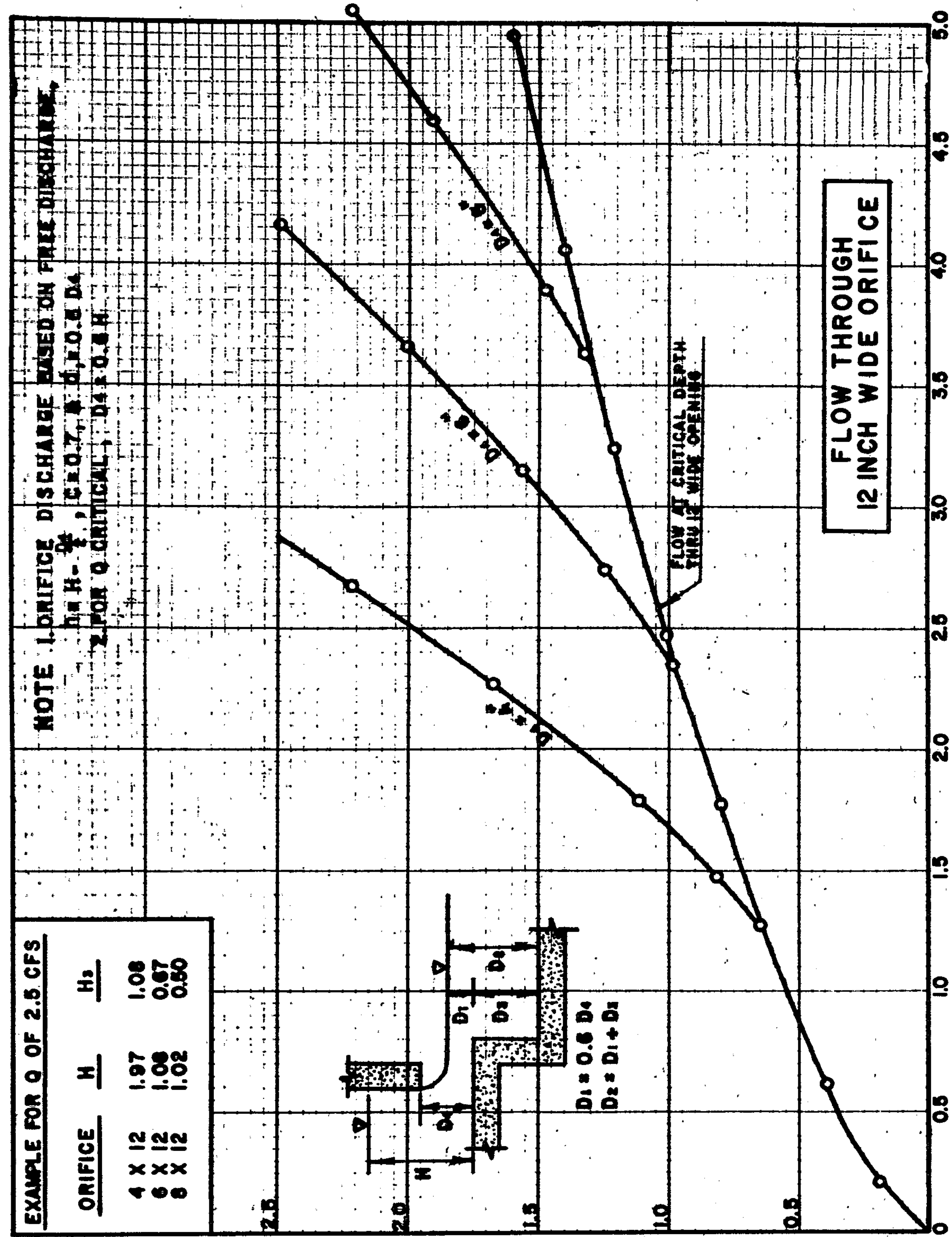
Check – From Fig. 2.1.3.2 $d = 0.88^1$

FIGURE 2.1.3.1



HYDRAULIC PROFILE (1)
MANUALLY OPERATED GATE

FIGURE 2.1.3.2



NOTE 1. ORIFICE DISCHARGE BASED ON FREE DISCHARGE,
h = H - D4/2 , C = 0.7 , & D1 = 0.6 D4
2. FOR Q CRITICAL , D4 = 0.6 H
FLOW AT CRITICAL DEPTH THRU 12" WIDE OPENING
D4 = 4"
D4 = 6"
D4 = 8"
EXAMPLE FOR Q OF 2.5 CFS
ORIFICE H H3
4 X 12 1.97 1.08
6 X 12 1.08 0.67
8 X 12 1.02 0.50
H
D1
D3
D4
D1 = 0.6 D4
D2 = D1 + D3
FLOW THROUGH
12 INCH WIDE ORIFICE
DISCHARGE - CFS
H - FEET
0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
4.0
4.5
5.0

2.1.3 Manually Operated Gate

	ELEVATION		
	Invert	HGL	EL
Set invert of orifice at 0.88[1] below flow line			
16.50 – 0.88 =	15.62		

Orifice Chamber

	Invert	HGL	EL
Orifice			
Invert	15.62		
HGL = 15.62 + d_c = 15.62 + 0.50		16.12	
Assume Velocity head loss			16.12

∴ Branch interceptor can be raised until HGL at upper end is 16.12

Rise = 16.12 – 13.61 = 2.51

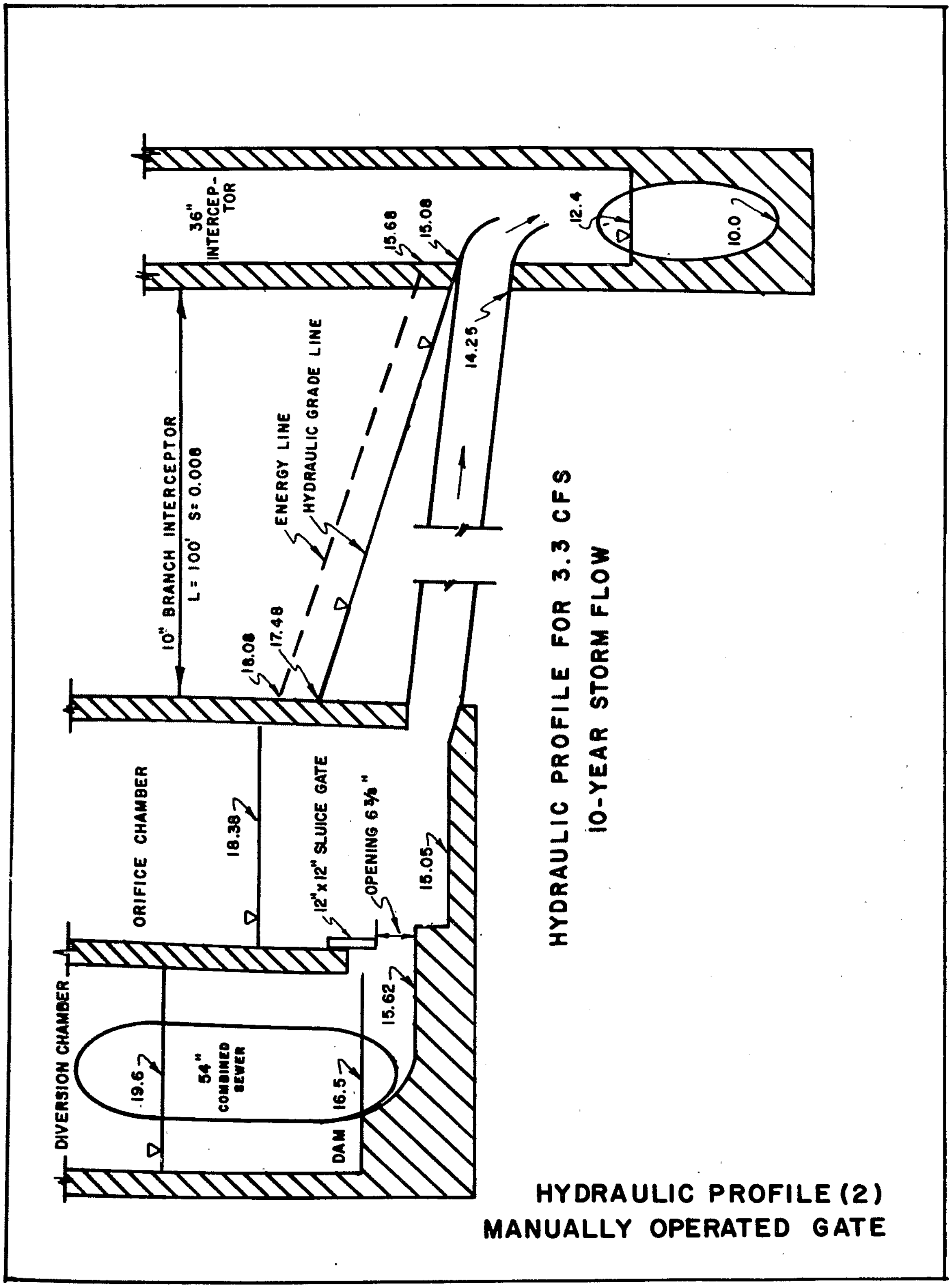
	ELEVATION		
Orifice Chamber	Invert	HGL	EL
Revised branch interceptor—Highest profile			
1./ Downstream 11.74 + 2.51	14.25		
14.25 + (.8) (0.83) = 14.25 + 66		14.91	
Outlet loss $V^2/2g$ = 0.27			15.18
Friction loss 100 x 0.008 = 0.80			
Upstream end	15.05	15.71	15.98
Pipe inlet loss 0.5 x 0.27 = 0.14			16.12
Required water surface		16.12	
2./ Invert of orifice chamber	15.05		

Notes:

1./ Critical depth will occur at downstream end of branch interceptor. However computation of backwater curve indicates normal flow depth will occur 10 feet upstream from outlet. Therefore the computations to determine upstream conditions can ignore critical depth.

2./ This is maximum elevation for invert at chamber and will result in greatest submergence of orifice when diverted flows exceed 2.0 cfs. The invert and branch interceptor can be set lower but this will decrease submergence of orifice at higher flows and will result in greater diversions.

FIGURE 2.1.3.3



HYDRAULIC PROFILE FOR 3.3 CFS
10-YEAR STORM FLOW

HYDRAULIC PROFILE (2)
MANUALLY OPERATED GATE

2.1.3 Manually Operated Gate

Determine flow diverted to branch interceptor in wet weather periods by trial and error

		10-year storm	1-year storm
HGL in combined sewer		19.60	18.50
Q diverted cfs-assume		3.4	2.9
V-fps		6.24	5.3
10" branch S for EL		0.024	0.018
Top branch lower end 1/		15.08	15.08
Exit loss $V^2/2g$		0.60	0.44
Friction loss 100 x S		2.40	1.80
Ent. loss $0.5\,V^2/2g$		0.30	0.22
HGL in orifice chamber		18.38	17.54
Total head (H) on orifice	19.6 – 18.19	1.22	0.96
$Q = 2.81\sqrt{H}$		3.3	2.9
Diverted Q cfs		3.3	2.9

Ratio $\frac{WWF}{DWF} = \frac{WWF}{0.5}$

1./ Critical depth is 0.78′. Backwater computation indicates pipe will be flowing full 2 feet from end. Assume pipe is flowing full at end.

$$Q = CA\sqrt{2gH} = 0.7 \times 1.0 \times 0.53 \times 8.03\sqrt{H} = 2.98\sqrt{H}$$

2.2 Fixed Orifices (Vertical)

2.2.1 Description

The regulator is similar in all respects to the manually operated gate except that no gate is used.

2.2.2. Design Guidelines

The design guidelines for the vertical orifice are the same as those established for the manually operated gate in 2.1. In the description of the latter device it is stated that if the flow in the branch interceptor is not subcritical it may be necessary to install a control in the orifice chamber to cause submergence of the orifice chamber to cause submergence of the orifice and thus reduce the amount intercepted during storm periods. To accomplish this purpose the Allegheny County Sanitary Authority has installed a "double-orifice regulator" which uses a rectangular orifice between the combined sewer and orifice chamber and a circular orifice on the outlet from the orifice chamber. The branch interceptor is designed with sufficient slope so that the outlet orifice functions with free discharge. Sample computations for a single fixed vertical orifice would be similar to those presented in 2.1.3 for a manually operated gate.

2.3 Fixed Orifices (Horizontal)
(The Drop Inlet)

2.3.1 Description

When the horizontal orifice is located in the invert of the combined sewer it may consist of an open slot or an inlet with a metal grating. After droping through the slot or grating the flow is conveyed by the branch interceptor to the interceptor. A dam is required immediately downstream of the orifice to prevent overflows during dry weather periods.

When the horizontal orifice is located in a separate chamber the regulator consists of a diversion chamber, orifice chamber and, when necessary, a tide gate chamber.

The diversion chamber is similar to that used for manually operated gates. The opening in the common wall between the diversion chamber and the orifice chamber should be made large enough so as not to act as a control orifice during dry-weather peak flows. The invert of the diversion chamber can be provided with a channel or a flat bottom.

Dry-weather flow in the combined sewer is diverted by the dam in the diversion chamber into the orifice chamber through an opening in the common wall between the diversion and orifice chambers. The orifice is set horizontally in the bottom of the orifice chamber at sufficient depth below the diversion dam to intercept the design flow. The flow passing through the orifice drops into the branch interceptor which conveys the flow to the interceptor. The orifice may be either circular or rectangular. If circular, provision should be made for replacing the orifice plate, when necessary, to change the size of the orifice. If rectangular, the orifice can be made by using two fixed plates and two removable plates so that the size of the opening can be revised. Stop planks should be provided in the diversion chamber to prevent flow to the orifice chamber when adjustments are made to the orifice.

2.3.2 Design Guidelines

The area of grate to provide for an orifice located in the sewer invert can be determined from the orifice formula, using the head on the grate caused by the dam. It is difficult to decide what allowance should be made for clogging. A reasonable assumption is that 50 percent of the grate opening area is available for flow. Hence, if a storm occurs when the grate is clean an excessive flow may be intercepted. On the other hand, the first rush of storm flow may carry so much debris that the grate becomes clogged very quickly. For the foregoing reasons it is considered preferable to place the horizontal orifice in a separate chamber.

During dry weather flow the hydraulic gradient at the upstream end of the branch interceptor should be below the orifice for proper functioning of the regulator. If the hydraulic grade line is just below the orifice during dry weather flows and the branch interceptor is designed for subcritical flow, then storm flows will cause the hydraulic grade line to rise above the orifice and the flow into the branch interceptor will be governed by the hydraulics of the branch interceptor rather than by the orifice. Since this will reduce the amount intercepted during wet weather periods it is desirable to make the branch interceptor of some length, say at least 100 feet, so as to develop such a backwater effect. It should also be noted that during storm flows the vertical opening between the diversion and orifice chambers will act as an orifice. The height of the opening is usually made large enough so that the vertical opening will have little effect on the size of the flow diverted to the interceptor. The height of this opening could be decreased to further decrease the diverted flow. This in effect, would be designing a double orifice regulator, with one orifice vertical and one horizontal.

2.3.3 Sample Computation
Horizontal Orifice

d = depth of flow
A = area
C = coefficient
L = length
g = gravity acceleration

Given

Interceptor Sewer

Dia. = 36", Invert el. = 10.0
Water surface = 12.4

2.3.3 Horizontal Orifice

Combined Sewer

Dia. = 54", Invert el. = 16.00, S = 0.0026
Manning n = 0.013
V (full) = 6.4 fps

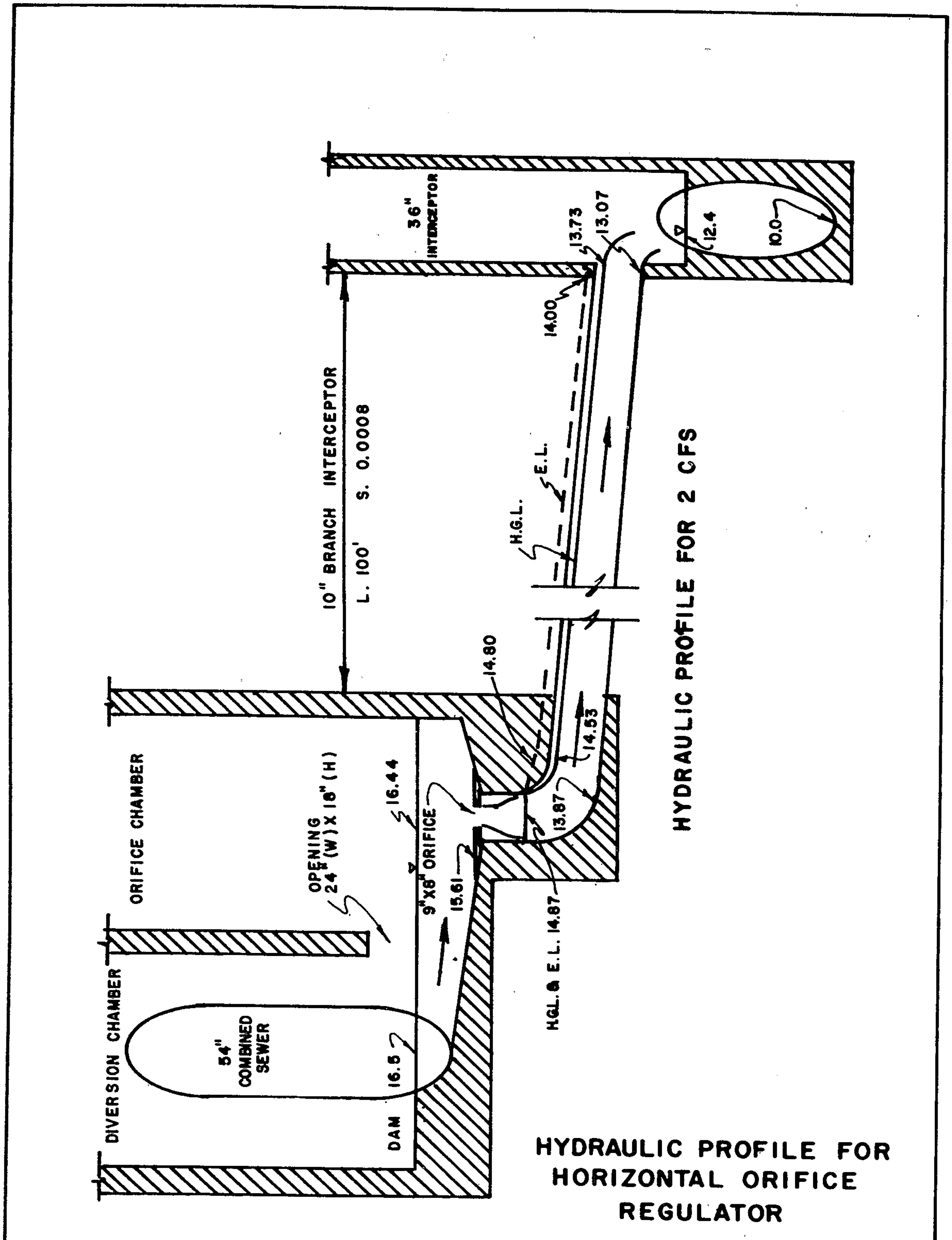
Flow	Q	d	V	WS
	cfs.	ft.	fps	el.
Dry-weather-av	0.5	0.3	1.8	16.3
Dry-weather-peak	2.0	0.5	2.5	16.5
1-year storm	60.0	2.5	6.7	18.5
10-year storm	100.0	3.6	7.2	19.6

Distance from interceptor to regulator = 100 ft.
HGL = hydraulic grade line
EL = energy line

Design Q = 2.0 cfs
See Figure 2.3.3.1

	ELEVATION Invert	HGL	EL
Interceptor	10.0	12.4	
Diversion Chamber			
Design flow in sewer = 16.5 cfs			
Dam elev = 16.5′	16.0	16.5	16.5
= 0.5′ above invert			
Side Opening			
Say 2.0′ wide			
Area = 2.0 x 0.5 = 1.0 sq. ft.			
V = 2.0 ÷ 1.0 = 2 fps			
Flow toward orifice chamber			
90° bend loss $V^2/2g = 0.06$		16.44	16.50
Orifice Chamber			
Assume loss of velocity head		16.44	16.44
Try 9" x 8" orifice			
$Q = CA\sqrt{2gH}$			
$2.0 = (0.6)(0.5)(8.03)\sqrt{H}$			
H = 0.83 ft.			
V = 2.0 ÷ 0.5 = 4 fps			
El. orifice = 16.44 - 0.83	15.61		

FIGURE 2.3.3.1



HYDRAULIC PROFILE FOR HORIZONTAL ORIFICE REGULATOR

2.3.3 Horizontal Orifice

	ELEVATION Invert	HGL	EL
Branch Interceptor			
L = 100′ n = 0.013 Q = 2 cfs			
D = 10″, S = 0.008 V = 3.7, $V^2/2g = 0.21$			
d/D = 0.8, V = 1.13 x 3.7 = 4.2 fps, $V^2/2g = 0.27$			
Downstream End		12.40	
12.40 − (0.8 x .83) = 12.40 − 0.66	11.74		
Exit loss $V^2/2g = 0.27$			12.61
Upstream End			
Friction loss 100 x 0.008 = 0.80	12.54	13.20	13.47
Bend loss 0.25 $V^2/2g$ 0.07		13.27	13.54

Distance of HGL below orifice
15.61 − 13.27 = 2.34

Use 10″ C.I. ASA 21.10 90° bend

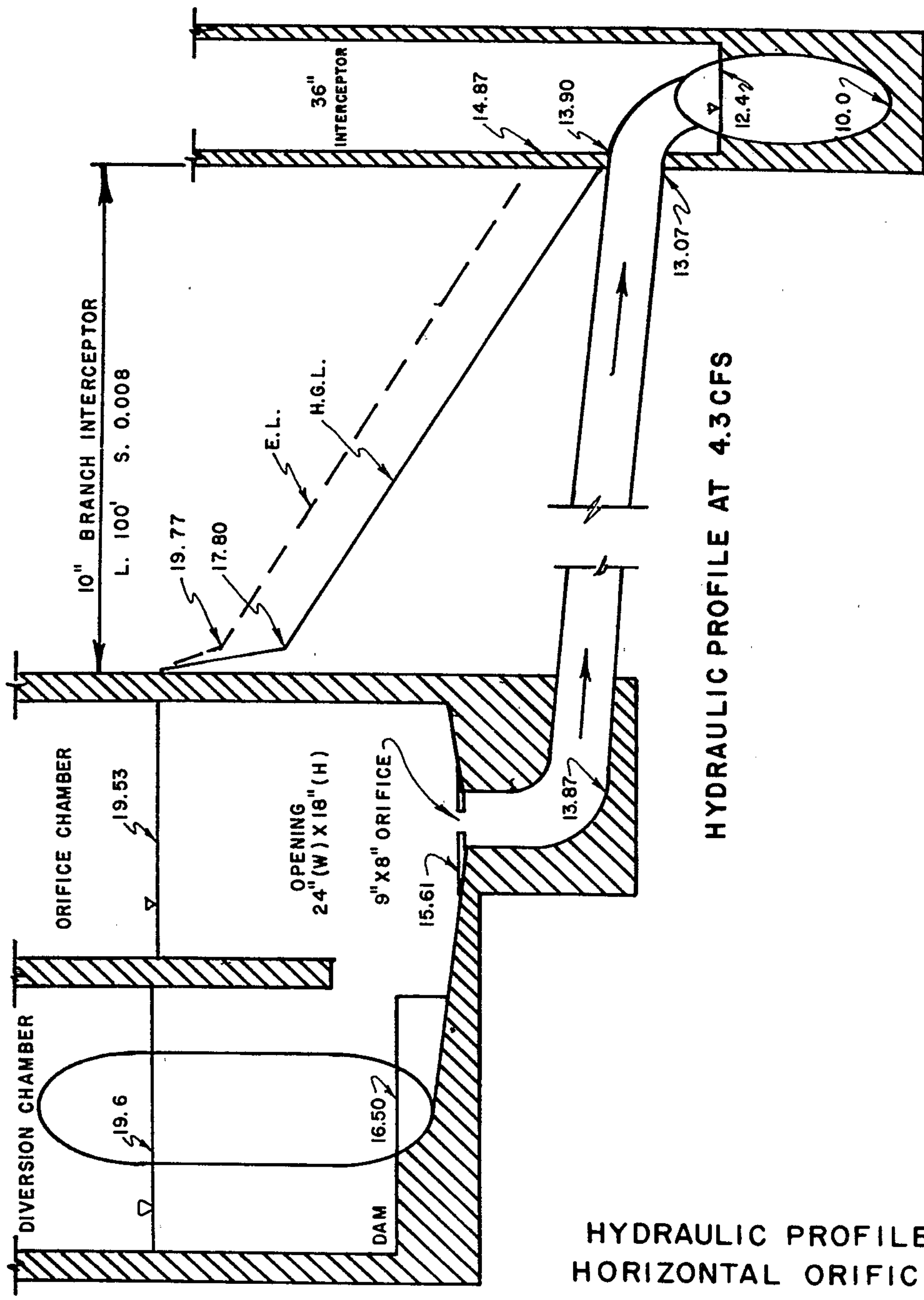
Highest invert = 15.61 − 1.67	13.94
From above inv. = 13.27 − 0.66	12.61
Raise branch interceptor	1.33

Note: Flow at critical depth will occur at downstream end.
Flow at normal depth will occur 10′ upstream.
Therefore, computation of upstream condition can ignore critical depth at lower end.

Revise elevations of Branch Interceptor	ELEVATION Invert	HGL	EL
Downstream end 11.74 + 1.33	13.07	13.73	14.00
	0.80	0.80	0.80
Upstream end	13.87	14.53	14.80
90° bend loss		0.07	0.07
Below orifice		14.87	14.87

Determine flow diverted in wet weather

FIGURE 2.3.3.2



HYDRAULIC PROFILE
HORIZONTAL ORIFICE
REGULATOR

2.3.3 Horizontal Orifice

Figure 2.3.3.2 Estimated		10-year storm	1-year storm
Q	cfs	4.3	3.9
10″	S =	0.039	0.032
	V =	7.9	7.2
Top of sewer downstream		13.90	13.90
Exit loss		0.97	0.81
Friction loss		3.90	3.20
Bend loss		0.24	0.20
Subtotal		19.01	18.11
HGL above horizontal		Yes	Yes
Orifice V		(8.6)	(7.8)
Entrance loss 0.5 $V^2/2g$		0.58	0.47
Enlargement loss		0.02	0.01
HGL in orifice chamber		19.61	18.59
HGL in diversion chamber		19.60	18.50
2.0′ x 1.5 opening			
$\sqrt{}$ = Q ÷ (0.7) (3) (8.03); H =		0.07	0.05
HGL in Orifice Chamber		19.53	18.45
Difference in HGL		0.08	0.14
		Close enough	Close enough
Ratio WWF: DWF = WWF/0.5		8.6	7.8

2.4 Leaping Wiers

2.4.1 Description

Leaping weirs are of two types: (1) continuous invert type; and (2) stepped invert type.

The continuous invert type has no drop in the invert at the horizontal orifice in the bottom of the sewer to change the elevation of the invert. The stepped invert type has the upstream invert raised above the downstream invert. Since regulators usually are constructed on existing combined sewers in which no drop has been provided, this requires that a plate with a raised lip be installed on the upstream side of the opening..

Some designs provide for the installation of adjustable plates to modify the size of the opening and thus the amount of intercepted flow. However, considering the effect of bridging and clogging of the weir with debris, the necessity of making such close adjustments is questionable. It is also doubtful whether such adjustments are ever made after completion, due to the difficulty of operating nuts or bolts in a constant stream of sewage.

2.4.2. Design Guidelines

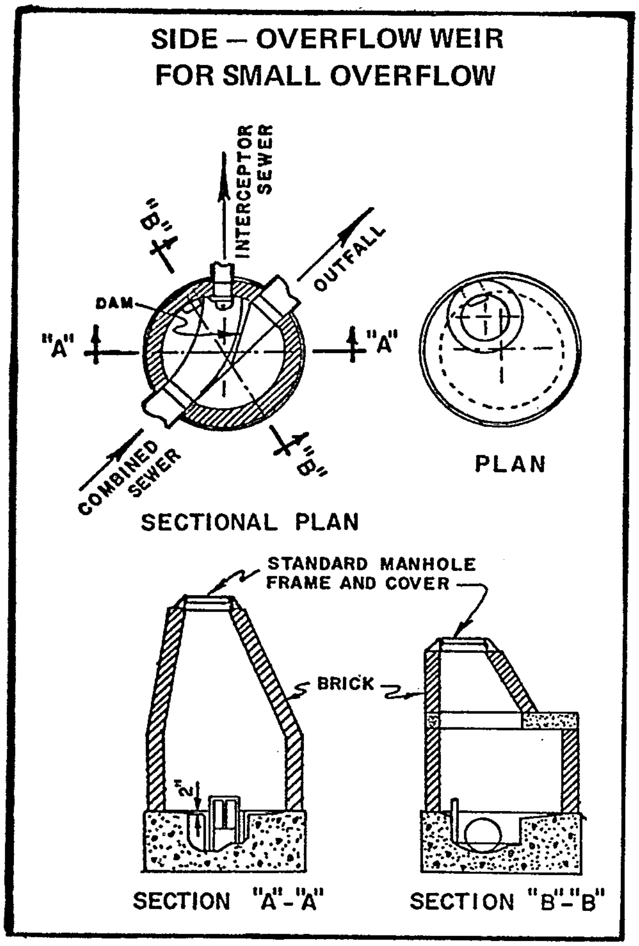
There are no generally accepted design criteria for a leaping weir. Several design methods are given in standard text books and designers are referred to these for further information.

2.5 Side-Spill Weirs

2.5.1 Description

The small regulator shown on Figure 2.5.1 illustrates how a side-spill weir can be constructed in an existing manhole. In the case shown, the designer also has added a manually operated gate at the outlet to the interceptor to further regulate the diverted flow. It should be noted that too great a restriction on the outlet may make the side-spill weir formula inapplicable.

FIGURE 2.5.1.1



Courtesy Institution for Civil Engineering

Studies recently have been carried out on the performance of side-spill weirs in England. (Ackers, P., et al., "Storm Overflow Performance Studies Using Crude Sewage.") The first spill occurred at an inflow of 0.54 cfs compared with the design figure of 0.90 cfs. The maximum flow of 0.54 cfs compared with the design figure of 0.90 cfs. The maximum flow to treatment was 1.4 cfs when the total inflow was 6.5 cfs. When the orifice was removed from the outlet pipe the flow to the interceptor was 2.6 cfs when the total flow was 8.2 cfs. The report states: "Attempts to calculate the discharges for this overflow from classic side-weir theory failed to give satisfactory agreement with observed values." It further states: "The poor degree of control achieved with the low side-weir overflow confirmed previous opinion. It spilled prematurely, as well as failing to limit flows to the desirable maximum."

A possible application of the side-spill weir using a double weir is shown in Figure 1.12.6

2.5.2 *Design Guidelines*

Various formulas have been presented in the past for the design of side-spill weirs. However, they have failed to be accepted and little or no data on actual performance in the field have been presented to confirm their validity. Two methods are described herein which appear to offer more reliability than previous formulas.

The characteristics of flow over side-spill weirs are related to the type of flow in the main channel. The work of investigators who dealt with this problem indicates that when the depth of flow in the main channel or pipe is at or below critical depth, with the wier height lower than critical depth or when the flow depth is greater than critical in a steep slope channel and the weir height is above critical depth, the surface curve of flow over the side-spill weir will be lower downstream. When the channel or pipe flow is at a depth greater than critical depth in a channel on a flat slope and the height of the weir crest is greater than critical depth, the surface curve over the side-spill wier will be lower at the upstream end but will rise downstream. For purposes of hydraulic analysis the assumption is made that the specific energy line H_O referred to the channel invert remains horizontal for the length of the side-spill weir. The error due to this assumption is generally within acceptable limits.

a. Determination of Weir Discharge by Use of Q Curve

A method for determination of side-spill weir flow proposed by de Marchi[5] utilizes the concept of the Q curve which is a graphical representation of the changes that occur in a flowing stream or channel of constant cross section of flow for a fixed value of the energy level H_O.

The specific energy (referred to low point of cross section, i.e. invert of channel) is generally expressed as:

(5) Energia Elettric, July, 1941

$H_o = d + Q^2/(A^2 \times 2g)$
H_o = Energy line level
d = Depth of flow
Q = Quantity flowing
A = Area of flow
Assume the H_o value to be fixed, then
$Q = A[2g(H_o - d)]^{1/2} = 8.02A\,[H_o - d]^{1/2}$

To construct the Q curve, let d vary from zero to H_o and for each increment compute value of Q. Plot computed values of Q on horizontal axis vs. corresponding value of d on vertical axis. For a given fixed H_o the maximum value Q will be at $d_c = (H_o - d_m)^{1/2}$ where d_m is the mean depth. For a rectangular section $d_c = 2/3\ H_o$; for a triangular section $d_c = 4/5\ H_o$; similarly for other cross sections. The value of d_c is the critical depth. For a given energy level H_o, the maximum discharge will be at a depth d_c if the Q flowing is the necessary quantity to support that flow depth.

The following is based in part on a presentation by Prof. K. Woycicki in his book "Kanalizacke", published in 1955.

In the case of channels in which flows upstream of the side weir are at depths greater than "d_c", the determinations of diversion over the weir are started from the downstream end of the side weir. In cases where the flow depth upstream of the side weir is lower than "d_c", the determinations should begin from the upstream end of the weir. The Q curve is drawn from the channel section immediately upstream of the side-weir. The length of weir is estimated for the first calculation and then adjusted by trial. Generally, known formulas may be used for first trial. A side-spill weir flow formula adjusted by a safety factor may be utilized for trial purposes.

$Q = 2.0\,l\,h_m{}^{3/2}$ (cfs)

(h_m = Mean value of head on weir)

For computation purposes the side weir is divided into an equal number of parts—L_1, L_2, L_3, etc. The above weir formula is also used for calculation of the partial flows. Horizontal lines are drawn to the Q curve to represent the level of the side-spill weir and the maximum flow elevation in the channel upstream or downstream as the case may be. The flow elevation downstream is determined in relation to the desired maximum flow to be delivered to treatment facilities, employing usual procedures. In the case of an upstream flow depth greater than critical, computations begin at the downstream end of the side-spill weir. The downstream water surface elevation, as above determined, minus the elevation of the weir crest, gives the weir head h_1 from which the partial flow is calculated; Subtracting the first partial discharge from Q on the Q curve establishes the next water surface level which, in turn, determines h_2 on Section l_2 Again Q_2 is computed and the procedure repeated until the entire water surface curve for the side-spill weir is established. As a check, the summation of partial Q's should equal Q_1 Q_2 the desired diversion quantity. In the case of an upstream flow depth less than critical, the procedure of calculation is similar except that computations begin at the upstream end. In the event of lack of agreement, the side-spill weir length must be modified, lengthened or shortened, and calculations repeated until agreement with Q_1 - Q_2 is obtained.

The above-described method entails, of necessity, a cut-and-try procedure because of the unknown varying head conditions on parts of the side-spill weir.

The theoretical considerations of the de Marchi method are predicated on the following:

1. Steady flow conditions exist;
2. Weir is in a long channel of uniform cross-sections;
3. Crest of the weir is parallel to the bed of the channel;
4. Uniform flow exists upstream and downstream of the weir;
5. Energy line is parallel to the bed of the channel; and
6. Discharge over weir may be computed by a weir-type expression.

Alternative Method for Design of Side-Spill Weirs

A more direct determination of weir length was developed by Mr. Peter Ackers in a paper published in the Proceedings of the Institution of Civil Engineers in 1957, titled "A Theoretical Consideration of Side Weirs on Storm Water Overflows."

The formulas developed by Ackers apply only to a falling profile and only when the weir height is less than half the height of the energy line relative to the channel bottom. If these conditions are satisfied the formulas presented by Mr. Ackers offer a rapid approach to the determination of required length for the side weir. This method is inapplicable with a relatively high side weir. The insertion of dip-plates (scum baffles) may greatly reduce the discharge if the clearances are small and formulas for this condition are also developed in the paper. Further this method does not take account of downstream control. Where a controlled outlet exists the method should be applied with discrimination and only where conditions are such that a falling profile would otherwise occur, i.e., c/E_W is less than 1. The paper also discusses the effect of a tapered channel on the method and states that a rate of taper in excess of the ratio of the overflow per unit length to the channel the weir is set relatively low.

The paper develops a general differential equation for the water profile along a side weir and by substitution of certain factors derives formulas for the length of weir based on a selected ratio of upstream head on the weir to downstream head on weir. The theoretical profile is shown in Figure 2.5.2. The formulas are as follows:

Ratio n	Formula for L
5	$L = 2.03B\,(2.81 - 1.55\ c/E_w)$
7	$L = 2.03B\,(3.90 - 2.03\ c/E_w)$
10	$L = 2.03B\,(5.28 - 2.63\ c/E_w)$
15	$L = 2.03B\,(7.23 - 3.45\ c/E_w)$
20	$L = 2.03B\,(8.87 - 4.13\ c/E_w)$

Notes to above: Add 10% for broad-crested weir. Halve length for double-sided weir.

The notation used is as follows:

L = length of weir
h_1 = upstream head on weir
h_2 = downstream head on weir
$n = h_1/h_2$
c = height of weir crest above invert
B = width of channel or dia. of pipe
E = specific energy related to invert
E_w = specific energy related to weir crest
α (alpha) = velocity correction coefficient
β (beta) = pressure correction coefficient.
Upstream of the weir α is 1.2 and β is unity.
Along weir α is 1.4 and β is 0.8

The original paper gave the following formula for computing total head based on flow upstream of the weir.

$$E_w = 1.2\ V^2/2g + (d_n - c) \qquad \text{(Equation 1)}$$

However, in discussion published subsequently Ackers stated the use of this equation "could lead to anomalies." Therefore, he suggests computing the head at the upstream end of weir based on the assumption that $h_1 = \frac{1}{2} E_w$ which results in following:

$$E_w = \alpha Q^2/2_g A^2{}_1 + \tfrac{1}{2}\beta E_w \qquad \text{(Equation 2)}$$

If the area of the water in a rectangular channel, i.e. $B\ (c + \frac{1}{2} E_w)$ is substituted for A_1 equation 2 becomes:

$$[E_w/c] \times [1 + (E_w/2c)]^2 = \alpha/(2-\beta) \times Q^2/(gB^2C^3)$$
(Equation 3)

Figure 2.5.3 b was developed to solve this equation for c/E_w for rectangular channels.

For circular sections the procedure might be the use of equation 1 for a preliminary value of E_w and then the use of equation 2 by trial and error for a more exact value.

Having determined c/E_w and having selected the ratio of n, the required length of weir can be determined by use of Figure 2.5.2, or by the equations given previously:

Example by Ackers Method:

A combined sewer 48 inches in diameter constructed on a slope of 0.002, with Manning n of 0.013 has a capacity of 65 cfs flowing full. The average dry-weather flow is 6.5 cfs. It is desired to divert flows in excess of 2 x DWF

Upstream flow = 65 cfs
Upstream sewer = 48 in. dia.
Flow to be diverted = 52 cfs
d_c = critical depth
d_n = normal depth
D = diameter
Q = sewer capacity
q = design flow

(1) Critical flow depth in 48-inch-diameter pipe vs. maximum flow depth: From Figure 26, ASCE manual No. 37
$d_c/D = 0.60$, $d_c = 0.60 \times 4.0 = 2.4$ feet
$d_n = 0.8 \times 4.0 = 3.2$ feet

Since d_n is greater than d_c, drawdown will occur at side weir.

(2) Determine c
for Q = 2 x DWF = 13 cfs
q/Q = 13/65 = 0.20
d/D = 0.30, d = 0.30 x 4.0 = 1.20 feet
therefore c = 1.20

Weir crest must be set 1.20 feet above invert so that 2 x DWF can continue directly downstream.

(3) Determine c/E_w

Assume Figure 2.5.3 B is applicable for circular channels.

$C/[Q^2/gB^2]^{1/3} = 0.59$

From Figure 2.5.2 B $c/E_w = 0.60$

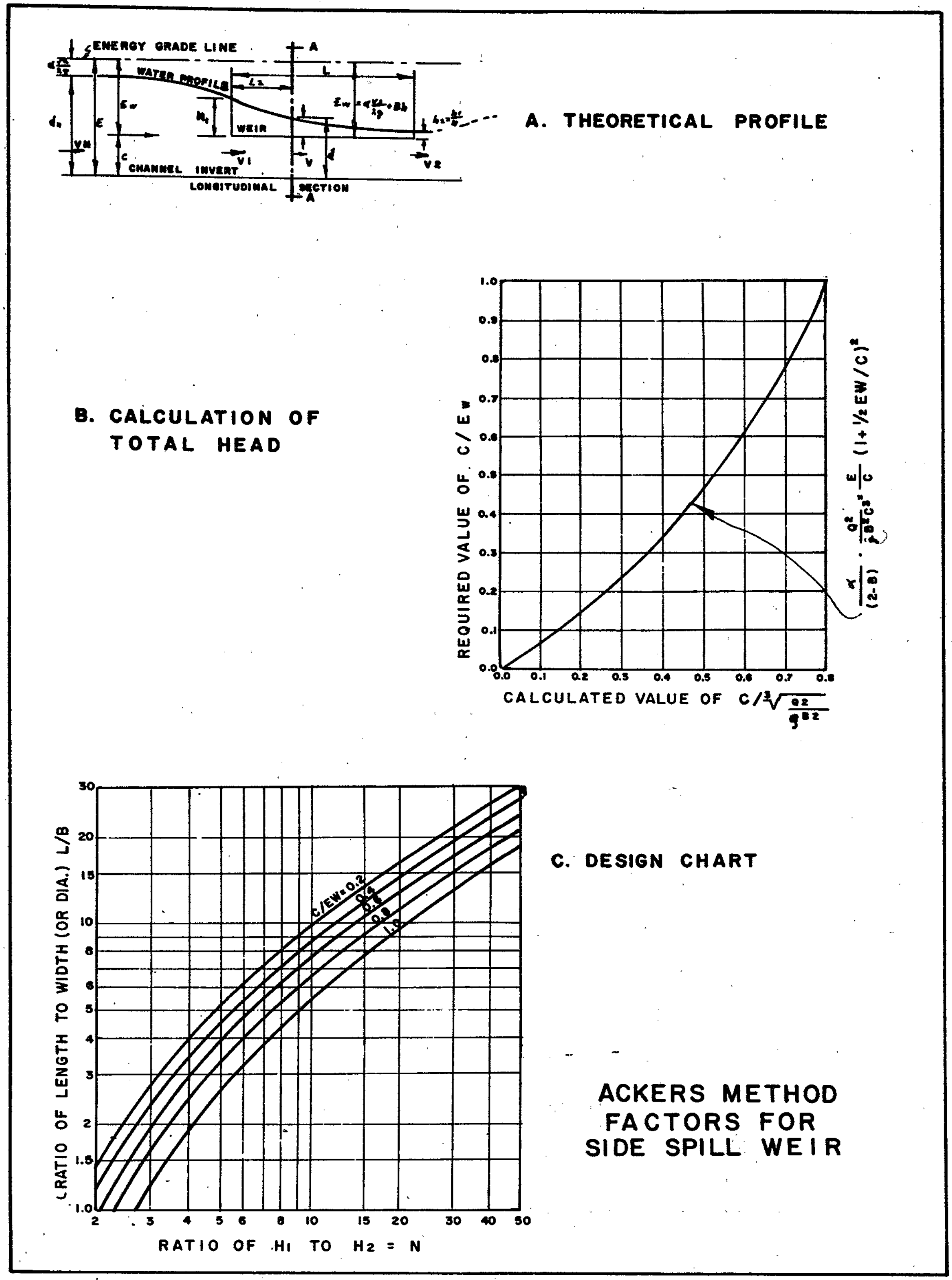
Since c/E_w is less than 1.0 a falling profile will develop.

(4) Determine L
Use n = 5 and $c/E_w = 0.60$
From Figure 2.5.2 c L/B = 3.9
L = 3.9 x 4.0 = 15.6 feet or
7.8 feet per side if double weir is used.

(5) Determine h_1 and h_2
$c/E_w = 0.60$
E_w = 1.20/0.60 = 2.00 = 1.00 feet (double weir)
$h_1 = 0.5\ E_w = 1.00$ feet
$h_2 = 1.00/5 = 0.20$ feet

(6) Determine flow to plant
$d = c + h_2 = 1.20 + 0.20 = 1.40$
d/D = 1.40/4.00 = 0.35
q/Q = 0.26 (from chart of hydraulic properties)
q = 0.26 x 65 = 17 cfs which is greater than 13 cfs desired

FIGURE 2.5.2



Courtesy Institution for Civil Engineers

(7) Determine flow to plant for n = 10

If n = 10 were selected then

L = 7.5 x 4.0 = 30 feet

h_2 = 1.00/10 = 0.10 feet and

q = 15 cfs

2.6 Internal Self-Priming Siphons

2.6.1 Description

Self priming siphons may be classified into two types: (1) Internal self-priming where the siphon action is induced by flow in the siphon, and (2) external self-priming where the siphon action is induced by flow in a priming tube situated outside the siphon. An internal type is shown in Figure 2.6.1. The following discussion relates to internal self-priming siphons.

The internal self-priming siphon consists of: (1) Entrance section; (2) upflow leg; (3) vertical throat section; (4) downflow leg; and (5) outlet section. The downflow leg is designed with an adverse slope to aid in creating a negative pressure at the summit. As the water level rises to the crest of the siphon the water flows over the crest in a sheet and strikes the opposite wall of the downflow leg thus sealing the siphon. As the sheet falls it carries air from the summit with it. When the upstream water level rises enough to seal the air vent the falling sheet of water carries out the remaining air in the summit and the siphon discharges at full capacity. The siphon continues to discharge until the upstream water surface falls below the air vent and enough air is admitted to the siphon summit to stop the siphonic action. For quick priming action it is advisable to provide a water seal at the siphon outlet.

2.6.2 Design Guidelines

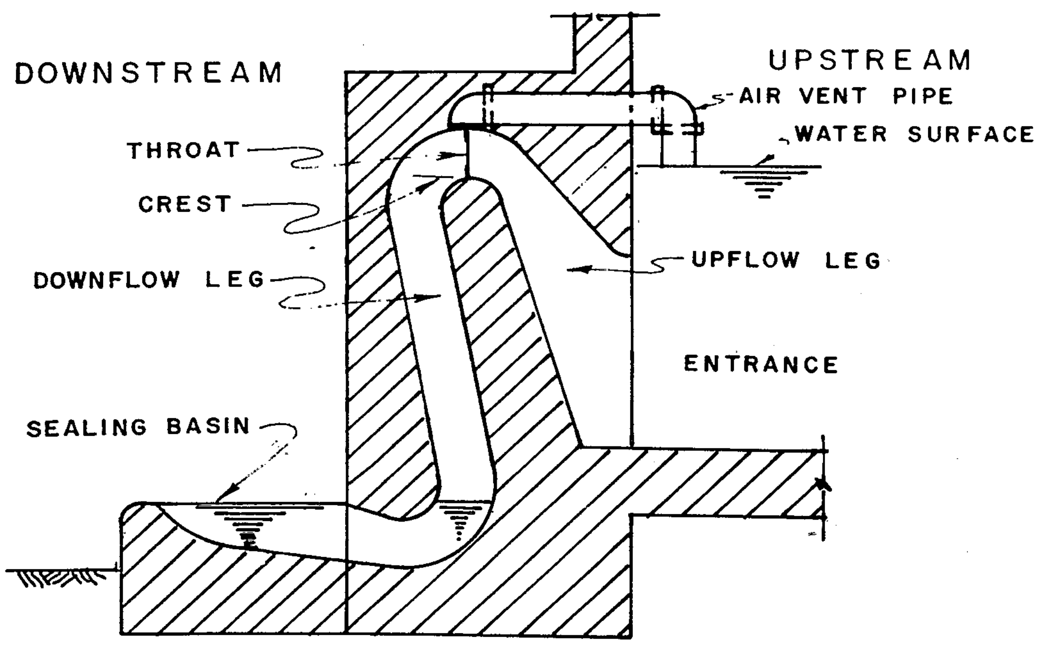
The use of this type of siphon is considered herein for discharging excess storm flows to the receiving waters.

The energy equation for flow through a siphon is:

$H = V^2/2g + K\,V^2/2g + f\,(l + V^2)/(D + 2g)$

where H = difference in elevation of upstream and downstream water surface in feet

FIGURE 2.6.1



INTERNAL SELF-PRIMING SIPHON

v = velocity in fps

f = friction coefficient

l = length of siphon in feet

D = diameter of siphon in feet

K = loss coefficients for entrance, transition, bend and exit losses.

Due to the difficulty in determining the proper loss coefficients in the above equation the following equation also has been used:

$Q = CA (2 gH)^{1/2}$ where

Q = discharge in cfs

C = coefficient

A = area of throat in sq. ft.

H = same as above in ft.

The value of C may vary from 0.3 to 1.0 but generally will range from 0.5 to 0.85 in a well-designed siphon.

Design criteria for siphon and values of the various loss coefficients are given in paragraph 207 of "Design of Small Dams, Bureau of Reclamation, First Edition 1960." Figure 237 of that publication is a chart for determining the value of C for use in an equation similar to the one given above.

Due to the uncertainty in selecting the proper C value, some engineers in the past have recommended this be determined by model test of proposed siphons.

On a recent project in England, in 1958, siphons were made of sheet copper and tested before installation. The copper siphons were then encased in concrete during construction.

The following criteria are based on British experience:

1. The air vent pipe should have a minimum area of six percent of the throat area.
2. A sealing basin at the outlet is necessary for efficient priming.
3. A two-inch depth of flow over the crest at the summit is the maximum necessary to prime the siphon.

DYNAMIC REGULATORS – SEMI-AUTOMATIC

2.7 Float Operated Gates

2.7.1 Description

This regulator may consist of three chambers: (1) Diversion chamber; (2) regulator chamber; and (3) tide gate chamber, when required (Figure 2.7.1).

The diversion chamber contains a dam to deflect the dry-weather flow into the regulator chamber. In flat regions, the diversion dam usually is set a maximum height of six inches above the invert of the combined sewer to minimize backwater effects upstream in the combined sewer during storm flows. The diversion channel invert is established so that peak dry-weather flow can be diverted without flowing over the dam. Excess storm flow will pass over the dam into the tide gate chamber and to overflow.

The regulator chamber contains the float, the regulating gate and the interconnecting linkage between the float and the gate. The gate is installed on an opening in the common wall between the regulator and diversion chambers. Usually the float is situated in a well which is connected by a telltale passage to the channel of the combined sewer in the diversion chamber, or to the channel of the intercepted flow in the regulator chamber.

There are three principal methods of controlling the regulating gate. These are designated herein as Types A, B, and C. In the Type A control the telltale passage extends from the float well to the combined sewer and thus reflects the water level in the combined sewer. This method is used if it is desired to prevent any diversion of flow to the interceptor when the water surface in the combined sewer reaches a certain level.

In the Type B control the telltale passage extends from the float well to the flow channel in the regulator chamber. This type of control is used if it is desired to divert a certain quantity to the branch interceptor before reducing the amount diverted from the combined sewer.

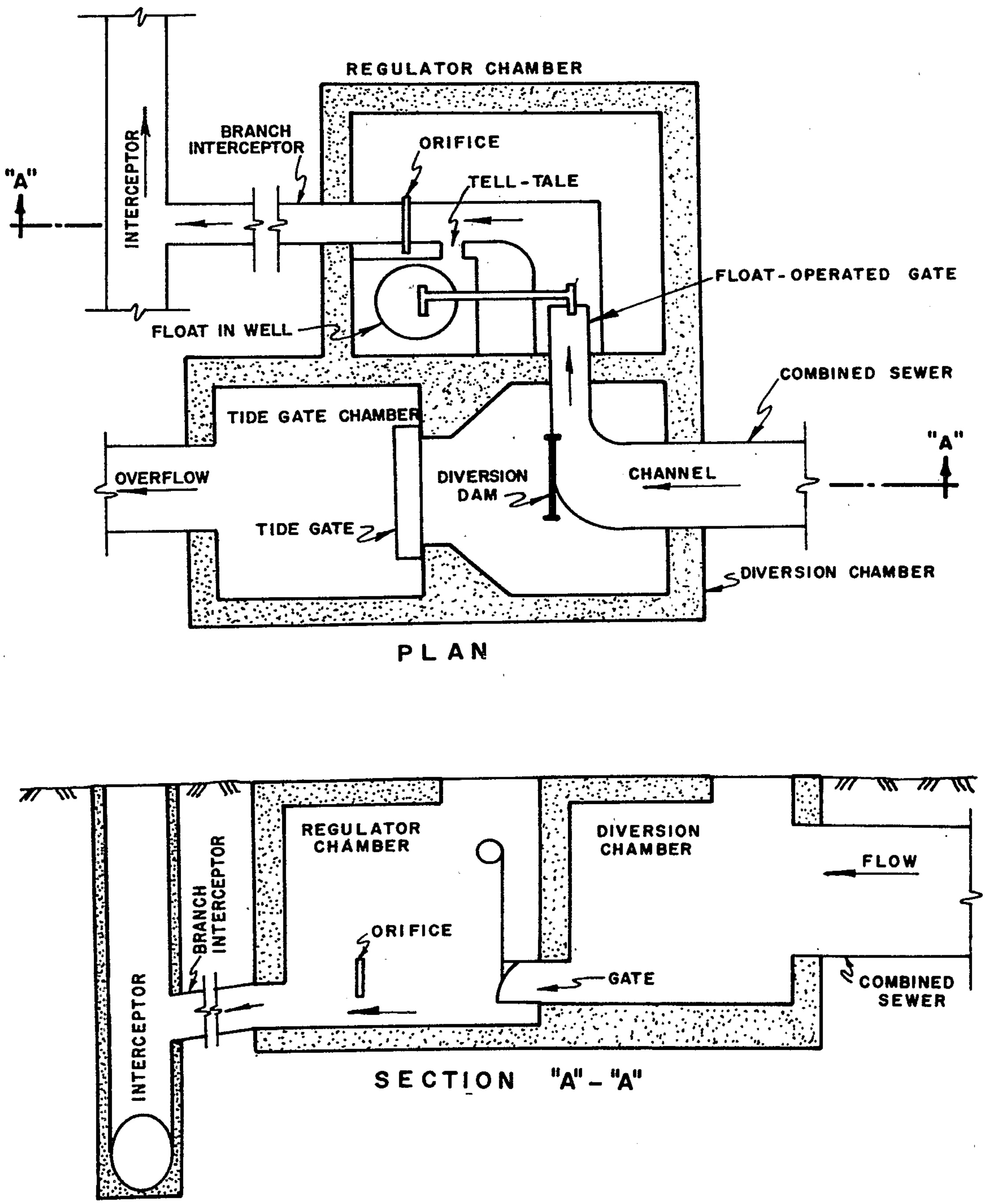
The Type C control is similar to Type B except that an orifice plate is installed in the flow channel downstream from the entrance of the telltale to the channel. This type of control is used if it is desired to pass a predetermined quantity through the regulator regardless of conditions in either the combined sewer or the interceptor. This type of control generally can be designed so that the desired discharge can be controlled within plus or minus 5 to 10 percent. Type C control is considered the most desirable and is used herein for illustrative purposes. A typical layout of a regulator using the Type C control is shown on Figure 2.7.1.

2.7.2 Design Guidelines and Formulas

Type C Control

For design, obtain all pertinent data for the combined sewer at the proposed location of the regulator including diameter, invert elevation, slope, average and peak dry weather flow and peak storm flow. Similar data must be obtained for the interceptor at the proposed junction with the branch interceptor. If the interceptor is being designed in

FIGURE 2.7.1



TYPICAL LAYOUT
FLOAT-OPERATED GATE

conjunction with the regulator, assume the elevation for the interceptor and adjust as necessary by subsequent computations.

First, set the limits on the maximum flow to be diverted to the interceptor. Usually the object is to divert the peak dry-weather flow to the interceptor and hence this value is selected as the maximum dry-weather discharge through the regulator. This represents the maximum discharge with the gate fully open and only dry-weather flow in the combined sewer. Then the maximum discharge through the regulator in wet weather will be the peak dry-weather flow plus a minimum amount varying 10 to 20 percent of the peak dry-weather flow—the maximum discharge through the partially closed gate with storm flow in the combined sewer. This minimum variation between the maximum dry-weather diversion and the maximum wet-weather diversion is necessary to provide adequate variation in the water surface in the float well to cause sufficient float travel as explained hereafter.

For dry weather conditions the total available head loss is divided between h ft., the head loss through the gate and H ft., the head loss through the orifice. Likewise, under maximum storm conditions the total available head is split between h inches, the head loss through the gate and H inches, the head loss through the orifice. The difference in the water surface upstream of the orifice under dry-weather and storm conditions determines the float travel. The difference in the water surface in the combined sewer under dry-weather and storm conditions determines the amount the gate must close, or the shutter travel. If the ratio of shutter travel to float travel exceeds 2 then: (1) A new head loss must be chosen for the orifice; or (2) the discharge through the regulating gate during storms must be increased.

Computations may be made in the following steps:

1. Design branch interceptor for peak dry-weather flow.
2. Using peak dry-weather flow, determine hydraulic gradient at the exit of the regulator chamber on the following basis:
   a. Determine the water surface in the interceptor. If a drop manhole is required at the interceptor so that flow at critical depth occurs, then investigate to see if the branch interceptor is long enough for the backwater curve to attain normal depth at the upstream end of the branch interceptor. Compute the hydraulic profile upstream to the regulator chamber.
   b. Determine the water surface in the regulator chamber. Select channel width. Determine critical depth. Make flow depth 15 percent or more greater than the critical depth. Determine the energy line, hydraulic grade line and invert.
3. Determine total available head between the hydraulic grade line at the exit of the regulator and the dam elevation in the combined sewer. To minimize backwater effect in the combined sewer during storm flows, it may be advisable to set the dam a maximum of six inches above the invert of the combined sewer.
4. Using about one-half of the total available head as H ft., determine the orifice area. Use $Q = CA\,(2gH_2)^{1/2}$ with $C = 0.70$.
5. Determine the hydraulic grade line immediately downstream of the orifice. Use $d_s = d_2\ [1+(2V_2{}^2/gd_2) \times (1-d_2/d_1)]^{1/2}$ (King 5th, Equation 4-25)

Where d_s = depth of flow downstream of orifice

d_2 = depth of flow downstream below turbulence

V_2 = velocity at d_2

d_1 = height of orifice

Q = quantity

A = area

C = coefficient

6. Determine the total available head on the basis of d_2 above.
7. Determine h ft. (head loss through gate).
8. Select regulating gate. Use $Q = CA\ (2gH)^{1/2}$ with $C = 0.95$ to obtain required area. Select the nearest regulating gate size. On the basis of the size selected, determine h ft. (Head loss through gate.)
9. Check orifice size. Determine new H ft. on the basis of h ft. determined in Step 8. Repeat computation for total available head and orifice size until the error is minor.
10. Establish regulator gate elevations so that the gate is submerged on the downstream side with peak dry-weather flow.

Using the maximum wet-weather diverted flow, proceed as follows:

11. Determine the hydraulic grade line at the regulator chamber exit. Proceed upstream from the upstream end of branch interceptor, selecting trial depths and comparing the energy line in the channel with the sum of the energy line in the branch interceptor and entrance loss.
12. Determine H in., the head loss through the orifice. Use $Q = CA\ (2gH)^{1/2}$ with $C = 0.7$.
13. Check the hydraulic grade line downstream

of the orifice, using equation in Step 5.

14. Determine the hydraulic grade line upstream of the orifice.

15. Determine the float travel (F.T.) as the difference in the hydraulic grade line upstream of orifice for peak dry-weather flow and the maximum storm diverted flow.

16. Determine shutter travel (S.T.), the amount the shutter of the gate must close so that the discharge during storm flow will not exceed the maximum diverted flow. It is necessary to use manufacturer's charts for this computation since the coefficient of discharge for the gate varies with the closing of the gate.

17. Determine ratio of S.T. to F.T. If this ratio is less than 2, design is satisfactory. If ratio is greater than two, then redesign must be made, as stated above.

Sample computations follow. The hydraulic profiles for these computations are shown in Figure 2.7.3.

In the design computations friction head losses in the channels are neglected since these losses are minor. It is also assumed that there is complete loss of velocity head of the flow entering the diversion chamber and of the flow entering the regulator chamber. A 90-degree bend in the channel of the regulator chamber is considered advisable to eliminate the latter velocity head so that the orifice design may disregard the effect of approach velocity.

2.7.3 Sample Computation Float-Operated Gate

Pertinent data

Interceptor sewer

D = 60″, Invert el. = 10.0 ft., W.S. = 14.0 ft. for 34.2 cfs diversion and 13.77 ft. for 30 cfs diversion

Combined sewer

D = 60″, Invert el. = 17.0 ft., S = 0.0022
Manning n = 0.013, V (full) = 6.2 fps

	Q	cfs	d ft.	W.S. El.	V fps
DWF =	Av. dry-weather	15	1.20	18.20	4.3
	Peak dry-weather	30	1.70	18.70	5.2
	Peak storm	120	4.00	21.00	7.0

Distance from interceptor to regulator on combined sewer is 100 ft.

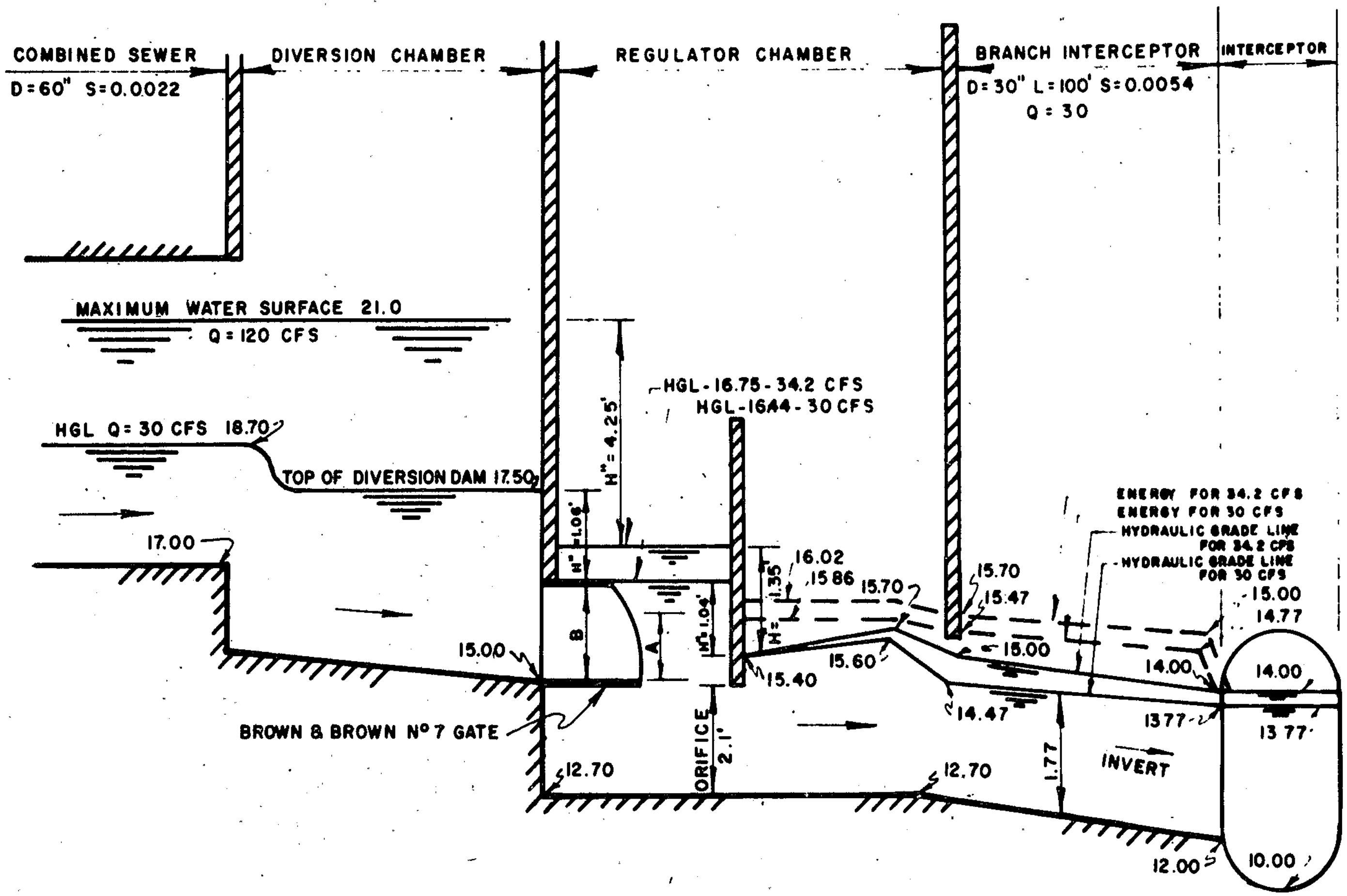
HGL = hydraulic grade line
EL = energy line

Use Brown & Brown Regulator with Type C Control. Maximum diversion will exceed design diversion by 10 to 20%. Use 14%.

h′ = head loss thru regulator gate during dry-weather flow

FIGURE 2.7.3

NOTE: A - GATE OPENING 0.91 FEET DURING STORM FLOW - Q' = 34.2 CFS
B - GATE OPENING 1.33 FEET DURING DRY WEATHER FLOW - Q = 30 CFS.
PROFILE BASED ON BROWN & BROWN TYPE C CONTROL



HYDRAULIC PROFILES FLOAT-OPERATED GATE

2.7.3 Float-Operated Gate

h″ = head loss thru regulator gate during storm flow

H′ = head loss thru orifice during dry-weather flow

H″ = head loss thru orifice during storm flow

d = depth of flow

D = diameter

d_c = critical depth

	ELEVATION		
	Invert	HGL	EL
Regulator should pass peak dry-weather flow			
Design diversion = Q = 30 cfs Maximum diversion = Q′			
Q + 14% = (30) (1.14) = 34.2 cfs = Q′			
Interceptor	10.00	14.00 13.77	
Branch Interceptor Use n = 0.013			
Determine hydraulic profile Q′ = 34.2 cfs Use D = 30 in., S = 0.007 V (full) = 7.1 fps V = 1.13 x 7.1 = 8.0 fps @ 2′ flow depth $V^2/2g$ = 1.00 d = 0.8 x 2.5 = 2.0			
downstream end	12.00	14.00	15.00
Rise = 100 x 0.007 = 0.70			
upstream end	12.70	14.70	15.70

Q = 30.0 cfs
Q/Q^1 = 0.88
d = 0.71 x 2.5 = 1.77 ft.
V = 1.12 x 7.1 = 8.0

2.7.3 Float-Operated Gate

	ELEVATION		
	Invert	HGL	EL
$V^2/2g = 1.00$			
downstream end	12.00	13.77	14.77
Rise = 0.70			
upstream end	12.70	14.47	15.47

Regulator Chamber $\quad Q = 30$ cfs
Use channel width = 2.5 ft.

$d_c = 1.62$ (Fig. 37 ASCE Manual 37)

For stable flow $d = 1.5 \times 1.61 = 1.86$

Determine d at chamber exit by trial for energy balance

	Invert	HGL	EL
$d = 2.9$ ft. $\quad HGL = 12.70 + 2.9$	12.70	15.60	
$V = 30 \div (2.9)(2.5) = 4.13$ fps			
$V^2/2g = 0.26 \quad EL = 15.60 + 0.26$			15.86

Entrance loss = $0.5\,(1 - 0.26) = 0.37$

$EL = 15.47 + 0.37 = 15.84 < 15.86$

Check OK

Neglect friction head loss in channel

Determine total head

Diversion dam = 17.00 + 0.50 =	17.50
W.S. regulator	15.60
Total head—1st trial	1.90

Trial $H' = 1.90 \div 2 = 0.95$

Determine orifice size

$$Q = C A \sqrt{2\,gH}$$

$$30 = (0.70)\,A\,(8.03)\sqrt{0.95}$$

$$A = 5.48$$

Try orifice = 2.5 ft. wide by 2.19 ft. high

2.7.3 Float-Operated Gate

	Invert	HGL	EL
	ELEVATION		

Determine HGL downstream of orifice

$$d_S = d_2 \left[1 + \frac{2V_2^2}{gd_2} \left(1 - \frac{d_2}{d_1}\right) \right]^{1/2}$$

(from King 5th, equation 4—25)

$$= 2.9 \left[1 + \frac{(2)(4.13)^2}{(32.2)(2.9)} \left(1 - \frac{2.9}{2.19}\right) \right]^{1/2}$$

$$= 2.72$$

HGL = 12.70 + 2.72 = 15.42 — Invert 12.70, HGL 15.42 Trial

Determine total head

Diversion dam	=	17.50
HGL downstream orifice		15.42
Total head - 2nd Trial		2.08

Determine regulating gate size

Trial h' = total head - H'

= 2.08 – 0.95 = 1.13

Q = $CA\sqrt{2gH}$

30 = (0.95) (A) (8.03) $\sqrt{1.13}$, A = 3.70 sq. ft.

From Brown & Brown Catalog

Use Gate No. 7 A = 3.81

16 in. high x 34¼" wide

Determine h' based on Gate No. 7

30 = (0.95) (3.81) (8.03) $\sqrt{h'}$

Final h' = 1.06

Re-determine orifice size

H' = 2.08 – 1.06 = 1.02 ft.

30 = (0.7) (A) (8.03) $\sqrt{1.02}$

A = 5.30 sq. ft.

Orifice = 2.5 ft. wide x 2.12 ft. high

Re-check Total head

D_S = 2.70

			Invert	HGL
HGL downstream orifice	=	15.40	12.70	15.40
Diversion dam	=	17.50		Final
Final total head	=	2.10		

2.7.3 Float-Operated Gate

	ELEVATION		
	Invert	HGL	EL
Re-determine orifice size			
Final $H' = 2.10 - 1.06 = 1.04$			
$A = 5.25$ sq. ft.			
Orifice = 2.5 ft. wide			
2.1 ft. high			
Upstream of orifice			
$HGL = 15.40 + H'$			
$= 15.40 + 1.04$	12.70	16.44	16.44
Regulator Chamber			
Determine setting of regulating gate			
Submerge by 0.11 ft.			
Top = 16.44 − 0.11 = 16.33			
Invert = 16.33 − 1.33 = 15.00			
Determine conditions for $Q' = 34.2$ cfs			
Head loss at orifice = H''			
$Q' = CA\sqrt{2gH}$			
$34.2 = (0.7)(5.25)(8.03)\sqrt{H''}$			
$H'' = 1.35$ ft.			
Upstream end of branch interceptor	12.70	14.70	15.70
Determine d at chamber exit by trial for energy balance			
$d = 3.0 + HGL = 12.70 + 3.0$	12.70	15.70	
$V = 34.2 \div (3.0)(2.5) = 4.56$			
$V^2/2g = 0.32$ EL $- 15.70 + 0.32$			16.02
Entrance loss = 0.5 (1.00 − 0.32) = 0.32			
EL = 15.70 + 0.34 = 16.04 > 16.02			
Check OK			

Check HGL downstream of orifice

$$ds = d_2 \left[1 + \frac{2v_2^2}{gd_2} \left(1 - \frac{d_2}{d_1} \right) \right]^{1/2}$$

$$= 3.0 \left[1 + \frac{(2)(4.56)^2}{(32.2)(3.0)} \left(1 - \frac{3.0}{2.1}\right) \right]^{1/2}$$

$$= 2.70$$

2.7.3 Float-Operated Gate

	ELEVATION		
	Invert	HGL	EL
HGL = 12.70 + 2.70 = 15.40		15.40	
HGL upstream of orifice			
HGL = 15.40 + H″ = 15.40 + 1.35		16.75	

Regulator Chamber

Float travel = F.T.

HGL for 34.2 cfs = 16.75
HGL for 30.0 cfs = 16.44
F.T. = 0.31

h″ = 21.00 − 16.75 = 4.25 ft.

From Brown & Brown Catalog

C = coefficient of discharge
P = % of gate opening

$$CP = \frac{Q'}{A\sqrt{2gh''}} = \frac{34.2}{(3.81)(8.03)\sqrt{4.25}}$$

$$= 0.54$$

From chart D.S. 347, Brown & Brown

For CP = 0.54
C = 0.79
and P = 68.5%

Height of gate opening = .685 x 1.33
= 0.91 ft.

S.T. = gate shutter travel
S.T. = 1.33 − 0.91 − 0.42 ft.

$$\frac{S.T.}{F.T.} = \frac{0.42}{0.31} = \frac{1.35}{1} < \frac{2}{1}$$

2.8 Tipping Gates

2.8.1 Description

The regulator structure is similar to that required for manually operated gates. A typical regulator with tipping gate and flap gate is shown in Figure 2.8.1.1.

The detail of the tipping gate as used in Milwaukee about 1919 is shown in Figure 2.8.1.2. The horizontal pivot is located so that one-third of the plate is below the pivot. The housing for the gate is precast concrete and was made with opening widths of 12, 18, and 24 inches.

The detail of the gate as used recently is shown in Figure 2.8.1.3. This gate is made in opening widths of 8, 12, 24, and 36 inches. Where an opening wider than 36 inches is required, multiple gates are used. This model of tipping gate is used by the Allegheny County Sanitary Authority, Pittsburgh, Pa. The gate differs from the gate used in Milwaukee in that: (1) The housing is of cast metal rather than concrete; (2) the bottom third of the plate below the pivot has a deflection angle of about 24 degrees with the upper part of the plate; (3) the top of the housing is curved to provide minimum clearance between the housing and the top of the plate as the latter rotates; (4) the maximum and minimum opening can be adjusted; and (5) the bottom is fixed rather than adjustable.

2.8.2 Design Guidelines

For design, obtain all pertinent data for the combined sewer at the proposed location of the regulator, including diameter, invert elevations, slope, average and peak dry-weather flow and peak wet-weather flow. The design flow is considered herein to be the peak dry-weather flow. Similar data for the interceptor at the proposed junction with the collector are required.

Compute the hydraulic grade and energy lines for peak dry-weather flow, starting at the interceptor and proceeding upstream along the branch interceptor to the tipping gate chamber. Determine the elevation of the diversion dam in the diversion chamber. Determine the differential head available and select the size of gate and gate opening from Figure 2.8.2. If the available differential head is greater than that required for the design flow, the head can be decreased by raising the branch interceptor and raising the water surface downstream of the gate. Since the data for Fig. 2.8.2 are based on a minimum downstream water depth of 10 inches, the tipping gate should be set with its invert 10 inches below the downstream water surface. If the differential head is not large enough to provide discharge equal to the design flow, the branch interceptor should be redesigned, or a larger gate used.

After selecting the gate opening required to pass the peak dry-weather flow, the minimum gate opening during wet-weather periods should be determined. This is done by the trial and error method using the following steps: (1) Assume diverted discharge; (2) determine downstream water surface; (3) determine differential head; (4) select opening from Fig. 2.8.2 for differential head nearest the assumed discharge; and (5) repeat until the assumed discharge is close to the discharge selected from Fig. 2.8.2 and approximates the design diverted flow.

Sample computations are given in paragraph 2.8.4. The given conditions are the same as those used in the sample computations for the manually operated gate. It should be noted that the ratio of peak wet-weather flow (WWF) to average dry-weather flow (DWF) is 4.9 for the tipping gate, compared to 6.8 for the manually operated gate. For this reason the tipping gate is considered preferable to the manually operated gate.

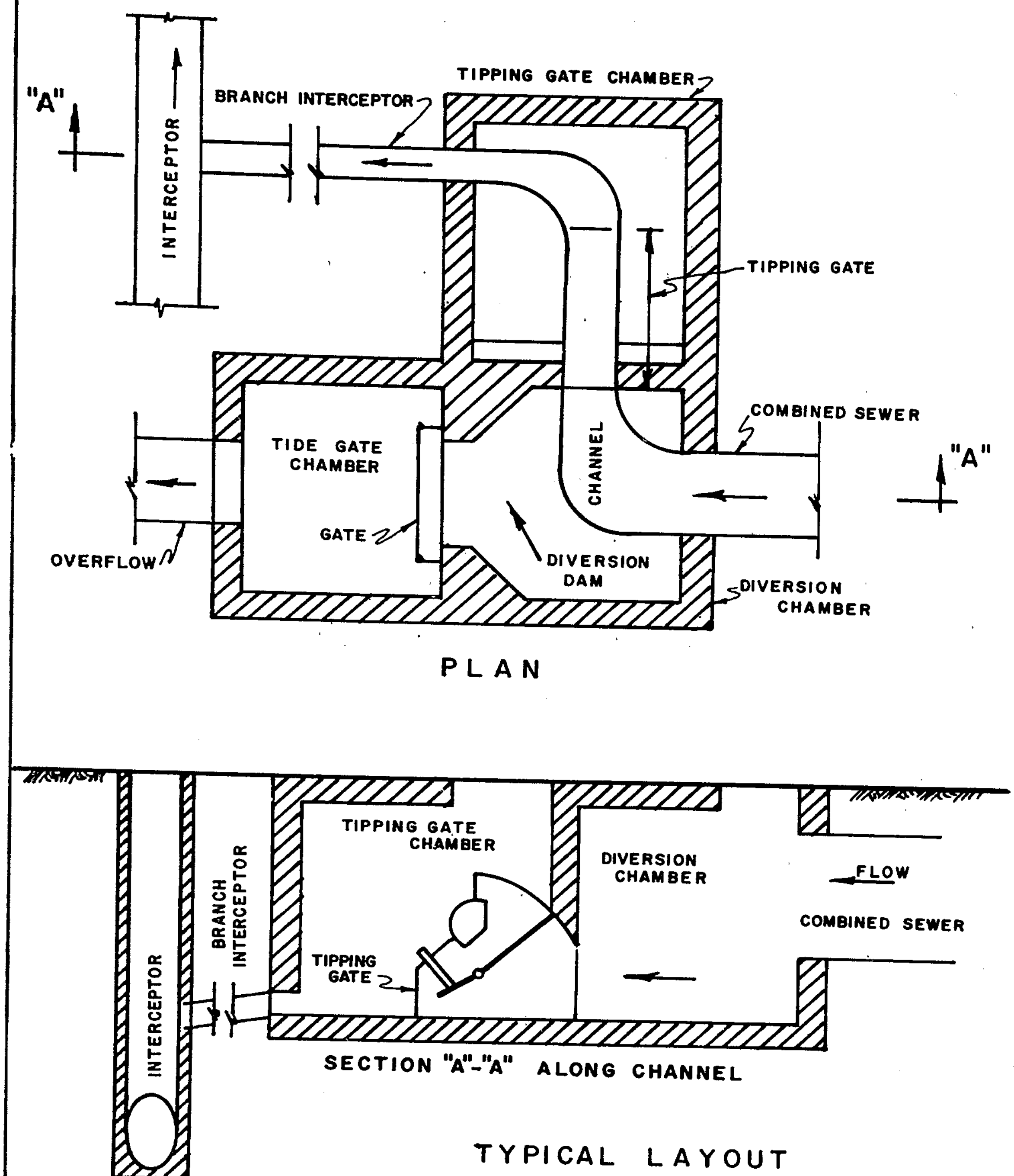
The hydraulic profile for the design diverted flow of 2 cfs is given in Fig. 2.8.4.2. The hydraulic profile for the maximum diversion of 2.45 cfs in storm periods is shown in Fig. 2.8.4.3. Fig. 2.8.4.1 illustrates the hydraulic conditions affecting the gate. At the diversion equal to the peak dry-weather flow of 2 cfs, the differential head (A) is 0.4 feet and the gate opening is 5.0 inches. When the differential head (C) is 0.9 feet, the upstream head is 2.0 feet and the gate begins to close. When the flow reaches its maximum elevation of 19.6 feet in the combined sewer the differential head (B) is 2.89 feet, the opening in the gate has been reduced to 2.56 inches and the flow diverted to the interceptor through the gate is 2.45 cfs.

2.8.3 Design Formulas

In connection with their contract for furnishing tipping gate regulators to the Wyoming Valley Sewerage Project, the Rodney Hunt Company was required by Albright and Friel, the project engineers, to have the gate calibrated by laboratory test. These tests were made at the Alden Research Laboratories, Worcester Polytechnic Institute, Worcester, Mass. The following charts from this report are reproduced through the courtesy of the *Wyoming Valley Sewerage Authority.*

a. Figure 2.8.2 shows the relation between discharge through a 12-inch wide gate and the differential head between the water levels upstream and downstream level constant at 10, 16, 22, and 28 inches and varying the upstream head. Where two curves are shown for a given opening the lower curve represents the flow for a downstream elevation of 10 inches. Otherwise

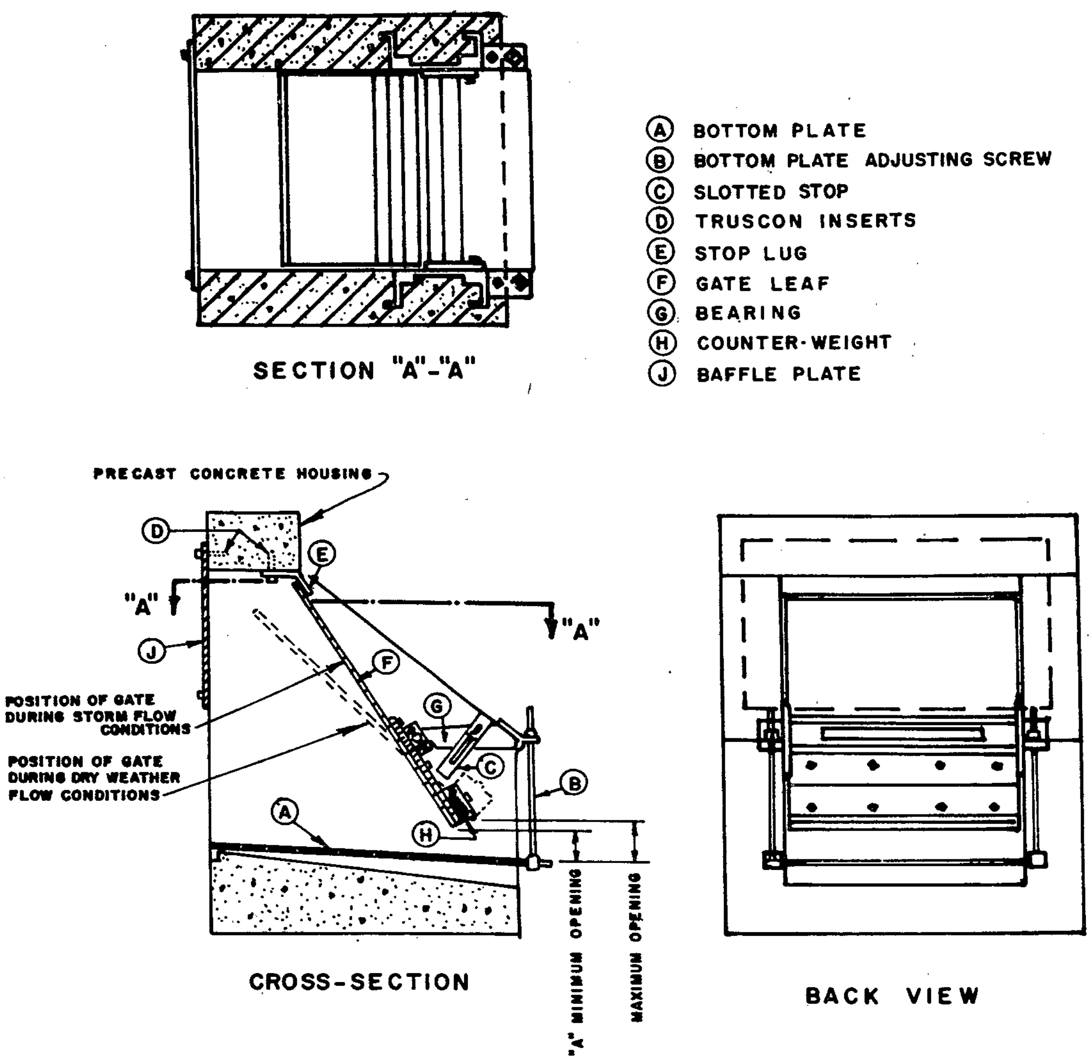
FIGURE 2.8.1.1



TYPICAL LAYOUT
TIPPING GATE REGULATOR

Courtesy Rodney Hunt Co.

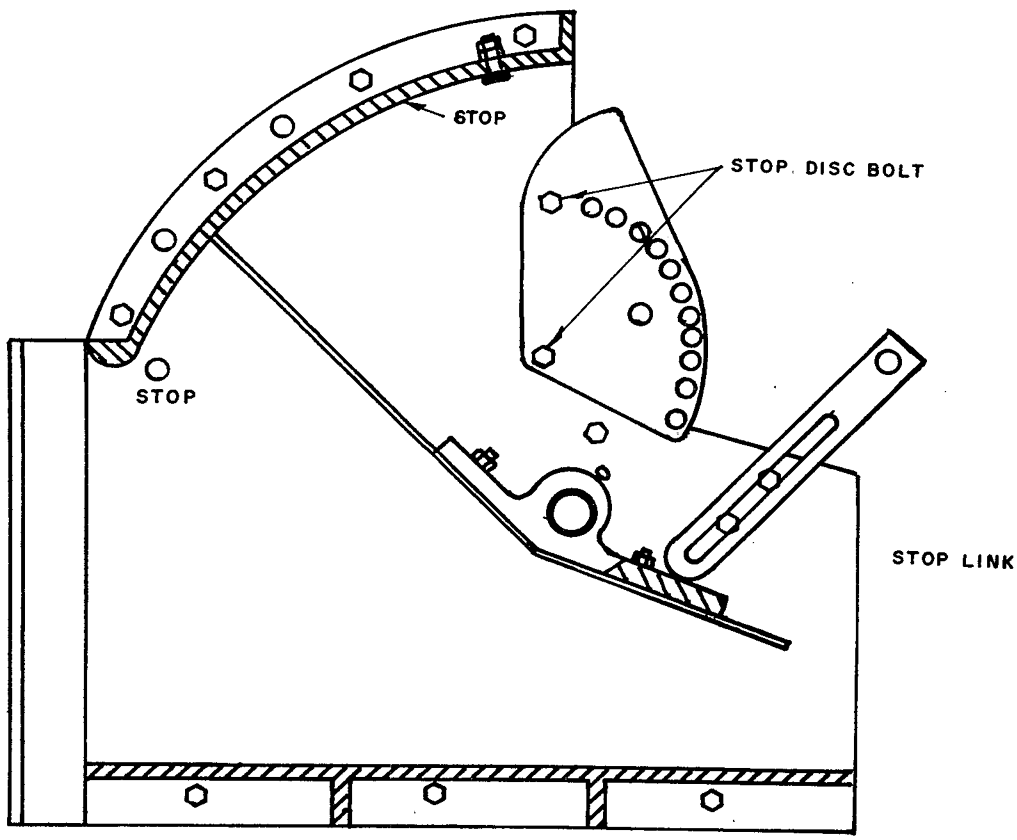
FIGURE 2.8.I.2



TIPPING GATE
USED AT MILWAUKEE

Courtesy Rodney Hunt Co.

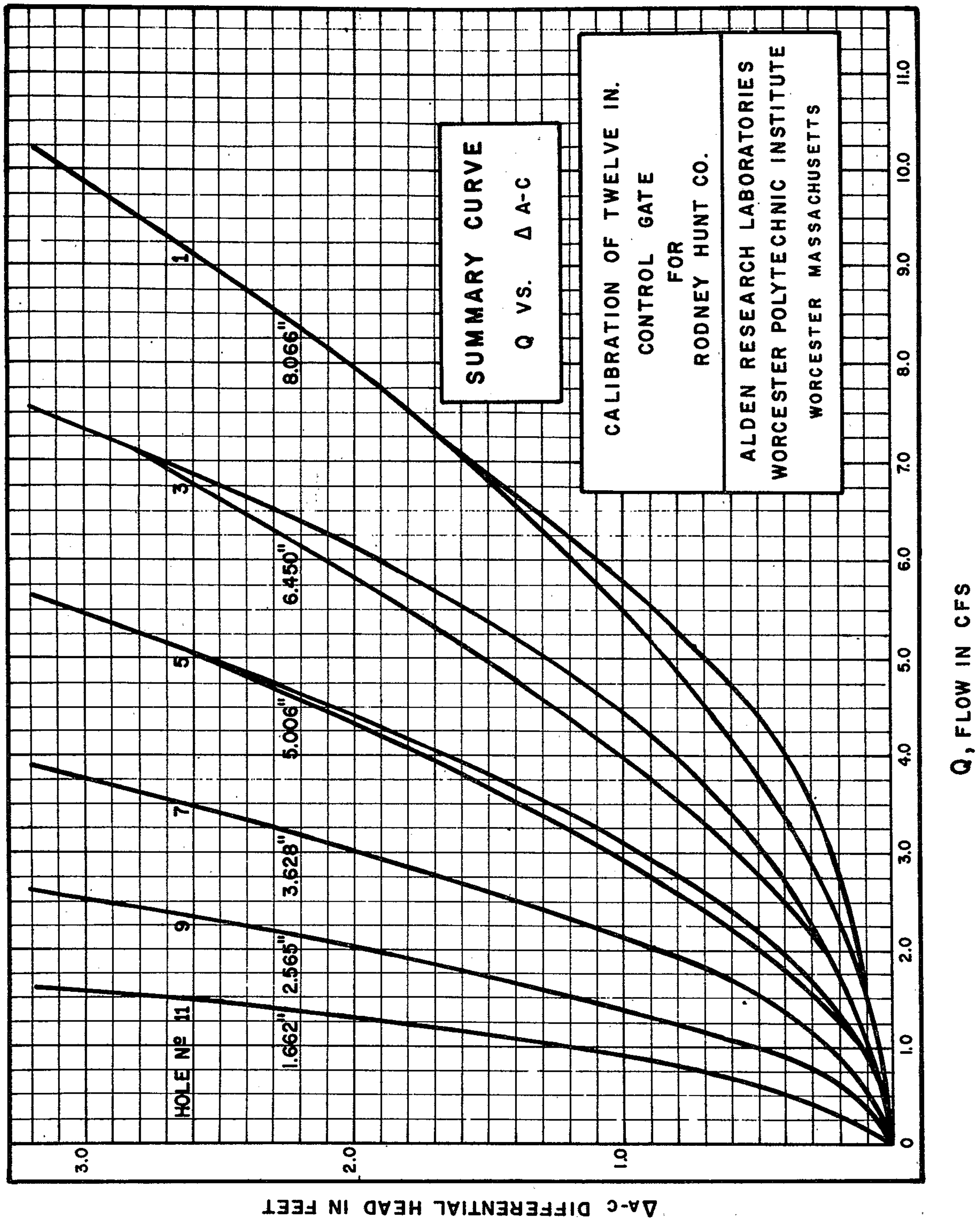
FIGURE 2.8.1.3



TIPPING GATE
USED BY ALLEGHENY COUNTY
SEWAGE AUTHORITY

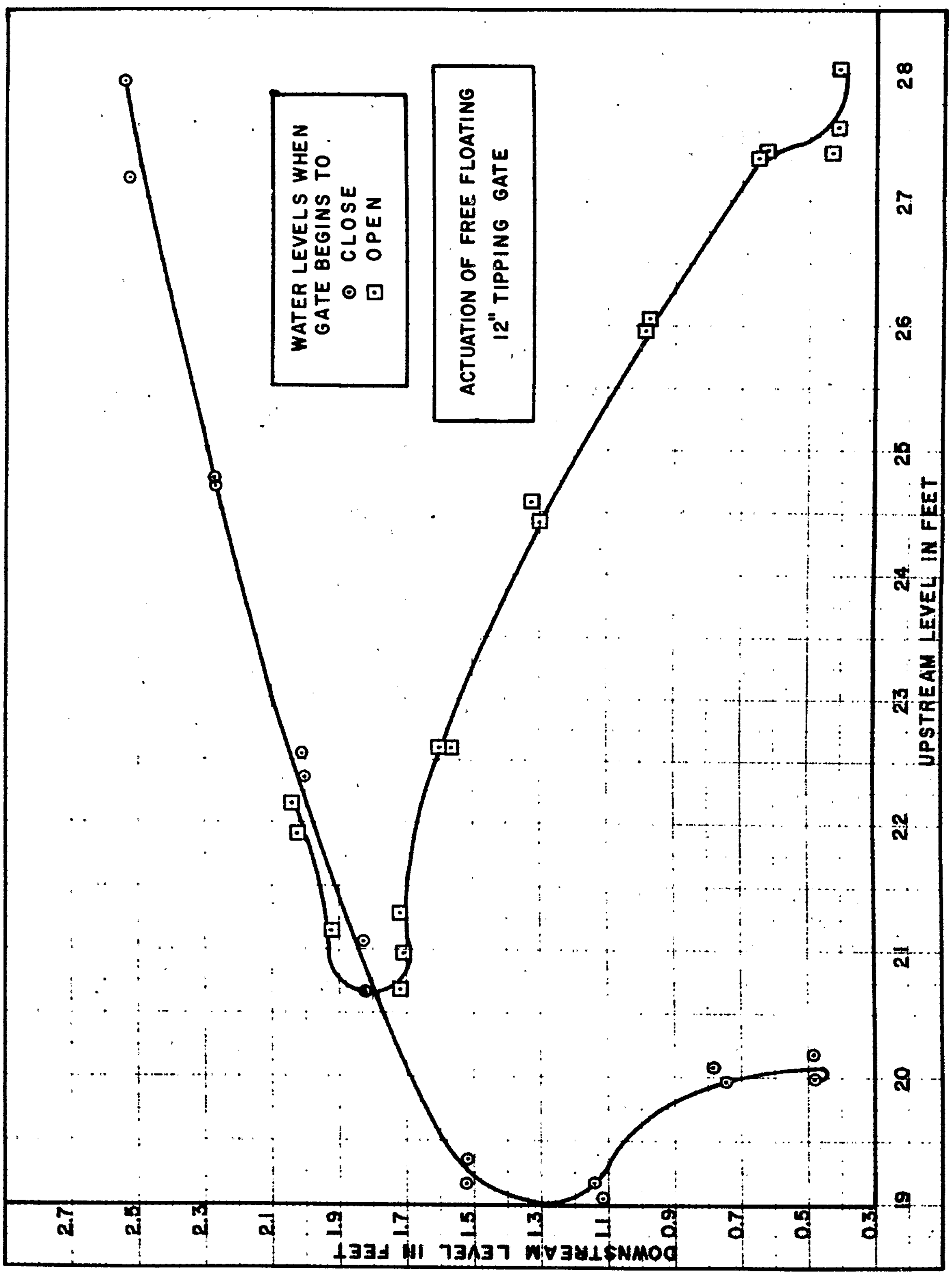
Courtesy Rodney Hunt Co.

FIGURE 2.8.2



Courtesy Rodney Hunt Co. and Alden Research Laboratories

FIGURE 2.8.3



Courtesy Rodney Hunt Co. and Alden Research Laboratories the curves represent the average of the data for all downstream heads.

b. Figure 2.8.3 shows upstream water levels at which the gate starts to close and to open. From these data it is apparent that the upstream depth must be about 2 feet before the gate starts to close and the difference in head may vary between 0.2 feet and 1.5 feet depending upon upstream and downstream levels. The data on the elevations at which the gate started to open were obtained by stopping all inflow to the chamber upstream of the gate. Since this condition is not likely to occur in the field, these data are considered to have little significance in the design of the gate.

For design purposes similar charts should be obtained from the gate manufacturer for the various size gates to be used.

2.8.4 Sample Computation Tipping Gate

Pertinent data

D = diameter
Q = quantity of flow
d = depth of flow
V = velocity
S = slope

Interceptor sewer
D = 36", Invert el. = 10.0
Water surface = 12.4

Combined sewer - Design Q = 100 cfs
D = 54", Invert el. = 16.0, S = 0.0026
Manning n = 0.013, V = 6.4 fps (full)

	Flow	Q cfs	d ft.	V fps	WS
DWF	= Dry-weather flow - av	0.5	0.3	1.8	16.3
	Dry-weather - peak	2.0	0.5	2.5	16.5
	1-year storm	60.0	2.5	6.7	18.5
	10-year storm	100.0	3.6	7.2	19.6

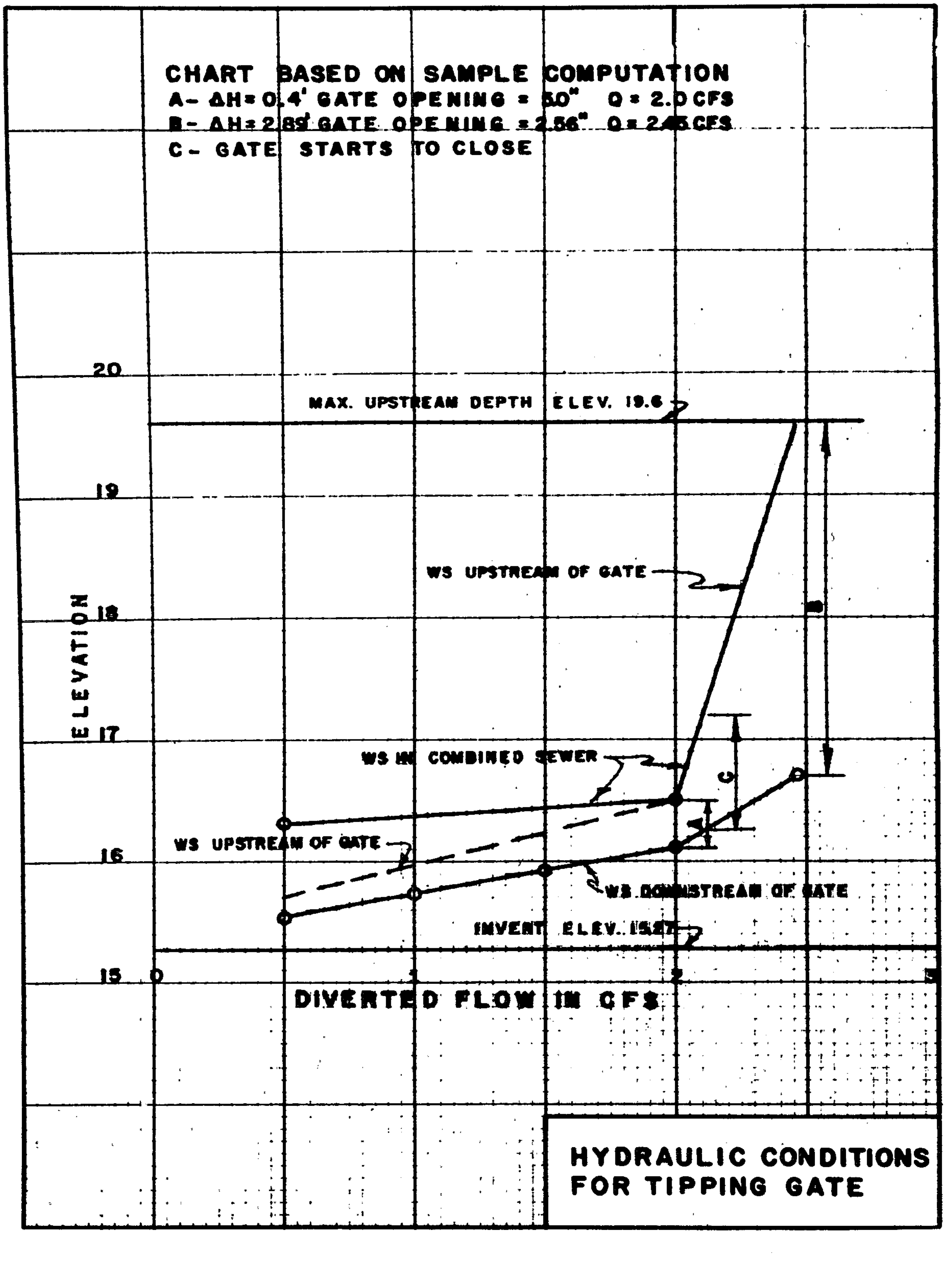
Distance from interceptor to regulator 100'

HGL = hydraulic grade line
EL = energy line

Design Q = 2 cfs

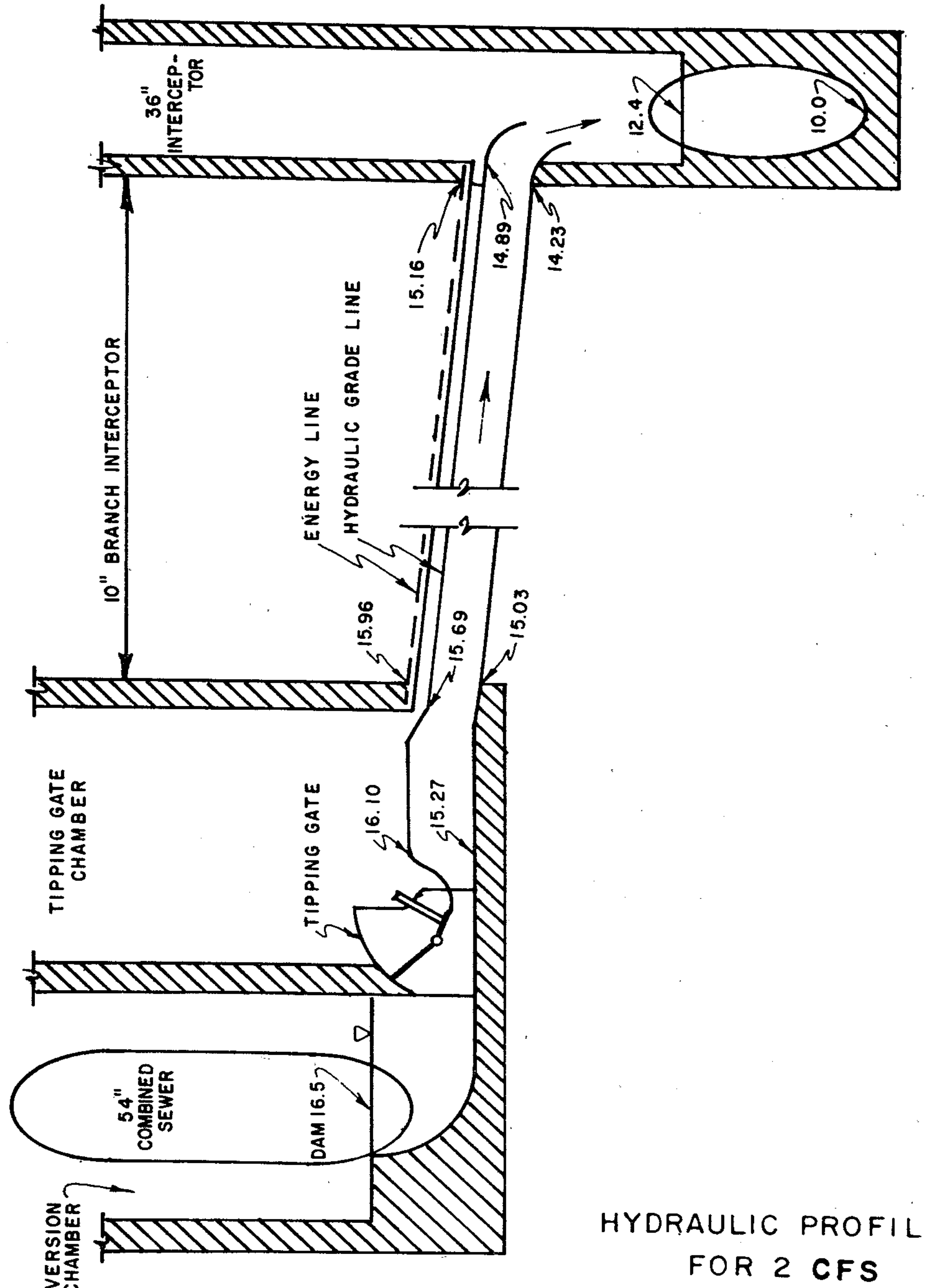
Branch Interceptor

L = 100' n = 0.013 Dia. = 10"
s = 0.008 V = 3.7 fps (full—
V at 0.8 depth = 1.13 x 3.7 = 4.2 fps
$V^2/2g = 0.27$



HYDRAULIC CONDITIONS FOR TIPPING GATE

FIGURE 2.8.4.2



HYDRAULIC PROFILE
FOR 2 CFS
TIPPING GATE REGULATOR

2.8.4 Tipping Gate

Diversion dam elevation

= 16.0 + 0.5 = 16.5

Try 12" gate Figure 2.8.2

For 5.0" opening and 10" tail water differential head = 0.40'

Downstream WS = 16.5 – 0.4 = 16.1'

In Figure 2.1.3.1 for vertical orifice, downstream W.S. is 15.92. Therefore lower branch interceptor by 16.12 – 16.10 = 0.02 from that shown in Figure 2.1.3.1.

	ELEVATION		
Branch Interceptor	Invert	HGL	EL
Downstream invert			
14.25 – 0.02	14.23		
Neglect critical depth 14.23			
+ 0.8 (0.83) = 14.23 + .66		14.89	
$V^2/2g = 0.27$			15.16
Friction loss 100 x 0.008 = 0.80			
Upstream end	15.03	15.69	15.96
Entrance loss 0.5 $V^2/2g = 0.14$	15.03	15.83	16.10
Neglect friction and bend loss in chamber		16.10	16.10
Set chamber invert 10" below			
WS 16.10 - 0.83	15.27		

differential head = 16.50 – 16.10 = 0.40

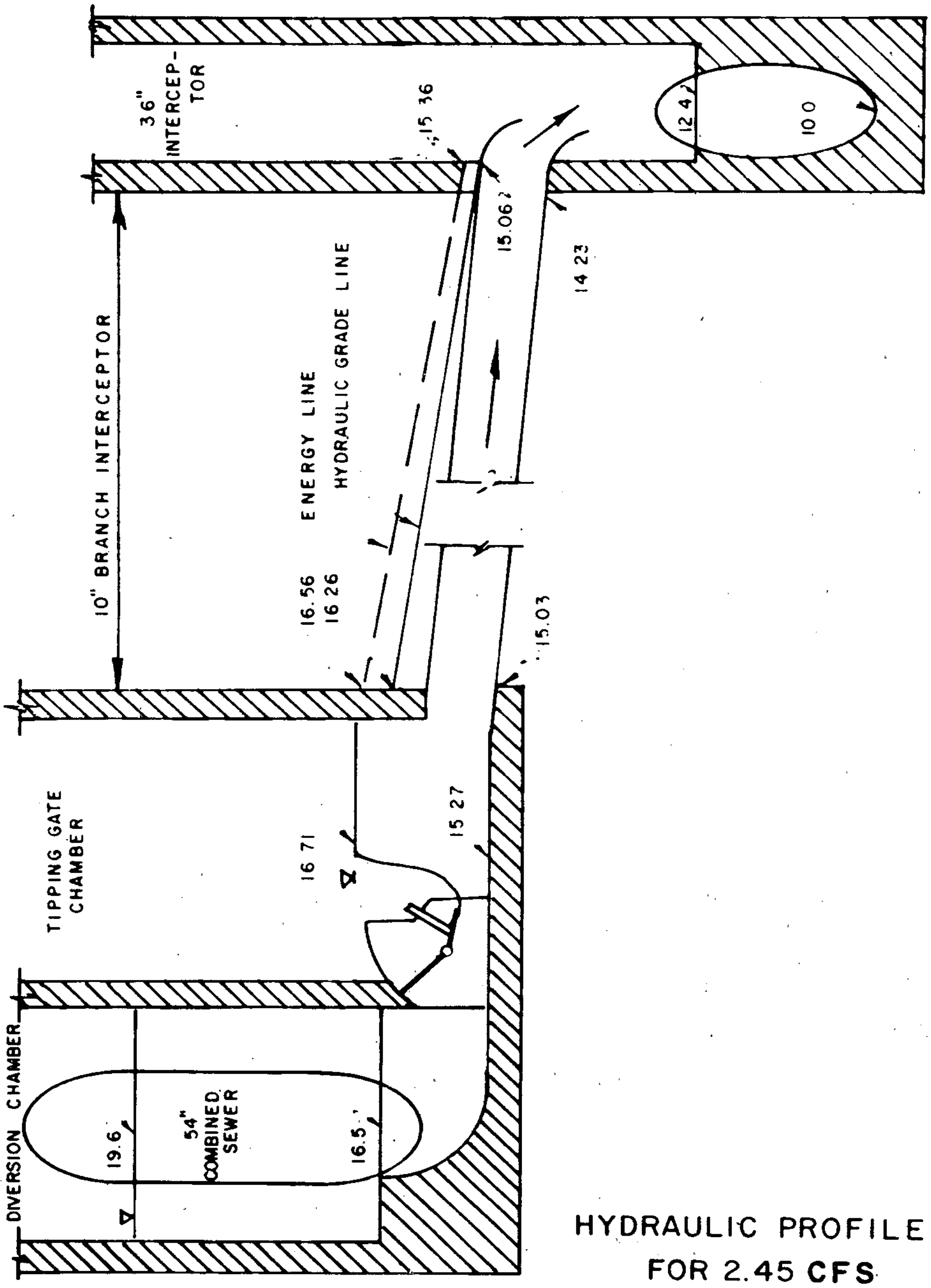
∴ Q = 2.0 cfs for 12" gate and 5.0" opening from Figure 2.8.2

Determine diverted flow in storm period

	10-year storm	1-year storm
HGL in combined sewer	19.60	18.50
Q diverted-cfs assume	2.4	2.2
10" branch S =	0.012	0.010
Lower end top elev. 1/		

1/ This neglects critical depth at lower end and assumes HGL is at top of pipe.

FIGURE 2.8.4.3



HYDRAULIC PROFILE
FOR 2.45 CFS
TIPPING GATE REGULATOR

2.8.4 Tipping Gate

	10-year storm	1-year storm
Exit loss $V^2/2g$	0.30	0.25
Friction loss 100 x s	1.20	1.00
Entrance loss $0.5\ V^2/2g$	0.15	0.12
HG downstream of gate	16.71	16.43
differential head on gate	2.89	2.07
12" gate 2.56" opening Q =	2.5	2.1
Diverted Q = WWF	2.45	2.15
Ratio $\frac{WWF}{DWF} = \frac{WWF}{0.5}$	4.9	4.3

2.9 Cylindrical Gates

2.9.1 Description.

An isometric diagram of the cylindrical gate is shown in Fig. 2.9.1. Combined sewer flow is diverted by a dam through an opening in the side of the sewer into the gate chamber. The diverted flow drops through the horizontal orifice to the interceptor.

The operation of this device, when controlled by the sewage level in the branch interceptor, is shown in Figures 2.9.1B and 2.9.1C. When the level in the interceptor is low, as in Fig. 2.9.1B, the air-vent pipe prevents the formation of a vacuum in the interior of the gate and the gate stays open. When the level in the interceptor rises to a predetermined elevation, the sewage blocks the air-vent pipe, as shown in Fig. 2.9.1C. The entrainment of air produces a vacuum in the interior of the cylindrical gate and atmospheric pressure forces the gate down and closes the orifice.

The control of the gate by the sewage level in the combined sewer is shown in Figures 2.9.1D and 2.9.1E. When the level in the sewer is low the counterweight keeps the gate open, as in Fig. 2.9.1D. When the sewage level rises, the weight of the liquid on the conical part causes the gate to lower and close the orifice

DYNAMIC REGULATORS—AUTOMATIC

2.10 Cylinder-Operated Gates

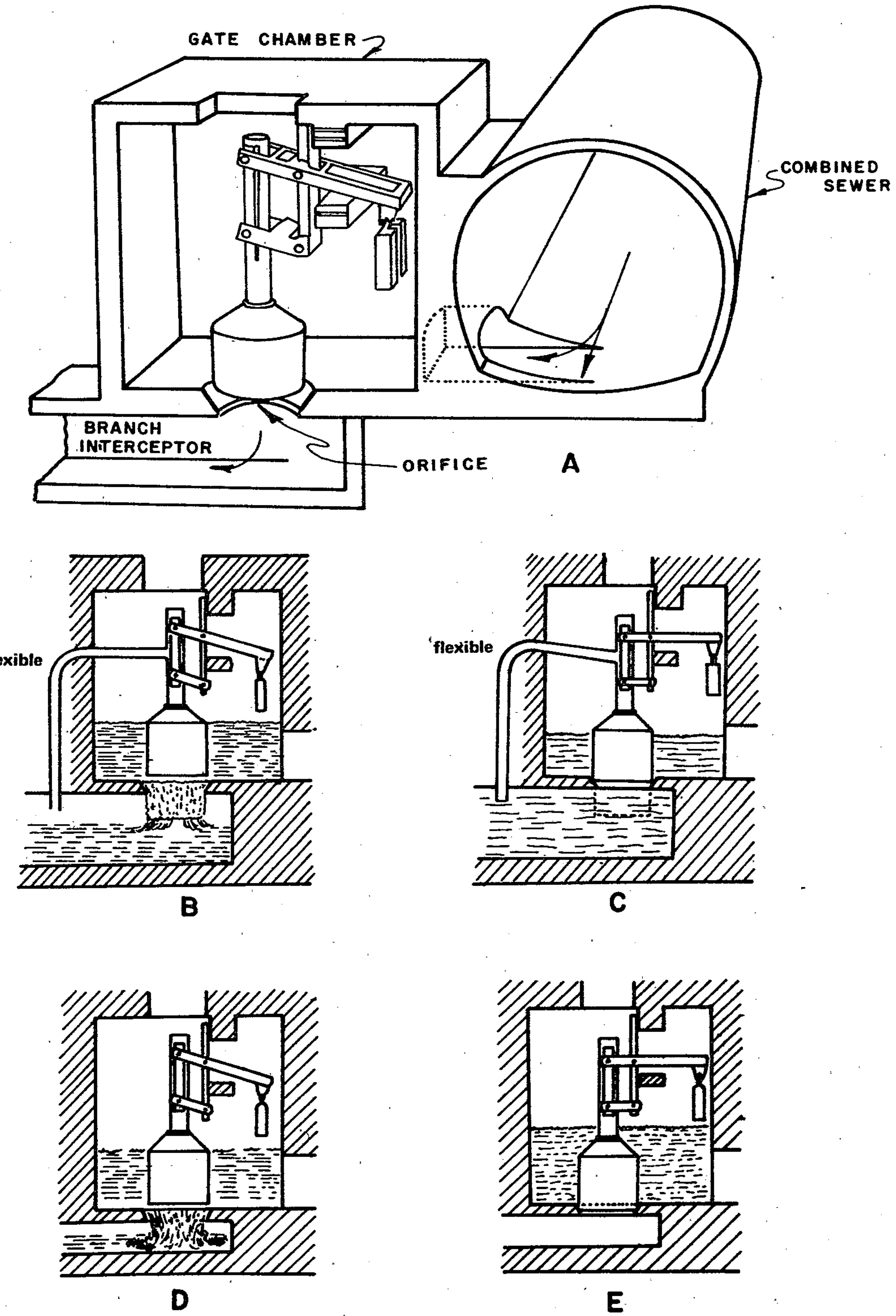
2.10.1 Description

The cylinder-operated gate may consist of two to four chambers: (1) A diversion chamber; (2) a regulator chamber containing sluice gate, cylinder and float or bubbler tube; (3) an equipment chamber when electrical equipment is required; and (4) a tide gate chamber, if required. On some deep and large chambers the diversion and flap gate chambers may be combined, but the other chambers should remain separate. Whenever possible it is desirable to construct the equipment chamber above ground.

The diversion chamber contains an overflow dam to divert the DWF into the adjacent regulator chamber. The top of the diversion dam is usually set to minimize the raising of the flowline upstream of the regulator during storm flows and prevent back flooding. The diversion channel invert is establised so that the peak DWF will be diverted without overtopping the dam. During wet-weather periods the excess flow goes over the dam to the tide gate chamber and thence to receiving waters. The opening between the diversion chamber and tide gate chamber is equipped with one or more tide gates.

The regulator chamber provides for a cylinder-operated sluice gate which governs the amount of flow to the interceptor. The action of the cylinder is related to the sewage level in the sewer by a sensing device which can be sensitive to either upstream or downstream flow conditions. The latter location is used if the main object of the regulator is to avoid overloading the interceptor and treatment plant. Generally, the sensing device is a float or a bubbler-tube. The cylinder is operated either by water, air or oil pressure. Floats may be used in conjunction with cylinders operated by water pressure to avoid the addition of compressed air equipment. During dry-weather periods the sluice gate is fully open. In wet-weather periods the rising sewage level will raise the float or increase the pressure in the bubbler tube so that the gate will partially close. The float or bubbler tube is located in

FIGURE 2.9.1



APPLICATIONS OF CYLINDRICAL GATE

Courtesy Neyrpic Canada Ltd.

a special well connected to the flow channel by a telltale passage.

When the sensing device is located downstream of the gate it is generally necessary to install a control device to maintain subcritical flow in the regulator chamber. One control which is satisfactory is vertical timber stop logs used to decrease the channel width. A vertical slide gate also can be used as a control to act either as a weir or an orifice on the bottom of the channel. The use of vertical wood stop logs has the advantage that the width of channel opening can be adjusted in the field, and there is nothing to impede the discharge of debris which may be carried on the bottom of the channel or may be floating on the surface of the flow.

While the hydraulics of the regulator can be computed, adjustments are usually required in the field to accommodate actual flow conditions. Usually the float or bubbler tube is set to act when the flow level is about one inch above the actual peak dry-weather flow line to insure that all sanitary flow in dry weather is diverted to the interceptor.

Water used as a pressure medium is usually obtained from the public water supply. Prevention of cross-connection hazards will require the use of check valves, vacuum breakers, and air breaks in drain lines, based on effective design criteria. Since the pressure in the water system may vary, the hydraulic cylinder is usually designed to operate on a minimum pressure of 25 psi. For small gates this pressure is adequate but for large gates a very large hydraulic cylinder would be required. None of the hydraulic cylinder manufacturers will guarantee cylinders for water operation. Therefore, the gate manufacturer is required either to build the cylinder or go to a speciality manufacturer for it. The chief advantage of the use of water is that no electrical power is required and, hence, the regualtor functions during power failures, and a spearate chamber is not needed for installation of air compressors or electrical equipment. Tightening of the packings around the piston rod and tail rod, if overdone, may increase the friction forces. Sometimes valves become inoperative due to rust or scale in the water supply. To prevent this a strainer should be installed in the supply line. Maintenance checks are necessary to insure that clogging of the strainer does not cause gate malfunctions.

When air is used as a medium for operating the gate a separate chamber is provided for the air compressors and electrical equipment. Air has been used at pressures of 90 to 100 psi. The disadvantages of this system are: (1) Electrical power is required which is subject to failure; (2) a separate chamber must be provided to house the electrical and compressed air equipment; and (3) difficulty is experienced in maintaining electrical equipment in subsurface chambers. Some jurisdictions using air pressure for cylinder operation have converted to oil pressure.

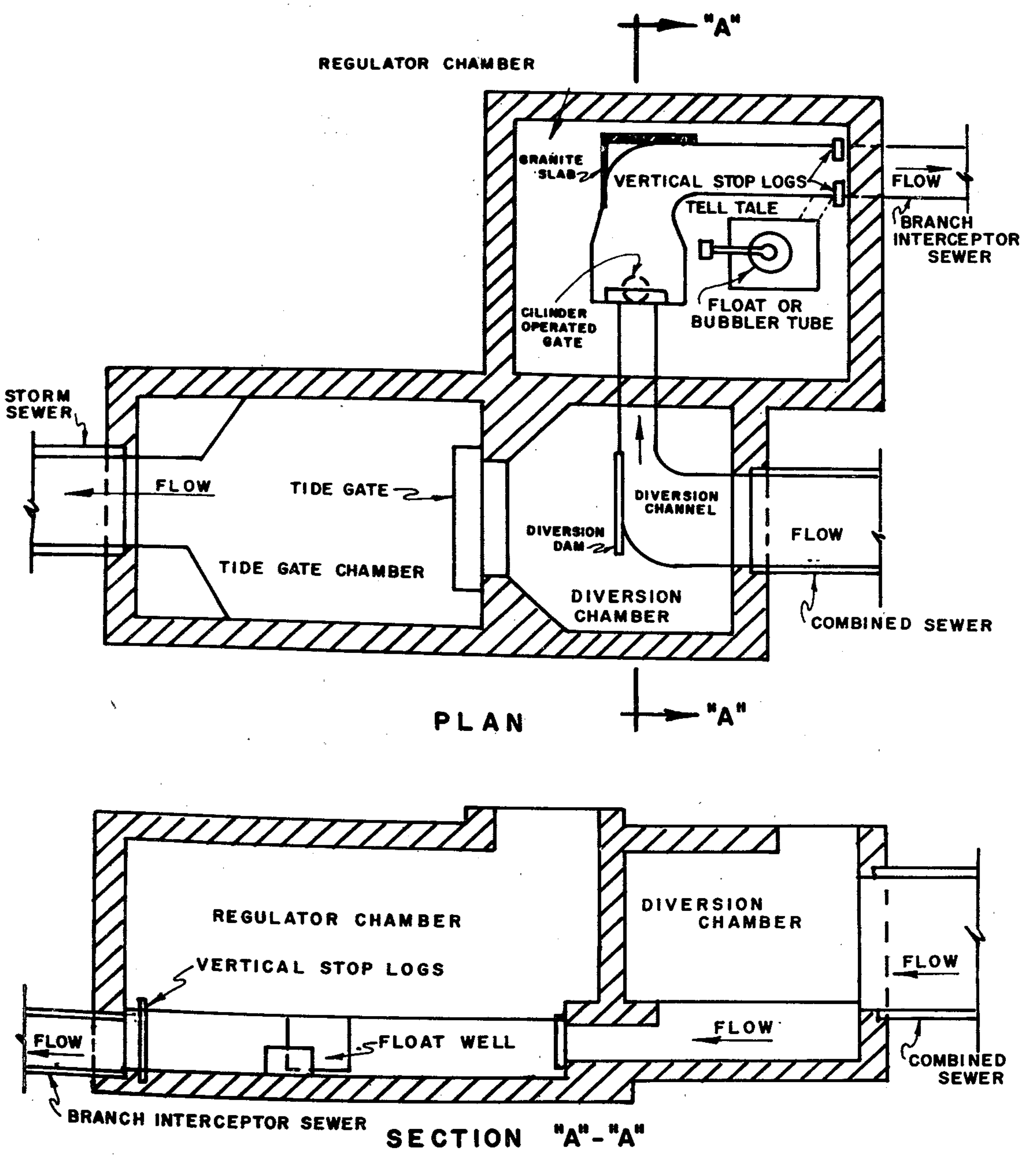
Recent practice in cylinder-operated gates seems to favor oil as a medium rather than air or water. Oil has been used at pressures of 350, 750, and 3000 psi but pressures from 600 to 750 psi are favored. To reduce corrosion, a separate chamber, preferably located above ground, is provided for electrical and pumping equipment. The use of oil results in less corrosion of valves and cylinders than the use of air or water. Smaller cylinders are needed to operate the gate due to the higher pressures used. The disadvantages are similar to those listed for air cylinders.

A typical plan of a cylinder-operated sluice gate regulator using water pressure is shown on Fig. 2.10.1.1. A schematic diagram of a cylinder-operated sluice gate regulator using water pressure is shown in Fig. 2.10.1.2 and one using oil pressure in Fig. 2.10.1.3.

The water-pressure operated cylinder, as shown in Fig. 2.10.1.2, functions basically as follows: (1) Water supply is usually obtained from a public water supply system which should be protected with adequate backflow prevention devices; (2) cylinders should be sized and designed for actual pressure of at least 5 psi less than minimum available pressure specified by the user; and (3) water pressure on the cylinder is controlled by a 4-way valve of the vertical plunger type which is actuated by a float in a well connected to the channel downstream of the sluice gate.

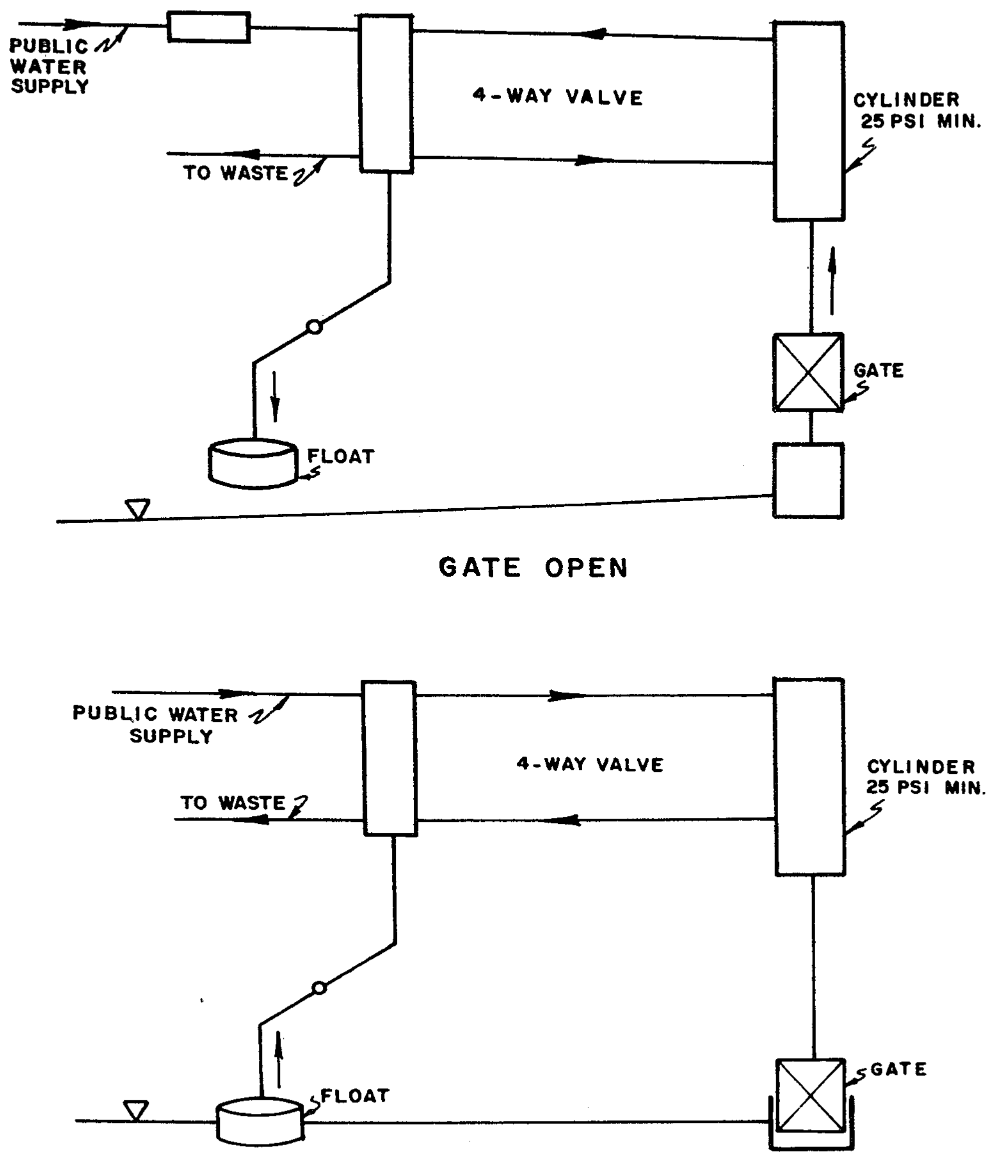
There are four pipe connections to the 4-way valve: (1) From the water supply; (2) to waste; (3) to the top of the hydraulic cylinder; and (4) to the bottom of the hydraulic cylinder. In dry-weather periods the float is down, the valve is up and the water pressure is supplied to the bottom of the hydraulic cylinder to keep the sluice gate wide open. During storm periods when the flow line in the channel reaches a predetermined level, the float rises, causing the 4-way valve to lower. This results in admission of water pressure in the bottom of the cylinder, thus causing the sluice gate to close. As the discharge through the orifice decreases the flow line in the regulator chamber falls, causing a reversal of this procedure. Thus the gate will "hunt" for its proper position. A needle valve is used to control the

FIGURE 2.10.1.1



CYLINDER-OPERATED GATE

FIGURE 2.10.1.2

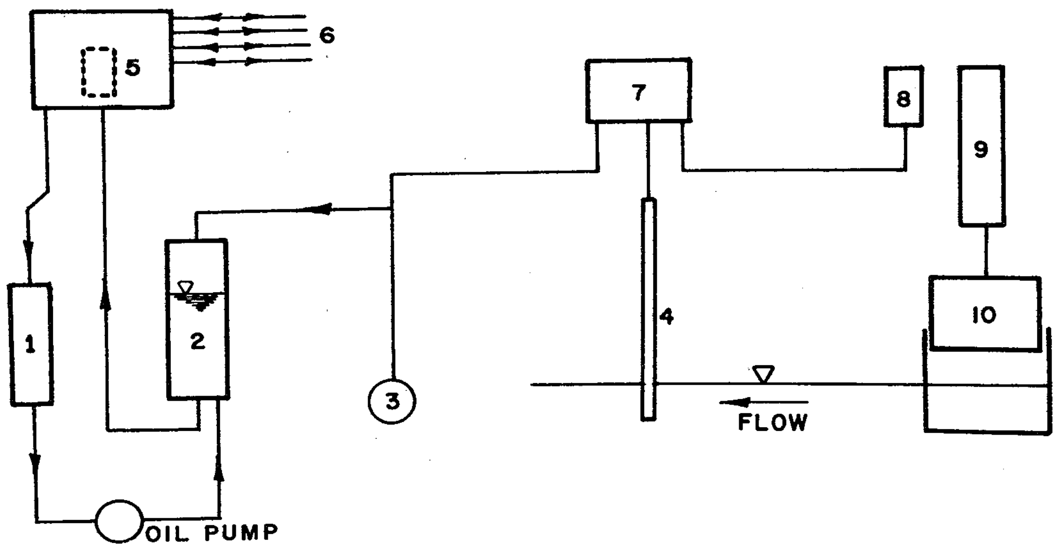


SCHEMATIC DIAGRAM

CYLINDER-OPERATED GATE
(WATER ACTUATED)

FIGURE 2.10.1.3

SCHEMATIC DIAGRAM



1 OIL RESERVOIR
2 ACCUMULATOR-OIL UNDER CONSTANT AIR PRESSURE
3 AIR COMPRESSOR
4 BUBLER TUBE
5 FOUR-WAY VALVE
6 OIL LINES TO POSITIONER & CYLINDER
7 ELECTRIC CONTROL
8 POSITIONER
9 HYDRAULIC CYLINDER
10 GATE

CYLINDER-OPERATED GATE (OIL ACTUATED)

rate at which the water pressure is transmitted to the cylinder to prevent rapid up-and-down movements of the gate. The usual installation will involve a vertical movement of the float of three inches and of the 4-way valve of 3/4 inch.

A cylinder-operated gate using oil pressure, as shown in Fig. 2.10.1.3, functions as follows: An air compressor supplies air continuously to a bubbler sensing device located downstream of the gate. The pneumatic pressure on the bubbler-tube is transmitted to a "positioner" on the cylinder. Oil is fed by an electric-motor-driven pump to the accumulator where the oil is kept under pressure by air supplied from the air compressor. When the position of the gate is out of balance with the level indicated by the bubbler-tube a signal from the positioner actuates a pneumatically operated four-way valve in the hydraulic control, which directs the oil to the top or bottom of the cylinder as required to open or close the gate.

Regulators of this type have been installed without a "positioner." Its use results in less "hunting" by the gate and is recommended.

A manual or diesel operated pump should be provided for gate operation in case of power failure.

The major advantage of this regulator is that the gate position can be transmitted to a remote control point either by a pneumatic signal, for distances up to 800 feet, or by an electrical signal to any distance. Such a system can also permit positioning of the gate from the remote control point.

2.10.2 Design Guidelines.

Obtain all pertinent data for the combined sewer at the proposed location of the regulator. This will include diameter, invert elevation, slope, average and peak dry-weather flow and peak storm flow. Obtain similar data for the interceptor at the proposed location of its junction with the branch interceptor. Also obtain data on high water levels in the receiving waters. If the interceptor is being designed in conjunction with the regulator, assume an elevation for the interceptor and adjust as found necessary by subsequent computations for the regulator.

After making a preliminary layout the energy and hydraulic grade lines should be computed for the branch interceptor, regulator and combined sewer. Insofar as possible, the designer should select elevations that will result in uniform flow. If non-uniform flow occurs it may be necessary to compute backwater or drawdown curves.

Initial design should be made on the basis of diverting the maximum dry-weather or peak sanitary flow, when peak sanitary flow occurs in both the interceptor and branch interceptor. The elevation of the water surface in the combined sewer as thus computed should be below the elevation of the diversion dam, which usually is established as six inches above the invert of the combined sewer. The sizes and elevations of the proposed structures should be adjusted as necessary so that this computed water surface is just below the dam elevation.

The hydraulic and energy grade lines also should be determined for the maximum storm flow and peak sanitary flow in the combined sewer and peak sanitary flow in the interceptor. If the receiving waters into which the combined sewer discharges are subject to variation in elevation the effect of this variation should be included in the computations.

Good design requires that the water surface in the float well will be dependent on the flow through the sluice gate and not be affected by flow conditions downstream in the branch interceptor or interceptor, and the diversion chamber and tide gate, if used, will not raise the water surface in the combined sewer during storm flows to an extent that damage from flooding will occur upstream of the regulator. If there are downstream constraints on the interceptor and if excessive flows are diverted to the interceptor, it may be possible for the hydraulic gradient to rise sufficiently in the regulator chamber to cause the float to close the sluice gate completely, thus eliminating all flow interception.

2.10.3 Design Formulas

The sample computations given are based on the hydraulic formulas outlined below. The designer may wish to analyze the problem in more detail than is given, by reference to texts on hydraulics.

As a result of turbulent flow in the regulator during periods of peak flow and because of clogging, it is doubtful if any regulator will act exactly as designed. The designer must use judgment as to how precise his computations shall be made. The following are design considerations:

a. Friction losses in pipes and channels. Friction losses in the computations herein are based on the Manning formula using a constant *n* friction coefficient with variation in depth.
b. Contraction or inlet loss.

   $H = K(V_2^2/2g - V_1^2/2g)$

   where H = head loss in feet

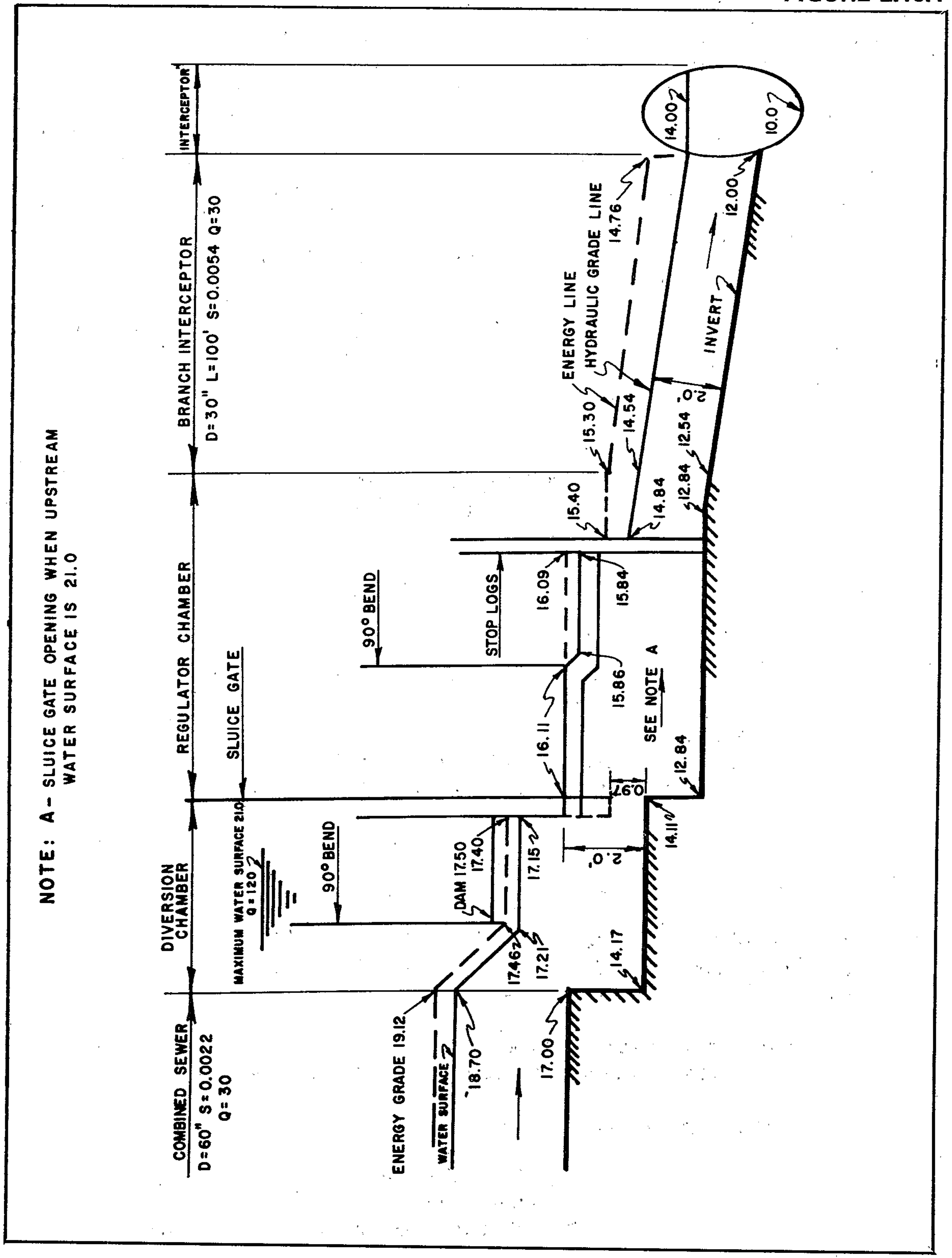
   $K = 0.5$ for sharp-cornered entrance

   $K = 0.1$ for gradual transition

   V_1 = upstream velocity in fps

   V_2 = downstream velocity in fps
c. Enlargement or outlet loss

FIGURE 2.10.4



HYDRAULIC PROFILE FOR REGULATOR
WITH CYLINDER-OPERATED GATE, Q OF 30 CFS

$H = K(V_1^2/2g - V_2^2/2g)$

where H = head loss in feet

K = 1.0 for sudden enlargement

K = 0.2 for gradual transition

V_1 = upstream velocity in fps

V_2 = downstream velocity in fps

d. Loss in 90-degree bend

$H = 0.25 - V^2/2g$

where H = head loss in feet

V = velocity in fps

e. Flow through orifice

$Q = CA\,(2gH)^{1/2}$

where H = head loss in feet

Q = discharge in cfs

A = area of orifice in square feet

C = 0.7 for sluice gate or vertical orifice

C = 0.6 for horizontal plate orifice

f. Discharge through opening between vertical stop planks

$Q = 3.09\ bH^{3/2}$

where Q = discharge in cfs

b = opening width in feet

H = total head upstream of stop planks

The above is applicable only if the downstream water depth is 2/3 or less of the upstream head.

g. Critical depth.

Critical depth may be determined for rectangular and circular conduits from Fig. 26 and for circular conduits from Table XVI both of which are in ASCE Manual No. 27.

h. Backwater and drawdown curves.

these curves may be computed by either of the methods illustrated in Tables XV and XVI in ASCE Manual No. 27.

2.10.4 Sample Computation
Cylinder-Operated Gate

Pertinent data

Interceptor sewer

D = 60", Invert = el. 10.0, W.S. = el. 14.0

Combined sewer

D = 60", Invert = El. 17.0, S = 0.0022

Manning n = 0.013 V (full) = 6.2 fps

Q	cfs	d ft.	W.S.	V fps
DWF = Av. dry-weather	15	1.20	18.20	4.3
Peak dry-weather	30	1.70	18.70	5.2
Peak storm [1]	120	4.00	21.00	7.0

[1] **Includes peak dry-weather flow**

Distance from interceptor to regulator on combined sewer is 100 ft.

For hydraulic profile see Figure 2.10.4

HGL = hydraulic grade line

EL = energy line

D = diameter, V = velocity, Q = discharge

d = depth of flow

S = slope

L = length

2.10.4 Cylinder-Operated Gate

	ELEVATION		
	Invert	HGL	EL
Interceptor — Peak dry-weather flow	10.00	14.00	
Branch Interceptor n = 0.013			
D = 30 in., Q = 30 cfs, s = 0.0054			
d = 0.8 x 2.5 = 2.0 ft.			
V = 1.13 x 6.2 fps			
$V^2/2g$ = 0.76 ft.			
Lower end	12.00	14.00	
Pipe outlet loss 1 (0.76 – 0) = 0.76			14.76
Upper end			
Friction loss 100 x 0.0054 = 0.54	12.54	14.54	15.30
Regulator Chamber			
Say b = 2.5 ft., d = 2.0 ft.	12.54	14.54	15.30
$V = \frac{30}{2.5 \times 2.0} = 6.0$ fps			
$V^2/2g$ = 0.56			
Pipe inlet loss = 0.5 (0.76 – 0.56) = 0.10			15.40
HGL = 15.40 – 0.56		14.84	
Invert = 14.84 – 2.0	12.84		

Determine if flow is stable from Figure 26 ASCE Manual 37
Q = 30 cfs b = 2.5 feet
dc/b = 0.65
dc = 0.65 x 2.5 = 1.62
$\frac{2.0}{1.62} = 1.24 > 1.15$ Flow is stable

However to reduce chance of backwater effect from interceptor raise flow line 1.0 foot by inserting vertical stop logs in channel

	Invert	HGL	EL
d = 3.0 feet	12.84	15.84	
$V = \frac{3.0}{2.5 \times 3.0} = 4.0$ fps			
$V^2/2g$ = 0.25 ft.			16.09

2.10.4 Cylinder-Operated Gate

	ELEVATION		
	Invert	HGL	EL
Determine stop log opening			
$Q = 3.09\ b\ H^{3/2}$			
$30 = 3.09 \times b \times (3.0 + 0.25)^{3/2}$			
b= 1.65 feet (opening)			
Condition upstream stop logs	12.84	15.84	16.09
Channel friction loss			
L = 10 ft. n = 0.015			
V = 4.0 fps r = 0.88			
From nomograph S = 0.0018			
Friction loss = 10 x S = 0.02	12.84	15.86	16.11
Assume complete loss of velocity of discharge from sluice gate from impact on granite slab:			
Upper end of channel	12.84	16.11	16.11

Regulator Chamber

Head loss in sluice gate
Try 30 in. x 24 in.

$Q = CA\sqrt{2gH}$
$30 = (0.7)\ (2.5)\ (2.0)\ (8.03)\sqrt{H}$
$H = 1.04$ ft.

Diversion Chamber

	Invert	HGL	EL
Invert = 16.11 – 2.0	14.11		
16.11 + 1.04		17.15	
Approach Channel			
b = 2.5 ft.			
d = 17.15 – 14.11 = 3.04 ft.			
$V = \frac{30}{2.5 \times 3.0} = 4.0$ fps			
$V^2/2g = 0.25$			17.40
Neglect friction loss in channel			
Bend loss 0.25 (0.25) = 0.06	14.17	17.21	17.46
Required upstream of bend		17.21	17.46
Available in combined sewer	17.00	18.70	19.12

Dam elevation 17.00 + 0.5 = 17.50
> 17.21 Design OK

2.10.4 Cylinder-Operated Gate

Determine sluice gate opening during peak storm

Q = 120 cfs in combined sewer
HGL = 21.00 in combined sewer
$H = 21.00 - 16.11 = 4.89$ ft.
$Q = CA\sqrt{2gH}$
Q = 30 cfs through sluice gate
$30 = (0.7)\,(2.5)\,(d)\,(8.03)\sqrt{4.89}$
$d = 0.97$ ft. – height of gate opening

Thus gate travel is $2.0 - 0.97 = 1.03$ ft.

The following adjustments could be made in the foregoing design. Since the computed HGL in the diversion chamber is 0.29 feet (17.50 - 17.21) below the top of the dam the invert of the structure between the diversion chamber and the stop logs could be raised 0.29 feet. Since the stop logs control flow upstream thereof, the HGL downstream of the stop logs could be lowered by decreasing the length and/or increasing the diameter of the branch interceptor. Since the drop in EL at the stop logs is 0.69 feet (16.09 - 15.40) any rise in water surface of the interceptor exceeding 0.69 feet will result in backwater effect on the regulator.

2.11 Motor-Operated Gates

2.11.1 Description.

A regulator using a motor-operated gate consists of two to four chambers; (1) A diversion chamber; (2) regulator chamber containing the sluice gate and a sensor; (3) a motor chamber; and (4) a tide gate chamber, when required. On some deep and large chambers the diversion and tide gate chambers may be combined but the other chambers should remain separate. Whenever possible it is desirable to construct the motor chamber above ground. The diversion chamber is as described for cylinder-operated gates in 2.10 of this Section.

The regulator chamber contains a motor-operated sluice gate which governs the amount of flow diverted to the interceptor. The action of the gate relates to the sewage level by a sensing device which can be used to respond to flows either upstream or downstream of the sluice gate. The latter location is used if the main object of the regulator is to avoid overloading the interceptor and treatment plant. The sensor could be a sealed electrode type, pressure cell type or possibly pressure-sensitive electric type. A float or compressed-air bubbler tube could also be used. The design of the regulator chamber is similar to that of the cylinder-operated gate (see 2.10.)

2.11.2 Design Guidelines.

Guidelines for hydraulic design are the same as those for the cylinder-operated gate. Design formulas for sample computations for this regulator are as indicated for the cylinder-operated gates.

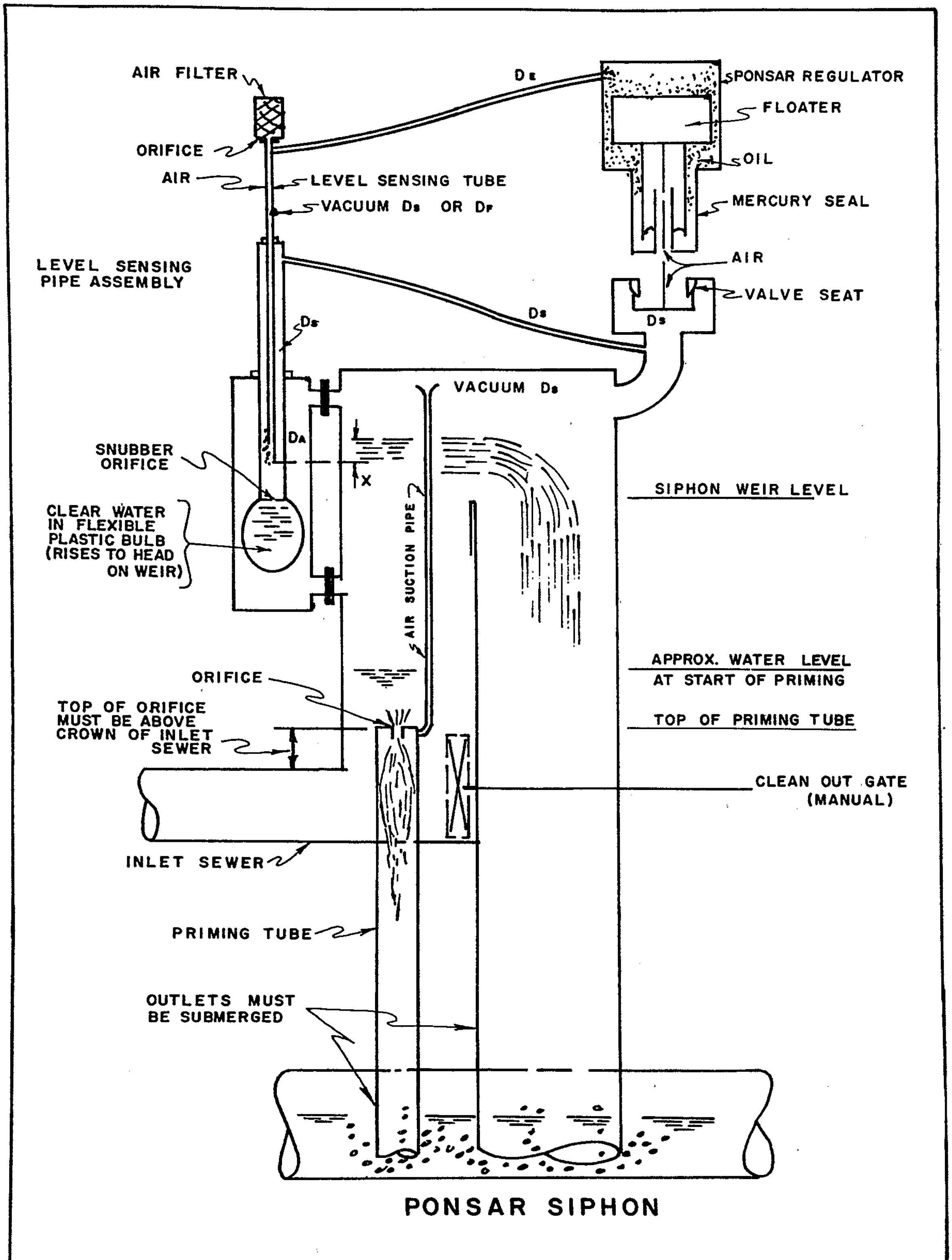
2.12 External Self-Priming Siphon

2.12.1. Description

This device is currently in use in Europe and is a new flow control method which could lend itself to "total systems control."

The external self-priming siphon uses an exterior device to prime the siphon and the design of the siphon conduit is not as critical. A siphon of this type, as shown in Fig. 2.12.1, was proposed by A. Moan and Y. Ponsar of France, for use as a flood-stage or tide-water check valve (tide gate) to protect the interceptor. When the upstream water surface is higher than the downstream water surface the sewage will flow through an orifice plate in the top of the priming tube and through the priming tube to the downstream side. This flow will carry air with it, causing a partial vacuum on the underside of the orifice. By connecting the summit of the siphon to the priming tube, a vacuum is created on the siphon summit, causing the siphon to discharge. To prevent siphonage in the opposite direction a float and air valve device is provided so that when the downstream level raises the float an air valve is open, allowing air at atmospheric pressure to enter the summit. The use of this type siphon with a "Ponsar Regulator" as a

FIGURE 2.12.1



PONSAR SIPHON

Courtesy Degremont S.A. and New York City, N.Y.

combined sewer regulator is described in subsequent paragraphs.

The Ponsar siphon also has been used in waste water treatment plants but, so far as is known, has not been used in either the United States or Canada in connection with combined sewer regulators. The treatment plant in Geneva, Switzerland, utilizes this type of siphon to automatically distribute flow equally to the remaining number of treatment units in operation if one or more is taken out of service, and to divert flows in excess of fixed maximums to other units of the treatment plant.

Its use also has been suggested for diverting a predetermined amount of combined sewer flow to a plant, as shown in Fig. 2.12.1. The minimum flow that can be diverted by this type of siphon is governed by the fact that, to prevent clogging, the priming tube should have a minimum diameter of eight inches and the orifice a minimum diameter of five inches. Using these sizes the application of the Ponsar siphon to diversion flows less than 8 cfs would not seem practical.

The priming tube is used to develop negative pressures in the siphon summit. If the inlet and outlet branches of a siphon are submerged, a priming tube can be utilized effectively to establish siphonage, provided air evacuation velocities are developed in the priming tube and its cross-sectional area is large enough to pass suspended materials transported by the incoming waste water flow.

Upon establishment of appropriate vacuum conditions within the siphon, the water column will rise to the summit of the siphon and flow will be established over the siphon crest or weir into the discharge branch. By controlling the intensity of vacuum conditions within the siphon, the discharge can be effectively regulated to desired discharge limits.

Basic Components:

The siphon installation comprises three basic elements: (1) An upflow branch with a priming tube within the upflow branch; (2) a vacuum chamber with a vacuum regulating device; and (3) a discharge branch. A self-contained siphon installation incorporating these basic elements is shown in Fig. 2.12.1.

The priming tube consists of a priming pipe with an orifice plate attached at the top. The orifice is set at a level high enough above the crown of the inlet sewer to assure continuously complete submergence at entry to the siphon upflow branch. The lower end of the priming pipe extends to the flow level of the receiving conduit where it must be submerged below the lowest downstream discharge level.

The orifice serves to establish a jet discharge which draws entrapped air from the space directly under the orifice, surrounding the jet contraction, and from the siphon summit through an air suction pipe installed for that purpose. The air and waste water mixture is drawn down the priming pipe where the air escapes to the downstream water surface. The rate of air evacuation tends to increase with the length of the priming pipe. The pipe connection from the siphon summit to the priming pipe, immediately below the orifice serves to remove air from the siphon initially and after full flow is established.

Consideration of Flow Conditions in Priming Pipe

Experimental work reported by Kalinske in "Hydraulics of Vertical Drains and Overflow Pipes, Bulletin No. 26, Studies in Engineering, University of Iowa," provides data on downdraft flow in pipes roughly comparable to flow conditions in a priming tube. In accordance with Kalinske's findings, the maximum ratio of air removal to water flow is approximately 0.65 when discharging about 1.1 cfs of water through a 6-inch diameter pipe 6.67 ft. long, and is approximately 1.0 when the water discharge is at a rate of 1.0 cfs in a priming tube 11.0 ft. long. For short pipes of 2- to 3- ft. length, jet flow occurs without touching the walls of the pipe. The negative pressure immediately below the pipe entrance increases with the discharge and length of pipe. Although longer pipes will produce a higher vacuum the discharge does not increase correspondingly. In general, the ratio of head "H" on the priming pipe to pipe diameter "D" is proportional to the Froude number, particularly when the priming pipe is not flowing full. However, the Reynolds number becomes important when the priming pipe begins to flow full. The critical values of H/D at which the latter occurs may be estimated at approximately 0.9 to 1.1 for pipe lengths ranging from 20 to 50 pipe diameters. The orifice may be expected to modify the critical head.

Air bubbles will tend to rise against the downward water flow at a velocity of approximately 0.75 fps. Therefore, downward current velocities greater than 0.75 fps must be maintained to evacuate air. For effective priming, a necessary condition is that the downflow rate of liquid in the priming pipe must exceed 1 fps and should preferably be about 2 fps. As air is removed from the siphon an equivalent volume of water will be drawn up into the siphon and into the priming tube. Similarly, after flow is established over the crest of the siphon, more air will be drawn out through the downdraft tube and the water level will likewise rise in the downdraft tube.

2.12.2 Design of Priming Pipe (Based on data furnished by Degremont Corp., Paris)

Operation of the priming pipe is similar to a hydraulic compressor in which hydraulic power is converted to compressed air power.

Let: Absolute suction pressure at inlet = P_a (ft. of water)

Discharge pressure due to submergence at outlet = P (ft. of water)

Ratio of pneumatic to hydraulic power = u

Air compression rate = $(P_a + P)/P_a$

Output ratio of volume of air to volume of water = Q_a/Q_w

Available operating hydraulic head = h

Then: $u = (34Q_a)/(h\,Q_w) \times \log_e (P_a + P)/P_a$

For priming tubes the ratio of (air power)/(water power) may be estimated at 40 percent.

The hydraulic air compressor may be considered the reverse of an air lift and the design could proceed on that basis. The relationship of available operating hydraulic head, depth of submergence at outlet, specific gravity of air plus water mixture and hydraulic losses may be expressed as follows: (based on "The Control of Water" by Philip A. M. Parker.)

$(h + d)/(1 + K) \quad -d = h_f + h_v$

Where:

Available operating hydraulic head = h (ft)

Depth of submergence at outlet = d (ft)

Losses due to friction = h_f (ft)

Misc. other head losses = h_v (ft)

$K = (34Q_a)(dQ_w) \times \log_e \times (34+d)/34$

In the above value of "K" the air supply is measured in cu. ft. at atmospheric pressure. In this connection it may be pointed out that 1 (1 + K) is the specific gravity of the air + water mixture.

The preceding considerations provide a basis for estimating the air discharge rate, "Q_a" and, correspondingly, the time required to produce the desired vacuum conditions and the duration of back-up conditions in the incoming sewer.

The use of an orifice of smaller diameter than the priming pipe is necessary to provide an initial jet at downdraft with partial vacuum around the jet below the orifice.

Control of Flow Through Siphon

The rate of flow through a siphon can be controlled to any desired limit below the rate of flow at full siphon action, by introducing air into the siphon chamber at a regulated rate so as to maintain the desired partial vacuum for design conditions. This objective can be accomplished by use of a vacuum pump with suitable provisions for its operation in relation to desired flow control. Either electrical power, or hydraulic power by water under pressure must be provided to operate the air pump.

An automatic, self-regulating device with appropriate accesories described as the Ponsar Regulator is available to perform the function of vacuum regulation inside the siphon without the use of external power.

The Ponsar Regulator is essentially a vacuum-operated, float-balanced air valve which responds to the vacuum intensity and liquid level in the siphon chamber. The air valve is normally closed. It opens only to admit air in response to siphon conditions, as required to maintain the design flow and to prevent vacuum intensity in excess of desired design conditions.

The Ponsar Regulator consists of two basic elements: (1) A regulator valve which has a floater at the top of the valve immersed in oil over a mercury seal and a connecting shaft to an air inlet valve at the bottom; and (2) a level sensing pipe assembly with its lower end inside a separate chamber where a water-filled plastic bag is attached at the bottom. A level-sensing tube is installed inside the level-sensing pipe. The top of the pipe has an airtight joint where the level-sensing tube passes to the outside. As the water level within the siphon rises due to increased vacuum conditions, the liquid in the plastic bag is compressed, raising the water level in the pipe until the mouth of the level sensing tube is submerged. Filtered air is drawn in through an air filter and a small orifice at the top of the level sensing tube. The quantity of air drawn in is dependent on the water level in the level sensing pipe. This air lowers the vacuum pressure in the level sensing tube and the reduced pressure is transmitted to the top of the floater. The lower part of the air valve has a tube connection to the top of the level sensing pipe, thereby maintaining vacuum conditions therein equal to the vacuum in the siphon. As the vacuum pressure increases, the water level in the siphon rises above the design level, the air valve is subjected to a higher vacuum intensity than the top of the float, the air valve descends, its air ports are opened, and air is admitted to the siphon until the vacuum intensity is reduced and the design flow level is restored.

For purposes of discussion, the vacuum suction pressure at the top of the float is designated D_f and the vacuum suction pressure in the siphon as D_s. If the air inlet orifice at the top of the level sensing tube becomes clogged or its diameter is too small to admit sufficient air, then D_f will approach D_s and the air valve will open, because the buoyancy of the floater is greater than the weight of the moving valve element by a vacuum pressure differential, X, equivalent to a predetermined value. Whenever D_s exceeds D_f plus X, the air valve opens. The air valve remains closed as long as D_s is less than D_f plus X. The level sensing tube is movable and can be reset to desired design elevations. A separate indicator gauge is provided to show the water level in the siphon, which permits setting the level sensing tube at the proper elevation.

Problems of Siphon Operation

Since siphon operation is governed by partial vacuum conditions, some air will always remain at its summit. It also can be expected that additional air will be introduced by release of entrained air in the incoming waste water flow and by direct inward leakage at siphon joints and connections. However, air admitted by the regulating device will be the major air component.

Changes in upward velocities of the siphon flow may cause some surging at the water surface. It is impractical to completely suppress surging effects inside the siphon, except as can be accomplished by a snubber provided in the level-sensing tube above the clear water container.

In general, the lower range of operation, to about 1/3 of the full siphon capacity, will be unstable in action because of inadequate air evacuation at low flows. Greater stability will prevail when the siphon operates at 2/3 or greater capacity.

Flow conditions in the incoming sewer will be affected by operation of the siphon. In order to prevent entry of air from the incoming sewer to the upflow branch of the siphon, entry to the siphon must always be submerged. The top of the priming tube therefore must be set higher than the inside top of the sewer connection entering the siphon. Additional backup in the sewer will be caused by the head required above the orifice to establish effective air evacuation and necessary vacuum conditions. The backup of flow will result in a temporary reduction of flow velocities and temporary storage in the incoming sewer. After appropriate vacuum pressures are established in the siphon, the flow in the sewer will be restored to normal levels related to submerged inlet conditions.

SECTION 3

DESIGN GUIDELINES FOR TIDE GATES, THEIR CHAMBERS AND CONTROL FACILITIES

CONTENTS

3.1. General Information

3.1.1 Principle

Tide gates, also called backwater or flap gates, are installed at or near sewer outfalls to prevent the back-flooding of the sewer system by high tides or high stages in the receiving waters. Tide gates are hinged at the top and are designed to permit discharge through the gate with small differential head on the upstream side of the gate and to close tightly with small differential head on the downstream side of the gate.

3.1.2 Application

Tide gates are required at all sewer outfalls where there is a possibility of the sewer system being flooded by a rise in the level of the receiving waters.

In combined sewer systems without regulators, tide gates usually are installed at the end of the outfall. The outfall is terminated in a concrete headwall and the tide gate is mounted on the face of the wall. This tide gate location very often makes maintenance work difficult, particularly if the gate is partially submerged. In this case, boats may be required to carry out normal maintenance procedures.

When regulators are constructed on combined sewer systems, consideration should be given to locating the tide gate in a chamber adjacent to the regulator chamber. This has several advantages. The gate will be easier to service than when located in or near the water. The gate can be inspected in conjunction with maintenance visits to the regulator. Provision can be made in the chamber for use of stop logs downstream of the flap gate so that the gate can be serviced "in the dry" if necessary. A further advantage of installing the gate in a chamber is that motorized equipment can be operated directly over the chamber for use in maintaining or replacing the gate.

The chamber width should provide a minimum of 6 inches clearance between the tide gate and the wall. When multiple gates are used the post or column between the gates should have a minimum width of 2 feet.

Shaft and surface openings should be provided with dimensions of about 1 foot greater than those of the gate. When the gate is always partially submerged it may be desirable to install two gates in series to provide a safeguard against one gate being clogged open. In the latter case the chamber can be enlarged to permit this series installation of gates.

3.1.3 Description

Tide gates are available in three types depending on the material used for the flap as follows: (1) Cast iron; (2) pontoon; and (3) timber.

Tide gates with cast iron flaps are available within circular, square or rectangular shapes. Circular flaps range from 4 to 96 inches in diameter. Sizes available from one manufacturer are given in Figure 3.1.3.1 Square and rectangular cast iron tide gates are available in sizes ranging from 8 inches square to 96 inches square. Sizes available from another manufacturer are shown in Figure 3.1.3.2.

Pontoon flap gates are fabricated of sheet metal to form a number of air cells. This increases the buoyancy of the gate and enables it to open under a smaller differential head than is possible with a cast iron gate, particularly 48 to 120 inches, as shown in Figure 3.1.3.3. Square and rectangular shapes are available in sizes ranging between 48 inches square to 120 inches square as shown in Figure 3.1.3.4.

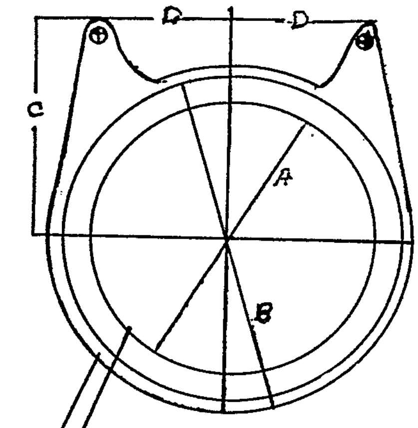
Type 316 stainless steel is used for large pontoon fabrications. It is weldable, has physical properties comparable to mild steel, and has satisfactory corrosion resistance.

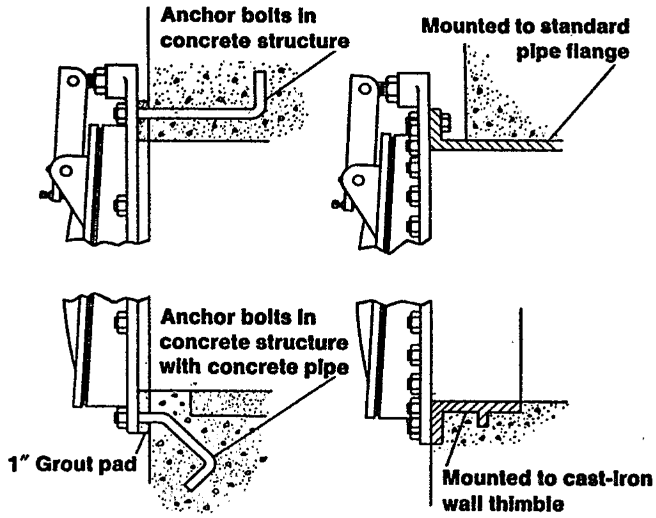
In recent years, there has been a revival of timber tide gate flaps. This is due in all probability to the pollution in major harbors. Teredos and ship worms cannot live in polluted water. Therefore, the timber will have a long life. Warping is partially prevented by the use of strong backs bolted to the timbers. These are pieces of railroad track that are hot dip galvanized after all machining is completed. Timber gate leaves are not as tight as the metal tide gates because it is difficult to seal the space under the metal seat band between the timbers.

Timber gates are available in the same sizes as the larger size cast iron square and rectangular gates. One manufacturer's models are shown in Figure 3.1.3.5. Creosoted yellow pine is frequently used in timber gates. Recently greenheart timber has been introduced. This wood grows only in British Guiana, requires no wood preservative treatment and is resistant to wood-destroying fungi. It is extremely dense, weighing approximately 70 pounds per cubic foot, and is well suited for flap gates as it requires little additional weight to offset buoyancy Greenheart also is more resistant to seasoning splits and checks than common structural woods, it machines well and resists distortion well. The following table compares the working stresses for greenheart and other structural woods.

FIGURE 3.1.3.1

CAST IRON CIRCULAR FLAP GATES





METHODS OF INSTALLATION

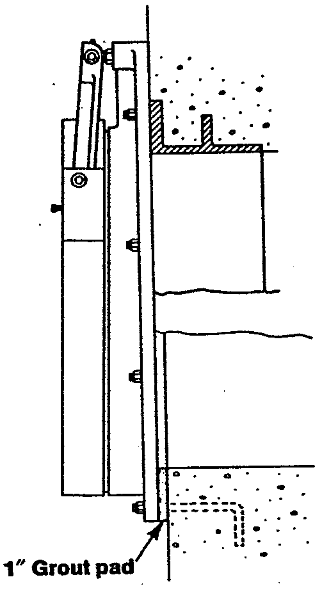
TABLE OF DIMENSIONS (Inches)

A—Dia.	B	C	D	E	F	G
4	9	5 9/16	2 3/4	4	7 1/2	3/4
5	10	5 9/16	3 1/4	4	8 1/2	3/4
6	11	5 9/16	3 1/2	4	9 1/2	3/4
8	13 1/2	6 3/4	4 3/4	4	11 3/4	3/4
10	16	8 1/4	5 7/8	4	14 1/4	7/8
12	19	9 3/4	6	4	17	1
14	21	12	7	4	18 3/4	1 1/8
15	22	12	7 1/2	4	20	1 1/8
16	23 1/2	12 3/4	8	4	21 1/4	1 1/8
18	25	14 1/2	9 1/4	4	22 3/4	1 1/4
20	27 1/2	16 1/8	9 1/4	6	25	1 1/4
21	29	16 1/8	9 3/4	6	26	1 1/4
24	32	19 1/2	11 1/2	6	29 1/2	1 3/8
27	35 3/4	21 5/8	12 3/4	6	33	1 1/2
30	38 3/4	24	14	6	36	1 1/2
36	46	28 1/2	17 1/2	6	42 3/4	1 5/8
42	53	33	18 1/2	6	49 1/2	1 3/4
48	59 1/2	38	21	6	56	2
54	66 1/4	42 1/2	24	8	62 3/4	2 1/4
60	73	47	26	8	69 1/4	2 1/4
66	79	51 1/2	28	8	76	2 1/4
72	86 1/2	54 1/8	30 1/2	8	82 1/2	2 1/2
78	93 1/8	60 1/2	33 1/4	8	89	2 1/2
84	99 3/4	65 1/4	35 1/2	10	95 1/2	2 5/8
96	113 1/4	74 1/2	40 1/2	10	108 1/2	2 3/4

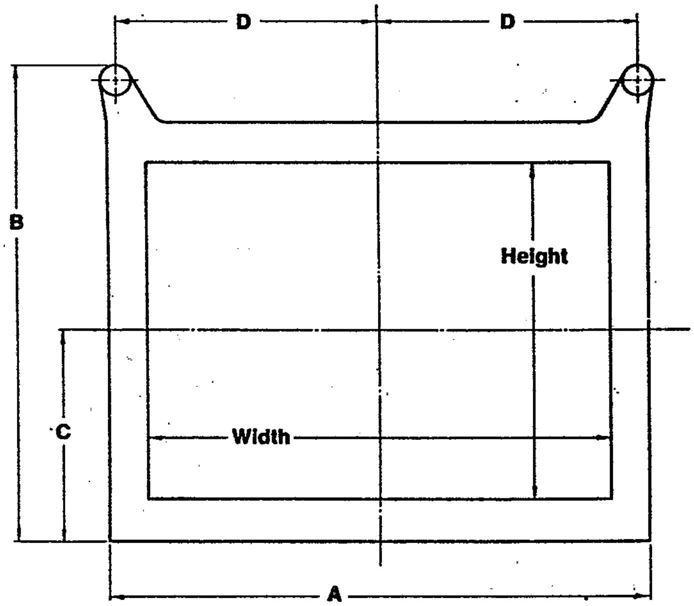
Courtesy Rodney Hunt Co.

FIGURE 3.1.3.2

SQUARE AND RECTANGULAR CAST IRON FLAP GATES



Recommended methods of installation include mounting on cast-iron wall thimble (top) or mounting on concrete wall (bottom)



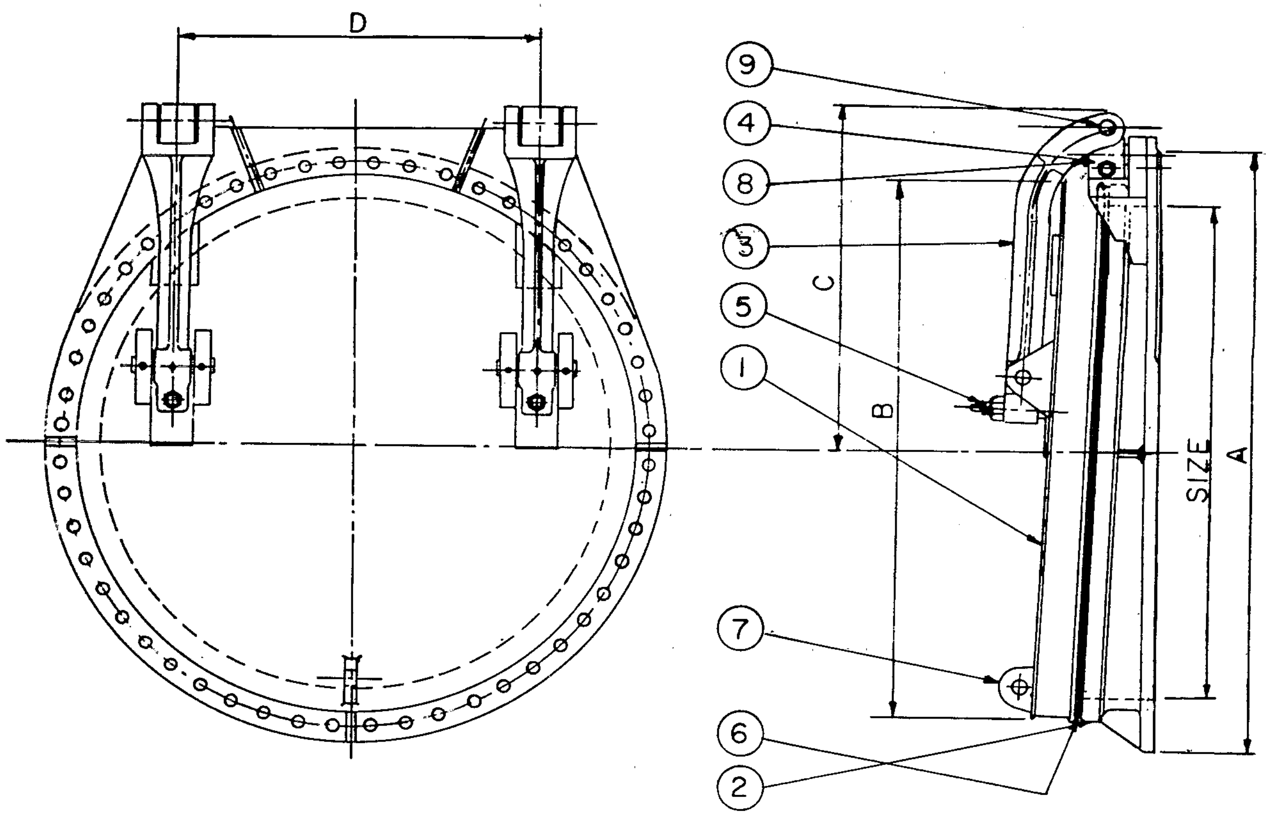
"E" thickness of mounting flange. Mounting flange drilled for mounting on a wall thimble or directly to concrete.

TABLE OF DIMENSIONS (Inches)					
Width x Height	A	B	C	D	E
12 x 12	19	19 1/4	9 1/2	7 3/4	1
18 x 18	25	27	12 1/2	11 1/16	1 1/4
24 x 24	32	35 1/2	16	14 9/16	1 3/8
30 x 30	38	43	19	18 1/16	1 5/8
36 x 24	45	36	16 1/2	21	1 1/2
36 x 36	45	51	22 1/2	21 1/4	1 5/8
36 x 48	45	66 3/4	28 3/4	21 1/4	2
36 x 54	46	72 1/2	32	21 1/2	2 1/4
42 x 42	51 1/2	58 3/4	25 3/4	24 1/4	1 3/4
48 x 18	56	28	13	26 5/8	1 5/8
48 x 24	56	35 1/2	16	26 13/16	1 5/8
48 x 30	56	43	19	27 1/16	1 5/8
48 x 36	56	51	22 1/2	27 1/16	1 5/8
48 x 48	57 1/2	66 3/4	28 3/4	27 1/2	2
48 x 60	57 1/2	81 3/4	34 3/4	27 1/2	2 1/4
54 x 54	64	74 1/2	32	30 1/2	2 1/4
60 x 36	69 1/2	51 1/4	22 3/4	33 1/4	1 3/4
60 x 48	69 1/2	66 3/4	28 3/4	33 1/2	2 1/4
60 x 60	69 1/2	81 3/4	34 3/4	33 1/2	2 1/4
60 x 72	69 1/2	95 1/2	41 1/2	33 1/2	2 1/4
72 x 48	81 1/2	64 3/4	28 3/4	39 1/2	2 1/2
72 x 60	83	82 1/2	35 1/2	39 1/2	2 1/4
72 x 72	83	95 1/2	41 1/2	40 1/4	2 1/2
84 x 84	95	112 1/2	47 1/2	46 1/2	2 3/4
96 x 96	107	124 1/2	53 1/2	52 1/2	2 3/4

Courtesy Rodney Hunt Co.

FIGURE 3.1.3.3

CIRCULAR PONTOON FLAP GATES



DIMENSIONS — INCHES

Size Diam.	A	B	C	D
48	59½	54	37¼	34
54	66¼	60	40¼	38
60	73	66	43¼	42
66	80	72	46¼	46
72	86½	78	49¼	50
78	93	84	52¼	54
84	99¾	90	55¼	60
90	104	96	58¼	64
96	113¼	102	61¼	70
102	119¾	108	64¼	74
108	126	114	67¼	77
114	132½	120	70¼	80
120	139	126	73¼	83

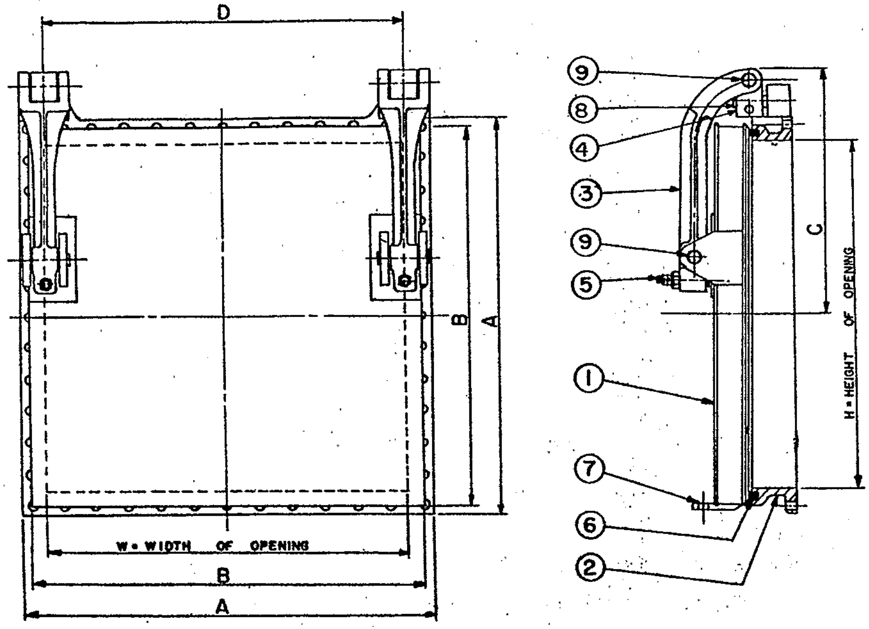
Dimensions are approximate.

1. - Flap—Stainless Steel
2. - Frame—Cast iron
3. - Hinge link—Cast steel
4. - Hinge—Bronze
5. - Adjusting screw—Bronze
6. - Seat—Neoprene
7. - Lifting eye—Stainless steel
8. - Hinge post—Bronze
9. - Pins—Bronze

Courtesy Caldwell-Wilcox Co.

FIGURE 3.1.3.4

SQUARE AND RECTANGULAR PONTOON FLAP GATES



DIMENSIONS — INCHES

Size Wdth. x Hgt.	A	B	C	D
48 x 48	57	54	37¼	48
54 x 54	63	60	40¼	54
60 x 60	69	66	43¼	60
66 x 66	75	72	46¼	66
72 x 72	81	78	49¼	72
78 x 78	87	84	52¼	78
84 x 84	93	90	55¼	84
90 x 90	99	96	58¼	90
96 x 96	105	102	61¼	96
102 x 102	111	108	64¼	102
108 x 108	117	114	67¼	108
114 x 114	123	120	70¼	114
120 x 120	129	126	73¼	120

These gates are also available in rectangular types in any width and height in 6" increments; for example—60"x48"; 72"x96"; 108"x120", etc.
Dimensions are approximate.

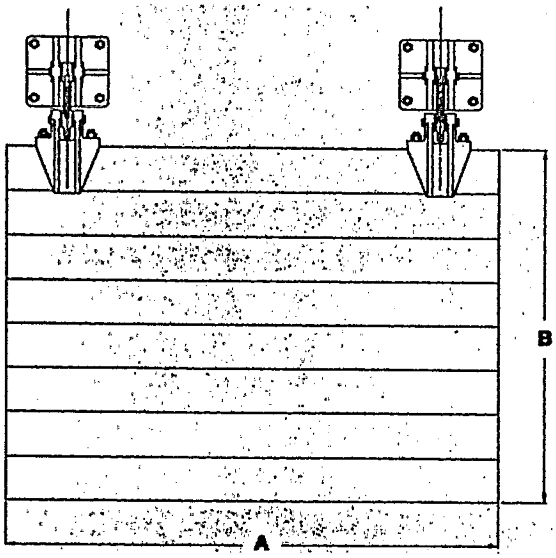
CONSTRUCTION

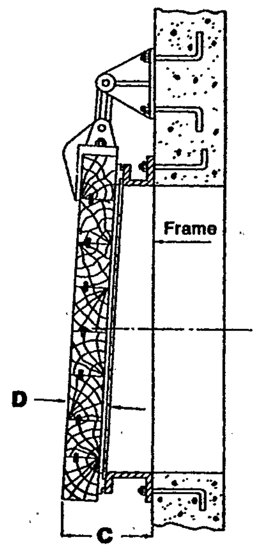
1. - Flap—Stainless steel
2. - Frame—Cast iron
3. - Hinge link—Cast steel
4. - Hinge—Bronze
5. - Adjusting screw—Bronze
6. - Seat—Neoprene
7. - Lifting eye—Stainless steel
8. - Hinge post—Bronze
9. - Pins—Bronze

Courtesy Caldwell-Wilcox Co.

FIGURE 3.1.3.5

SQUARE AND RECTANGULAR TIMBER FLAP GATES





TIMBER FLAP VALVE DIMENSIONS				
Width x Height	A	B	C	D
36x48	46	58	13	3¾
36x60	46	70	13½	3¾
48x48	58	58	13¾	4½
48x60	58	70	14¼	4½
60x36	70	46	14	5½
60x48	70	58	14¾	5½
60x60	70	70	15¼	5½
72x48	82	58	14¾	5½
72x60	82	70	15¼	5½
72x72	82	82	16	5½
84x48	94	58	17¾	7½
84x60	94	70	18¼	7½
84x72	94	82	19	7½
84x84	94	94	19½	7½
96x48	106	58	17¾	7½
96x60	106	70	18¼	7½
96x72	118	82	19	7½
108x36	118	46	19	9½
108x48	118	58	19¾	9½
108x60	118	70	20¼	9½
120x72	130	82	21	9½
120x84	130	94	21½	9½

Courtesy Rodney Hunt Co.

ALLOWABLE WORKING STRESS (psi)

Timber		Parallel to Grain		Perpendicular to Grain
Compression	Tension	Shear	Compression	
Yellow Pine	1550	2000	135	455
Cypress	1466	1733	133	300
Douglas Fir	1100	1450	95	390
Greenheart	3000	3300	400	1500

Greenheart (Ocotea Rodioli) is used for the lock gate sills in the Panama Canal. The greenheart is eaten by the teredo and has an average life of 9 years compared to 2 to 4 years for oak or pine. The high temperature of the water is considered responsible for its short life in Panama since there are records of its use in England and Germany for periods up to 40 years.

Generally cast iron gates are used for smaller sizes and pontoon or timber gates for the larger sizes. The use of cast iron for large flaps makes the gate difficult to handle and increases the differential head under which the gate will open. For this reason New York City limits the application of cast iron gates to 48 inches square.

The choice between timber and pontoon types depends on several factors. In New York City, where tidal waters are corrosive, the life span of pontoon gates is 10 to 12 years compared to upwards of 30 years for timber gates. The pontoon gates have a more stable shape than timber gates but eventual corrosion of the plates causes the air cells to fill with water and destroy the flap buoyance. One procedure to prevent this is to fill the cells with a plastic such as styrofoam. While timber gates have greater life they are subject to warping and destruction by marine borers.

Tide gates may be installed on concrete walls by use of anchor bolts, cast iron pipe flanges or cast iron wall thimbles embedded in the concrete wall. The use of a wall thimble is preferable.

The gate is attached to the frame by at least two hinge arms. Each arm should be provided with two pivot points with lubrication fittings. Proper maintenance requires periodic lubrication of these fittings.

The seat between the flap and frame can be either bronze or resilient material such as neoprene or Buta-N rubber. The use of a resilient material is preferable to achieve water tightness.

The gate should be provided with a lifting eye on the lower edge. It is desirable to provide a permanent chain from the lifting eye to an accessible point so that the flap can be opened when clogged, for removal of debris.

Tide gates require periodic inspection during low tides for cleaning and during high tides for observation as to their water tightness.

3.2 Design

3.2.1 Guidelines

The addition of a tide gate, a regulator and the required chambers at a sewer outfall increases the head loss through the sewer and raises the backwater effect in the sewer during periods of high stages in receiving waters. This increase of backwater levels may not be great enough to be of serious concern; however, as a precaution, the possible change in the hydraulic profile should be computed for anticipated high water levels in the receiving waters.

Since the peak storm flow and the maximum tide or stream elevation are both events of short duration, the probability of the simultaneous occurrence of the two events may not be very great and is outside the scope of this Manual. The designer must use his judgment in selecting the tide or stream high water level for use in his computations.

The additional hydraulic losses resulting from installation of the regulator and tide gate are due to: (1) Loss through the tide gate; (2) losses in the diversion chamber of the regulator; and (3) losses in the tide gate chamber. It should be noted that the friction head loss in the storm sewer downstream of the regulator may decrease due to the lessened discharge resulting from some flow diversion at the regulator.

The discharge through the tide gate will be the flow in the upstream sewer, less the flow diverted to the interceptor. If the tide gate is placed at the end of the storm sewer the flap usually will be the same size as the sewer. If the gate is placed in a chamber it usually will be square or rectangular in shape, with a width equal to the diameter of the upstream sewer, and a height somewhat greater than the water depth upstream of the gate. The gate invert will be the same elevation as the regulator diversion dam.

A typical plan of a regulator structure with a tide gate chamber and the hydraulic profile through the storm sewer and regulator is shown in Fig. 3.2.1. Sample computations for the profile shown in Fig. 3.2.1 are given in the following paragraphs.

The sample computations are based on the data used for design of cylinder-operated gates. In the latter computations the water surface in the diversion chamber was arbitrarily selected at elevation 21.00. The computations which follow indicate that the water surface should be at elevation 21.77 as a result of additional hydraulic losses due to the tide gate chamber. Therefore, in final design the computations shown herein for the cylinder-operated gate should be revised to reflect this higher water surface.

3.2.2 Design Formulas

The head loss through tide gates may be assumed to be 0.2 feet, according to some gate manufacturers. The design criterion of one city is to select a tide gate with an area 10 to 15 percent greater than the area of the combined sewer and to place the invert of the gate not more than 0.5 feet above the invert of the combined sewer. One city specifies that the head loss through the gate shall not exceed 0.5 feet; another city specifies a maximum of 0.33 feet. The relation between tide gate head losses and conduit velocities is presented in "Hydraulics Design Chart 340-1" in "Hydraulic Design Criteria" by the U.S. Army Engineers, Waterway Experiment Station, Vicksburg, Mississippi. With respect to this chart the design criteria state:

1. Flap gate head losses can be determined by the equation: $H_L = K V^2/2g$

where

H_L = head loss in ft. of water

K = head loss coefficient

V = conduit velocity in ft. per sec.

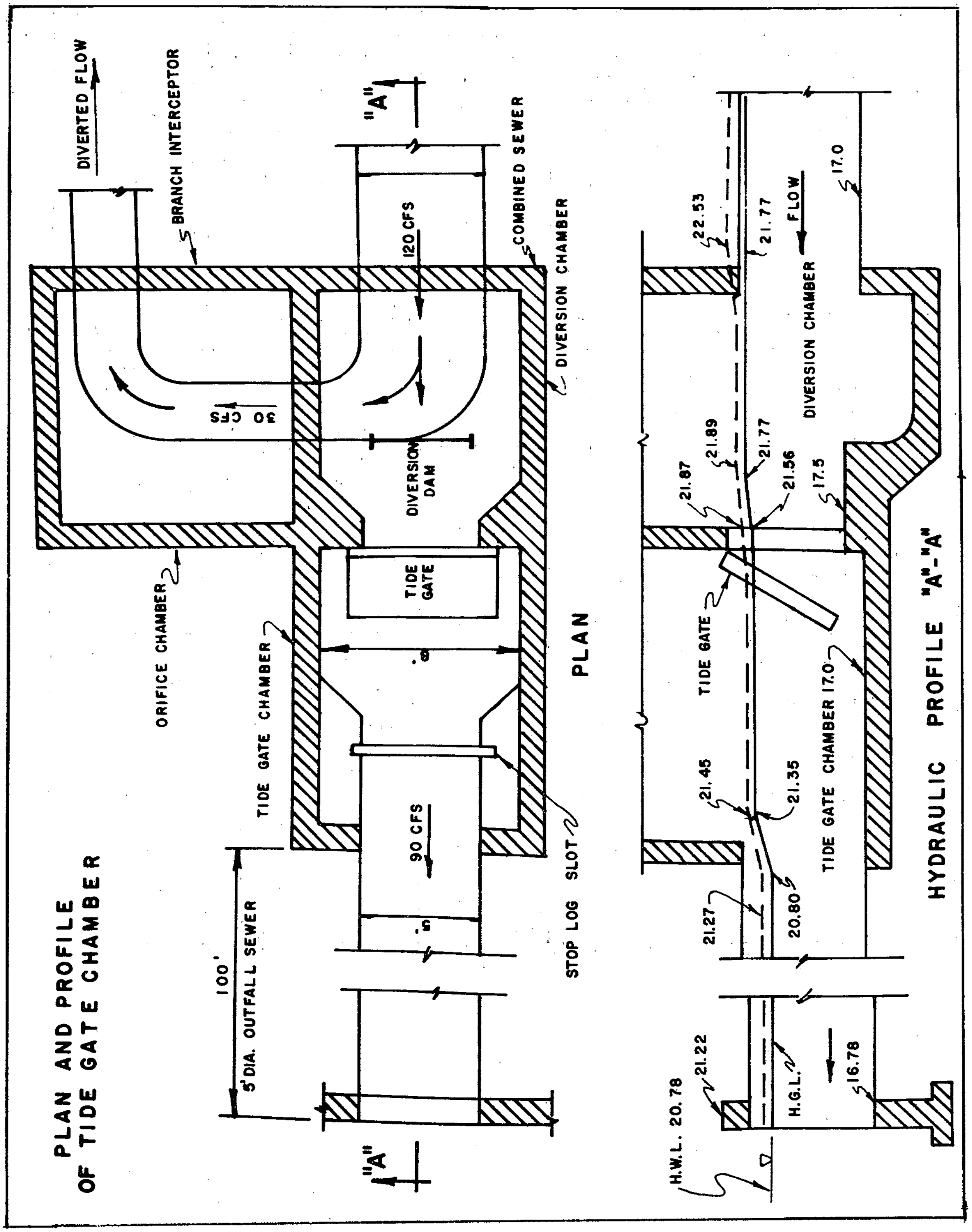
2. Hydraulic Design Chart 340-1 presents head loss coefficients for submerged flap gates. The data result from tests by Nagler[6] on 18-and 30-inch diameter gates.
3. Modern tide gates are heavier but similar in design to those tested by Nagler. It is suggested that Chart 340-1 be used for design purposes for submerged flow conditions until additional data become available. Head loss coefficient data are not available for free discharge.

The chart in Fig.3.2.2 relates head loss through the gate to the velocity in the conduit. This chart is based on circular gates attached to circular conduits of the same size; hence the velocity in the conduit and through the gate will be similar. In the case of the regulators considered in this Manual the velocity in the upstream sewer and through the tide gate may differ; therefore, in the sample computations herein, the velocity through tide gate has been used in connection with Fig. 3.2.2.

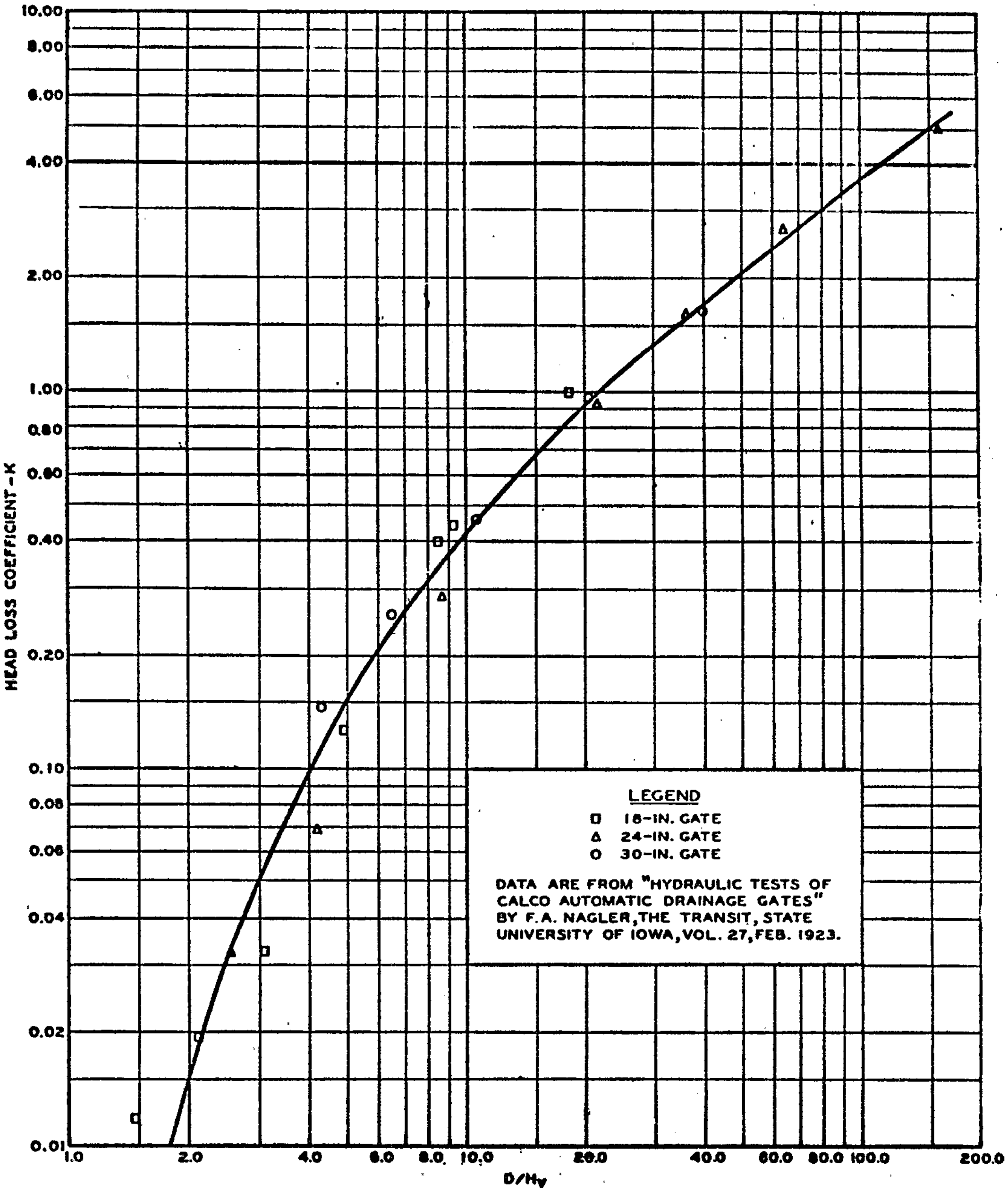
Other hydraulic losses through the regulator are computed from the applicable formulas outlined in the subsection on cylinder-operated gates, in Section 2 of this manual of practice.

[6] F. A. Nagler, "Hydraulic tests of Calco automatic drainage gates," *The Transit*, State University of Iowa, vol. 27 (February 1923).

FIGURE 3.2.1



PLAN AND PROFILE
OF TIDE GATE CHAMBER
DIVERTED FLOW
BRANCH INTERCEPTOR
"A"
COMBINED SEWER
120 CFS
30 CFS
DIVERSION CHAMBER
DIVERSION DAM
ORIFICE CHAMBER
TIDE GATE
TIDE GATE CHAMBER
8'
90 CFS
STOP LOG SLOT
5'
100'
5' DIA. OUTFALL SEWER
PLAN
22.53
21.77
FLOW
17.0
DIVERSION CHAMBER
21.87
21.89
21.77
21.56
17.5
TIDE GATE
TIDE GATE CHAMBER 17.0
21.45
21.35
21.27
20.80
H.W.L. 20.78
21.22
H.G.L.
16.78
HYDRAULIC PROFILE "A"-"A"



EQUATIONS

$$K = \frac{H_L}{H_V} \; ; \; H_V = \frac{V^2}{2g}$$

NOTE: K = HEAD LOSS COEFFICIENT
H_L = HEAD LOSS, FT
D = CONDUIT DIAMETER, FT
V = CONDUIT VELOCITY, FT/SEC
g = ACCELERATION OF GRAVITY, FT/SEC^2

FLAP GATES
HEAD LOSS COEFFICIENTS
SUBMERGED FLOW

HYDRAULIC DESIGN CHART 340-1

WES 8-60

Courtesy Waterways Experiment Station, Corp. of Engineers

3.3 Sample Computation
Flap Gate

Note: Use same data as for cylinder-operated gate.

Assume that design high water level in receiving stream is same as normal depth in combined sewer for 10-year storm prior to construction of regulator. Determine effect of regulator on water surface in combined sewer.

V = Velocity
V_1 = upstream velocity
V_2 = downstream velocity
d = depth of flow
D = diameter
g = acceleration of gravity

	ELEVATION		
	Invert	HGL	EL
Data on combined sewer prior to construction of regulator			
At regulator	17.00		
$D = 5.0'$ $s = 0.0022$ $Q = 120$ cfs			
$d = 4.0'$ v (full) = 6.2 fps		21.00	
$V = 7.0'$ $V^2/2g = 0.76$			21.76
At outlet 100′ from regulator			
$100 \times 0.0022 = 0.22$	16.78	20.78	21.54
∴ Design high water level is		20.78	
After construction of regulator			
Diverted $Q = 30$ cfs			
Q in storm sewer = 120 − 30 = 90			
Storm sewer $L = 100'$			
Downstream end	16.78	20.78	
d (normal) = 3.20			
d (actual) = 4.00			
∴			
compute backwater effect in storm sewer			
$V = 5.4$ fps $V^2/2g = 0.44$			21.22
Compute backwater curve upstream by standard-step method (see Table XVII, ASCE Manual No. 37)			
Upstream end	17.00	20.80	21.27
$V = 5.5$ fps $V^2/2g = 0.47$			

3.3. Flap Gate

	ELEVATION Invert	HGL	EL
Flap Gate Chamber			
$V = \frac{90}{8 \times 4.5} = 2.5 \quad \frac{V^2}{2g} = 0.10$ (in chamber)			
Entrance Loss			
$0.5 \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right) = 0.5\,(0.47 - 0.10) = .18$	17.00	21.35	21.45
Neglect friction loss			
Flap gate loss - use Figure 3.2.2 Use 60" x 60" gate			
$V = \frac{90}{5.0 \times 4.0} = 4.5$ fps			
$V^2/2g = 0.31$			
$\frac{D}{H_V} = \frac{5.0}{.31} = 16 \qquad K = 0.7$			
$H_L = 0.7 \times 0.32 = 0.21$			
$21.56 + 0.21 = 21.56$			
Upstream of flap gate	17.50	21.56	
$21.56 + 0.31$			21.87
Diversion Chamber			
$V = \frac{90}{8 \times (21.5-17.5)} = 2.8$ fps			
$V^2/2g = 0.12$			
Contraction loss $= 0.1 \left(\frac{V_2^2}{2g} - \frac{V_1^2}{2g}\right) = 0.1\,(0.31 - 0.12) = 0.02$	17.50	21.77	21.89
Outlet loss $= \left(\frac{V_1^2}{2g} - \frac{V_2^2}{2g}\right)$	17.50	21.77	21.89
$= 0.76 - 0.12 = 0.64$			
Combined sewer	17.00	21.77	22.53

Therefore the installation of the regulator will raise the water surface of the combined sewer upstream of the regulator during a 10-year frequency storm by 21.77 – 21.00 or 0.77 feet.

SECTION 4

INSTRUMENTATION AND CONTROL OF REGULATOR FACILITIES

CONTENTS

4.1. Elements of Instrumentation
Activation: General

This section is directed principally at the more recent techniques of control and instrumentation that may be adaptable to the development of more suitable means of regulation. It is prepared from the viewpoint of the electrical and instrumentation designer: the subject matter pertains specifically to equipment for:

1. Metering - means of measuring and sensing system conditions;
2. Telemetering - means of transmitting data to some data gathering points;
3. Communications - means of interconnection from remote to data gathering point;
4. Data Handling - means of collection, display, storage and manipulation of data;
5. Decision Making - means of translation of data to control requirement;
6. Supervisory Control - means of dispatching control to activation facilities; and
7. Activation of Control Element - means of activating regulating device.

These concepts imply the establishment of a total system control center at which point a complete system of sewers and all its associated components appurtenances may be under the direct supervision and control of management. This will be a necessity at some large jurisdictions and to a lesser degree at smaller jurisdictional systems where only a few manually supervised elements are used. The general principles will be the same and installation in the instrumentation-control field should embody the concept of the ever-widening scope of which each element may become a part.

The development of the regulation control center must be a step-by-step undertaking, starting with the gathering of data and implementing remote control while gradually increasing the control center's data handling capability with additional equipment.

Present regulator practice makes use of both fixed and adjustable hydraulic devices such as: Dams, horizontal orifices, vertical orifices, overflow weirs, leaping weirs, adjustable gates, tipping gates, and siphons.

The amount of flow in each case is seldom measured, but by nature of the regulator design is supposed to be within certain limits based upon hydraulic calculations under assumed conditions such as, for example, free discharge downstream. Two major conditions arise, however, which make the effectiveness of present regulation practices a continuing problem. First, the desired orifice becomes altered or gates malfunction because of clogging and second, downstream channels may become surcharged. Since there is usually no means of indicating flow, there is no means of indicating the degree of regulation which occurs.

In the case of fixed or static regulator stations, nothing can be done to correct a control malfunction except to remove the obstruction. Sensing of the problem can be accomplished and communicated with suitable identification to supervisory personnel and conveyed to maintenance personnel for remedial action.

Adjustable regulating stations can be monitored, and gate openings can be remotely adjusted from a central control, provided such facilities are made available, to compensate for the changed conditions affecting the station. If, for example, a partial blockage restricted an orifice, the gate could be opened wide to effectively change the port area to allow the desired flow.

With regard to instrumentation and activation facilities, instrumentation only applies to static regulators. In this respect, monitoring devices with suitable communication facilities can detect and indicate the hydraulic conditions either in the vicinity of the regulator or at some point on the system that reflects the operating condition at the regulating station.

Both instrumenation and activation facilities can be used with automatic dynamic regulators. In this application instruments can be used for sensing the conditions and for handling information, whereas the activation facilities may be employed to manipulate the configuration of the regulating stations.

The concept of controlling an individual regulating station is a simplification of the concept of regulation as it applies to a complete waste water system. Complete system regulation requires the development of facilities and techniques to efficiently route, limit divert, transfer, and "park" or store waste waters.

Regulation of sewers, like any other control system, can be broken down into the following elements: Measurement; status determination; information or data gathering; manipulation of data; decision making; execution, verification, and evaluation.

Measurement and status determination is effected by sensors of various types developing electrical signals to represent such objectives as flow, level, head, differential pressures, gate positions, and equipment status. Data gathering equipment is used to condition or code electrical signals for transmission over usual communications channels. Equipment for manipulation of data consists of indicators, recorders, loggers, alarms, and computers. Decision making is performed by supervisory personnel or by computer programs. Execution is by dispatching service and maintenance personnel, or by employment of supervisory control equipment for remotely operating the stations. Verification is carried out by updated measurements and status indications. Evaluation is a decision or judgment-making process, either manual or automatic, to determine the results of the initial execution of a control action. Repeated control cycles are performed until satisfaction occurs.

The control equipment may take many forms. Generally, sluice gates or shear gates will be employed to limit and re-route flows; whereas pumping equipment may be used for transfer to and from off-system storage or holding facilities. Pumps also may be used for transfer within a system to take full advantage of system capacities for storage. Not only is it essential to manage storm water flows, but also some treatment, chlorination for instance, may be required. In such instances, the control equipment will likely include remote dosage monitoring of chlorinators with the rate of chlorine feed automatically paced by the measured rate of storm water flow. Additional sensors may be required for determination of such situations as chlorine residual and contact time before release from the system.

Usually, means to transfer flow has a variable capacity in order to match the incoming flow. Equipment used may include control and monitoring facilities for variable speed pumping equipment.

An interesting example of regulated pumping is the case where pumping from the downstream terminal of an interceptor sewer is performed only at a rate of flow necessary to lower the hydraulic gradient so that incoming flow to the interceptor can be accepted, thus always pumping at the minimum head necessary to match the incoming flow. Under this condition the pump station wet well is at minimum level at maximum flow or maximum level at low flow or low pumping rates. Moreover, the capacity of the interceptor is used to its maximum capability for storage. Such a scheme becomes quite complex and requires programmed control which may either be built into the control equipment or be effected by computer.

Although the treatment of sewage falls outside the scope of regulation, those who manage the regulation of a complete system must be concerned with the operating capacities of plants and with outfall sewer conditions. Thus, plant flows and receiving body water levels are pertinent data required at the regulator control center. Additional stream monitoring data pertaining to water quality also might be transmitted to the control center for overall guidance and record purposes. Additional data such as weather reports from the Weather Bureau and rain gauge data telemetered to the control center should assist management to effect adequate controls when a storm impends.

Successful development of means to effectively regulate complex sewerage systems, either combined sanitary and storm wastes or separated wastes, requires a management information and control system. In spite of the interest that exists, as is evident from the number of articles and discussions on the subject, few such information systems actually have been implemented. One reason for this has been cost. Another reason is that there is neither a well-defined perception of the range and extent of the information and control requirements nor tried and proved methodology for its implementation. The traditional justification for information systems, monitoring of sewer levels for example, has been based to a great extent only on reduction of operating costs. Justification for an information system, in the future, should be based on the net worth of the information. The net worth may be defined as the difference between the value of the information obtained and the cost of obtaining this information. Traditionally, the instrument and control designers determine the cost; Management must determine its value.

Information is even more valuable when it flows to and from management, allowing decision-making and execution, and thus more responsive control. From an instrument and control standpoint, a sewer monitoring system is an example of information flowing only one way—to management. By implementing supervisory control equipment for remote control, the value of the information is greatly increased if it allows management to exercise real time control. Real time control is the ability of management to detect and correct a deviation from plan or standard before the deviation becomes so great that it is not possible to return to the original plan or standard. It is not necessarily instantaneous or even fast control, since the speed of response required for a real time control depends upon the nature of the activity being controlled. For waste water systems control the speed of response usually may be in minutes and hours.

The constraining factors in the development of information and control systems are control facilities, communications, power, and site preparation. None of these except hardware is necessarily difficult and needs no particular discussion in this manual. Very little has been accomplished, however, in the design and development of suitable hardware for sensing the rate of flow. In this respect, and at the present stage in the art of instrumentation, it should be recognized that immediate values of velocities and levels are of more value than totalized quantities of waste water.

Determining the specific information and control requirements is probably the most critical part in the design of any information and control system. These needs also must be taken into account in the future design of sewer systems. The cost to harness a regulating station with monitoring devices and remote control has little relationship to the size of the station. In future designs it may be prudent to employ only a few large regulating stations rather than a large number of smaller ones. It will also be important to locate regulators where they can be conveniently attended and serviced.

Information and control systems offer tremendous benefits to the water pollution control agencies and represent practically a new frontier for instrumentation and data handling. Examples of beneficial regulating practices are many.

In one jurisdiction where storm water overflow or diversion occurs the major purpose is to avoid hydraulic overload of treatment works in addition to avoiding flooding of local areas. Overflow chambers are metered and other overflows are electrically controlled and telemetered to the plants for their operation. The control center is located in a main office building to which data are transmitted concerning rainfall incidence and waterway elevations. The engineer of waterway control provides information to the engineers of treatment plant operation who decide the course of action.

To achieve the objective of using available storage within the existing combined sewers for regulating storm water flows, the jurisdiction installed a "Computer Augmented Treatment and Disposal System." Reduction of sewage overflow frequency and magnitude also is part of its objective. When overflows cannot be avoided, the system controls discharges at selected stations to minimize harmful effects on marine life or public beaches. Storage control is effected through remote electric control and local automatic controls of the pneumatic type. These local control units come into use only when the remote control equipment fails. Gates at each outfall are remotely controlled by electronic circuitry. Remote control of the datum level chosen at each location is carried out by transmission of an operation command signal to the remote terminal equipment. When the desired datum has been reached, a command signal is transmitted to the remote terminal to deactivate the equipment. When the remote control units fail, local automatic control is restored. The control center is outfitted with an operator's console and wall map which relate to the system. Data are telemetered to a central location over leased telephone lines and the information is entered in a process control computer which also directs data gathering.

At another jurisdiction, it has been found that about 80 percent of the annual overflow volume is discharged from 20 overflow structures. Consideration is being given to the elimination of all overflows in some areas. This would be accomplished by bringing all combined sewers to a central gate chamber where the gates would be power-operated and remotely controlled by means of a telemetering and supervisory control system. The chamber would be operated as a single overflow during periods of wet weather. Control of the overflow would be accomplished by a central operator who would have telemetered data concerning waste water flows at treatment facilities, the water level and available storage capacity in the intercepting sewers and the water level at critical points in the system and at the gate chambers. Thus the operator at the control center could affect the flow throughout the system. Use would be made of the monitoring and telemetering system in the operation and control of the system, in particular for surveillance of the pump stations for malfunctions, failures, and other system problems.

The benefits of flow control in combined sewers at one sanitary district appear to warrant a significant expenditure for equipment. Flow in the sewer system was studied, using pneumatic depth of flow recorders placed at key positions. Purpose of the gauging was to determine the extent of system loading, in addition to gathering other data. In 1960 it was found that a volume equal to 10 mgd in the joint interceptor sewer required an expenditure of $600,000 based on dry-weather flow. This means that every 10 mgd of storm water removed from the joint interceptor had an equivalent worth of $600,000 of sewer capacity. This concept of providing capacity for sanitary flows led to consideration of using powered regulators and a system of controls for removing flow from, or permitting flow to remain within the interceptor system. Effective control of the regulators was seen as a measure for reducing the overall quantity of raw sewage diverted to receiving waters during rainfall periods. The system being considered included an integrated system of regulator operations with supervisory control of key regulators. The regulator operation would be based upon interceptor usage. Power-operated gates controlled by a supervisory system from a control center would use telemetering to provide information concerning the gate positions, flows, and flow levels in the interceptor sewers. Data collected would inform the operator of the system the situation and permit system adjustments. The operator would have the choice of bypassing flow quantities at certain locations. Evaluation of the receiving water conditions at various locations would be made by the operator who would make required adjustments. Automatic readout of data for analysis and use of manual or computer techniques are visualized.

Still another example of the requirements of monitoring and supervisory control is the situation in another sewer district where a second conduit will be installed to parallel an existing relief sewer. In this case, it will be necessary to retain peak flows in one of the two conduits until a quantity is reached where it is possible to divide the flow between the two sewers and still maintain adequate velocities in both. Obviously, some form of level and velocity monitoring and power-operated gates are likely solutions.

4.2 Metering

Regulation, herein is defined as the control of waste water flow. Control implies exercising direction over the amount, the rate of flow, and the routing of the flow. Effective regulation therefore must be accompanied by some means of measurement.

Important characteristics of waste water flow are: (1) Conduits usually are only partially filled; (2) substantial amounts of debris are carried; (3) flow is not under pressure other than gravity; and (4) conduits frequently are located at considerable depths below grade.

Conventional measuring devices used on water distribution systems are not practical for measuring waste water flow. Open channel flow metering structures can be employed with some degree of success but they are accompanied by additional problems peculiar to the waste water system. Such structures can become very large and expensive and are most appropriately placed near the surface where they are more accessible. Open channel meters are not only directional, but also become inoperative whenever they become submerged.

Always a problem of using an open channel metering structure is the measurement of water level. Most of the open channel flowmeters are designed with the intent of using floats for measuring levels. The use of floats in waste water metering practices only can be practical for temporary meters or those which will be installed within stilling wells and continuously purged with clear water, or those which can be continually supervised and maintained. The use of bubbler systems for measurement of differential levels across Parshall flumes has been employed with greater success. Bubbler systems for direct level measurements provide some advantages over floats.

None of the conventional flow measuring devices is designed for the express purpose of waste water measurement and all presently in use are designed for a much narrower range and higher accuracy than would be required for the monitoring of waste water flow. Therefore, the practice of regulation can not be expected to be any more successful than the available means for sensing flow.

Successful regulation practices will depend on the development of new methods, principles, and designs of flow measuring devices adaptable specifically to sewers and waste water characteristics and environments. Unfortunately, the present state of the art of waste water flow metering is seriously behind most other metering accomplishments.

Managers of sewer systems must make known these requirements to research, development, and manufacturing organizations. Rather than concentrating on the development of flowmeters, for example, it might be wise to explore the development of a pair of sensors, one to determine level and the other velocity. Then with level and velocity as basic data, the flow could be computed by taking into account the conduit configuration. Immediate velocity and level measurements would provide management with a better description of the conditions taking place at some point in a conduit than would a flow measurement, and each possibly could be more accurate than some emperical determination reflecting the combination of the two.

Another concept that might be explored is that of sampling for level and velocity measurements. Such an approach might lend itself to the possibility of employing retractable devices or probes for these purposes so that fixed obstructions would not impede the normal transport of solids and debris. Retractable probes could continually sense level and velocity, interrupted occasionally for retraction, so that any debris held by the probes could float on downstream. The probe could not only be level and velocity sensitive, but could also be bi-directional, sensing velocity in either direction. Such a probe should be sufficiently strong to withstand the impact of heavy floatables and be constructed of corrosion-resistant materials. Probes of this type are patented and are presently being considered for development by several instrument suppliers. It may be wise to ease the requirements for accuracy and sensitivity normally expected for metering and settle for greater durability and simplicity of equipment.

Other new developments in the field of measuring liquid velocity and volume flow rates make use of sonar principles to determine transient travel of an acoustic pulse between submerged probes. The meter probe provides an unobstructed flow path without head loss. This equipment is represented to measure flows from 0.02 ft./sec. to 300 ft./sec. continuously and to have successfully measured flow in a 24-foot-diameter conduit at no more than one percent error.

A relatively new device which is convenient for use in measuring liquid level is the controlled leak,—a precision, porous-metal gas flow restricter. This device can be used to bleed gas such as nitrogen from a bottle, at a rate of less than one bubble per second. The back pressure on a pipe bleeding the gas into a fluid bears a direct relationship to the length of the submerged portion of pipe. For measuring liquid level, this allows the use of a nitrogen bottle and controlled leak to replace the usual installation of an air compressor and differential regulator and the accompanying appurtenances normally required for a bubbler system. Such a system can last for many months without refilling a standard size gas bottle.

Solid state pressure, level, and flow transmitters are being developed for telemetering which can be powered over a standard telephone line, thus eliminating the requirement for power at the transmitter site.

4.3 Telemetering

4.3.1 General

Telemetering is a means of Conversion of a measured variable or sensed condition into a representative electrical signal, the transmission of that signal, and its reconversion to a suitable quantitative form which may be displayed, indicated, recorded, logged, or stored and utilized to compute or control.

The selection of the type of telemetering equipment to insure its maximum effectiveness for a particular application requires not only a careful evaluation of the many types available, but more significantly of the design criteria which may be imposed by the particular purpose for which it is applied.

For regulation practices, certain design criteria are dictated by the characteristics of a sewer system. Such design criteria are:

1. Wide coverage—data are required from all over the system and its environs;
2. Relatively small amounts of data—only a comparatively few and in some cases only one point of data is required from each site;
3. Unattended sites—data measurement are frequently required at unattended sites;
4. Uncontrolled environment—data measurement are generally at site of uncontrolled environment; and
5. Alterable and expandable—points of data requirement occur gradually and in small increments, only to keep pace with sewerage system expansions.

From the above, it readily can be determined that certain features of the outlying station telemetering equipment are particularly desirable, such as:

1. Inexpensive;
2. Infrequent service requirement—not more than quarterly or semi-annually;
3. Applicability for wet and corrosive environment;
4. Applicability for unregulated, normal power supply;
5. Applicability for ordinary two-wire telephone;
6. No distance limitations; and
7. Adaptability to grouping of a number of data signals on one communications channel.

Of the many major types of telemetering equipment available, some of the most commonly employed with respect to their transmitted signals are as follows:

4.3.2 Current Type

The output signal is a variable electric current. This type requires a continuous two-wire, fully-metallic individual circuit and is quite limited in distance. It is most suitable for in-plant and on-site telemetering, particularly where electronic instrumentation is involved.

4.3.3 Voltage Type

The output signal is a variable voltage. This type requires a continuous, two-wire fully metallic individual circuit, is limited in distance, and is quite subject to induced interference. In many instances this type requires a shielded circuit.

4.3.4 Frequency Type

The output signal is a variable frequency. This type requires two-wire circuit or the equivalent. It has no distance limitations and is suitable for high speed data transmission. It has not experienced widespread use for water or waste water signaling, perhaps because it generally has been more expensive and more of a proprietary item than other types available.

4.3.5 Pulse Count Type

The output signal is a variable rate of operation of a contact-making device. This type is a simple, inexpensive electro-mechanical system. It requires only a two-wire circuit or the equivalent and has no distance limitations. Primarily, it is used for dynamic variables such as flow or running counters, and generally is not adapted to more static variables such as pressure, level, or position.

4.3.6 Pulse Duration

The output signal is a variable duration of the closure period of a contract-making device being operated at a constant rate. This type is frequently called time-impulse. Usually it is a simple, inexpensive, electro-mechanical system, utilizing any two-wire circuit or the equivalent. It has no distance limitations and commonly has been used for water and waste water signaling of such variables as flows, pressures, levels, and positions.

4.3.7 Digital

The output signal is a coded pulse train. This type of equipment generally is more sophisticated and expensive; however, its signal format is particularly suitable for automatic data logging and data handling. It has very high accuracy and speed transmission, operating over a two-wire circuit or the equivalent without distance limitations. It is the newest in the art of telemetering; although not yet developed to the stage where transmitters specifically designed for waste water measurements are yet available.

4.3.8 Commentary Concerning Telemetering Equipment

In addition to taking into consideration the characteristics of an overall system, the desired features of the particular application, and the most appropriate transmission signal, it generally has been necessary to choose and specify equipment for which there are competitive suppliers. Since pulse duration type of telemetering is furnished by a number of manufacturers and also meets the normal requirements, it is perhaps the best known and most widely used system for control of combined sewer flows. Presently, governmental jurisdictions successfully utilize a large number of pulse duration type telemeters. In such cases it might not be prudent to change unless there is sufficient justification. Advantages of the other types, however, should not be overlooked.

Presently, telemetering consists of remote transmitters and indicating or recording receivers located at pumping stations and treatment plants, or central operations control centers. No automatic data handling other than the pointer or pen positioning on the receiver is usually involved. If additional handling of the data is required, however, other considerations are necessary. For example, a time-impulse signal cannot be adapted for automatic logging. So the various types of telemetering devices should be reviewed with the prospect of automatic data logging in mind.

Automatic data logging and computer data handling require that the data signal be in digital form. It would seem that digital telemetering would be a more suitable choice of equipment. Unfortunately, at this time no primary metering equipment for waste water type measurements has a direct coded digital output. To obtain a digital output signal, it is necessary to employ a conventional primary meter with a current or voltage output, (an electrical analog output), then to use an analog-to-digital converter to obtain a digital output for transmission to the control center. Such a scheme requires expensive and elaborate facilities at outlying stations. This is contrary to the design criteria, and rules out the use of digital telemetering as a suitable means for wide area gathering of flow, level, pressure, and position data. This is not to say that its use will not be common when more practical digital transmitters are developed. Nor should this be interpreted to apply in other circumstances where a block of digital information is available at a site with suitable environment for transmission to another suitable location. It could be concluded that digital telemetering at least, would not be considered as a substitute or replacement for the time-impulse type.

Frequency-type telemetering is suitable as a substitute for pulse duration telemetering. However, because it is not available from as many sources as the pulse duration types, there is no particular price advantage, and because many jurisdictions have a large amount of pulse duration equipment in service, a case for its specific use would not seem arguable.

Pulse-count-type telemetering, from an electrical standpoint, also is suitable from the standpoint of design criteria. Its application is generally limited, however, to displacement flowmeters or counting systems which require a rotating body in the stream;

hence it is not suitable for waste water applications. Moreover, it is not as readily adaptable to time division or tone-type multiplexing which are simply techniques for grouping a number of transmitted signals on one communications circuit. Usually where a number of analog data functions are involved, the pulse count code would be converted to one of the other forms of telemetering for multiplexing.

Voltage-type telemetering only should be considered for on-site applications because of its distance limitations. It does have the advantage of having the data measurement in an electrical analog form which lends itself to direct analog-to-digital conversion.

Current-type telemetering should be considered only for on-site applications because of its distance limitations. It has the advantage of having a data signal of an electrical analog form suitable for direct analog-to-digital conversion. Furthermore, it is not so susceptible to electrical interference and usually would not require a shielded cable for its transmission circuit. Consideration should be given to the use of current-type telemetering for on-site applications. Not only is it adaptable to digital data, it has immunity to interference. The inherent requirement of only a simple ammeter for an indicating instrument make it desirable for compact panel arrangement. The present trend in electric or electronic instrumentation is toward the current and voltage types. The current type is most frequently used. Wherever the measurement data are required in digital form, it is first necessary to obtain it in either its current or voltage analog. For the accumulation and conversion of much data to digital form at any on location, current-type metering is quite appropriate.

Pulse duration telemetering holds a somewhat unique position for water and waste water measurement. Certainly none of the digital equipment presently available can supplant its existing utilization in water and waste water facilities. Most types of chemical feeders as standard equipment are presently designed to take pulse duration signals. Many pump controllers and most analog instrumentation are designed for use with pulse duration equipment. Not until the requirement for data in a digital form arises is the use of pulse duration telemetering seriously questioned.

Perhaps the most straight-forward means for converting pulse duration to digital form is the employment of a standard re-transmitting slide-wire-type potentiometer in a pulse duration receiver. Voltage or current in the potentiometer circuit could then be converted to a digital signal proportional to the position of the slide-wire. The use of a slide-wire, however, introduces another component requiring service and replacement. Therefore, it is desirable to obtain a current or voltage signal conversion from pulse duration without the requirement of a slide-wire.

Another possibility of converting the pulse duration signal to digital form is the employment of a shaft position to digital encoder in a pulse duration receiver. Although this is not available with present standard receiving equipment the possibilities of this concept should be more thoroughly investigated before commitment is made to acquire digital conversion equipment.

4.4 Communications

4.4.1 Communications Facilities

The most common communication link for telemetering and supervisory control in the waste water field is the leased telephone line. The monitoring of regulation practices, like that of monitoring in most other related industries, has used leased telephone lines for both their existing telemetering circuits and their supervisory control circuits. Generally, most experiences with the leased lines have been reasonably successful, particularly in cases where the importance of the communications link has been sufficiently impressed upon the telephone utility. Another means of communications which could be employed would be by microwave.

4.4.2 Microwave Facilities

Microwave, a form of directional—point-to-point—communications utilizing ultra high frequency equipment, provides a very large communications signal capacity. Usually, there is not a sufficient quantity of data signals at any given station on a waste system to justify the choice of microwave on that basis alone. Generally it is justified on point-to-point applications where the distances are quite great, and it is found that leased telephone service either is not available or that the service simply is unsatisfactory. Microwave equipment must be operated on a line-of-sight basis from transmitter to receiver, with intermediate repeater stations as required by the nature of the terrain. Generally, microwave is too expensive for the wide coverage necessary for control and supervision of the many remote stations which may make up a waste water system.

In cases where it is necessary to lease communications links from several intermediate telephone companies to form a complete circuit, it may be necessary to employ microwave equipment.

Microwave equipment is available from a number of suppliers, is reliable and flexible, but requires maintenance by an organizations's own personnel or by the service contract.

4.4.3 Telephone Lines

Leased telephone lines, as normally furnished by a telephone company, fall into three principal classifications. Other classifications have been recently established for very high speed data transmission. These classifications may vary in quality and performance from one place to another and between companies.

1. Class 1, or Class C—is a direct current slow speed telegraph circuit. Pulsing speeds are limited to 15 cycles per second. These circuits are generally continuous metallic with ground return. Such circuits are quite suitable for single pulse duration telemetering or individual control circuits and presently are used under appropriate conditions by many municipalities.
2. Class 2, or Class B—is also a direct current (DC) or low frequency (AC) teletypewriter or teleprinter circuit. Pulsing speeds are limited to a range generally of 60 to 100 pulses or cycles per second. Frequently, the Class 1 and Class 2 circuits are not available in residential areas. No particular need for this class of circuit is foreseen.
3. Class 3, or Class A—is a voice grade circuit. Generally, this class circuit may pass audio frequency (AC) signals up to 3000 cycles per second on short distance lines. Longer lines using repeaters usually pass audio tones from 300 to 3000 cycles per second. This type of line is most suitable for normal requirements in the subject field.
4. Although not normally required, other special classes used for high speed data transmission have been developed and can be furnished under special arrangements. Prior to consideration being given to the actual acquiring of any high capacity, high speed data handling equipment, the telephone company should be contacted regarding the type of equipment, the nature and characteristics of their communications circuits, availability, and cost.

4.5. Data Handling

4.5.1 General

Data is information that can be expressed symbolically—measurements, time, equipment status, identification, etc. Indicating and recording instruments, alarms, mimic busses, and lights all have been used for data handling. Data use will become more complicated as it is used with regulation control practices and techniques. At some point, automatic handling of data will become necessary. The system designer then must consider the use of computers for logging and control.

Data logging has generally been the first step toward data handling beyond the usual display of data on indicator scales or recorder charts, or lamp indications. The first models of data loggers usually used some means of receiver self-balancing potentiometer to position a shaft upon which a shaft position encoder was mounted to digitize the output of the receiver. Various inputs were switched to the receiver encoder combination by telephone-type stepping switches. The output of the encoder was then fed to an electric typewriter. With the exception of the encoder, the components—receivers, stepping switches, and electric typewriters—generally had been in use. Applications of such equipment have proved quite successful in logging a great amount of data in a short period of time, and where there was adequate time between uses for service and maintenance. Utilities requiring trouble free, day-in, day-out logging service, however, found such equipment to be in frequent need of service and repair. These electro-mechanical-type data loggers are not recommended for the continuous service required for waste water regulating practices.

A second generation of data loggers has been developed, using solid state voltage or current generators in place of the self-balancing potentiometers, digital voltmeters to digitize in place of shaft position encoders, and reed or mercury relay type scanning programmers in place of stepping switches. Such instruments greatly reduced some of the problems encountered with the earlier loggers and permit higher speed and more flexibility in setting alarm points or set-point control.

Either first or second generation loggers are basically prewired systems. The second is easier to modify because of its use of pin boards for convenient changing of alarm, span, and zero suppression settings. In either case the output of the logger is basically a fixed log only, with no convenient means of data manipulation such as might be required if the data are to be used in conjunction with a computer for trend alarms, computational analysis, averaging, or other programmed requirements.

The most recent data logger is the general purpose, stored-program digital computer which is used as the central data processer of a modern data handling system. Once the digital computer is put into the system, it will do the jobs which were difficult or impossible to do with the original logger. It will also do anything the second generation loggers could do, and store data, permitting the data to be used in whatever manner a programmer may call for, including closed loop computer control.

4.5.2 Digital Computer Capacity

The data handling digital computer can have the following capabilities:

1. Computer memory can store alarm settings, and zero and suppression settings, thus eliminating fixed wire pin boards or separate instrument alarm contacts.
2. Subroutine programs can eliminate the clock and calender required in the earlier type loggers.
3. Computer can remember alarm points during the last scan and initiate and display programmed instructions when an alarm is found.
4. Linearizing of signals can be done by computer programming.
5. Pulse inputs can be counted directly by the computer, eliminating the individual pulse counters.
6. Time duration inputs can be read directly by the computer by means of program interruptions.
7. Analog-to-digital converters can become a part of the computer.
8. The computer can measure zero drift on low level signals and compensate for it.
9. The computer can sense an excessive rate of change of variable, to give trend alarms prior to reaching the point of alarm condition.
10. Simple to complex computations can be performed on the data received.
11. Memory of the computer permits an operator to recall data prior to an alarm for examination. For example, in the event of an alarm, the data handling system can automatically or on demand print out the values of significant variables for say every ten seconds of the five minutes preceding and following the time of alarm.
12. A trend recorder can plot data from storage. For example, in only a few seconds a 24-hour recording of an input such as a flow, pressure, or temperature can be made by simply selecting the variable to be analyzed.
13. The computer can accept new commands from the operator.
14. The complete logging program may be revised or modified without making any external changes.
15. The digital computer-type data handling system can be adapted to closed loop computer control.

The greatest advantage of the digital computer logger is that once it is put into the system, it can be programmed to do additional work unforseen at the time of its original installation, at almost no additional cost.

The data handling facilities must have the capacity for handling a vast amount of data, yet be capable of discriminating between that which is normal and that which must be called to an operator's attention. Data requirements for the future can not be determined with accuracy. Thus a hard-wired electro-mechanical logging system should not be purchased. Hard-wired systems become more and more complex and inflexible as the number of points of data increase. The computer-type logger, on the other hand, can originally be purchased with a great amount of data handling capability with characteristics and execution methods to be changed as the input data amount increases.

Concentration of effort on the acquisition and accumulation of suitable data must precede installation of a system.

4.6. Decision Making

When measuring devices, telemetering, communications, data handling, supervisory control and outlying station activation facilities have been acquired, only the decision and execution need be performed to close the regulating control loop. How well the system is regulated then becomes a matter of interpreting the data and judging the amount of control to be executed. Deciding what to do may be an extremely flexible arrangement wherein the operations control center operator or attendant uses his best judgment: Or it may be a fixed and inflexible arrangement determined ahead of time and incorporated in a programmed operation automatically performed by the data handling equipment or computer. Arguments can be made for both arrangements.

A capable attendant can permit a wide range of allowable conditions under various circumstances, not necessarily defined by only the measured inputs; whereas, with the computer, more nearly identical controlled conditions can be depended upon during similar situations.

Closing the control loop manually is known as off-line control. Closing the control loop automatically is known as on-line and real-time control.

A system which includes data logging, data handling and supervisory control equipment of the recent and more sophisticated types, can, with slight modification function with some on-line control. Any system with the capabilities of on-line control must be capable of switching to off-line control when desired.

4.7 Supervisory Control

4.7.1 General

For management to exercise fullest authority over the regulating control facilities, it is desirable to have information regarding the effectiveness of a regulator, and means to override or put into effect additional controls. Control is a result of some decision-making process and may be performed manually or automatically.

Supervisory control equipment consists of three basic elements: (1) The dispatcher's equipment; (2) the means of communication; and (3) outlying station equipment. Systems will vary from the simplest—a control switch, directly connected to a regulating gate operator—to a sophisticated regulating system operated by a computer on closed loop control. The methods and techniques used to transmit information are similar to those used for remote control. A computer handles not only data, but also generates control output, either manually or automatically initiated. It is necessary to recognize the essential differences between conventional supervisory and computer control. Because supervisory control and computer control are such broad, generic terms these are defined for the purpose of this Manual.

Conventional Supervisory Control

A custom-designed group of selector switches, push-buttons, indicators and electronic circuits connected by a wiring harness and packaged in completely integrated assemblies for installation at the central and remote stations. These stations are specifically designed and wired for the specific application. The operation and performance is fixed by the wiring, which has led to the use of the term "hardwired" control.

Computer Control

A digital computer with standard logic, memory, and wiring, installed at a central station. The customizing for a particular application is done by a list of instructions and programming, which are magnetically stored in the machine. This electronics programming gives rise to the term "softwired" type for its particular application.

Either of these two classes of electronic control systems allows the operation of remote equipment from the point of central control. The central station may have the data logging, data display and other data handling options. Both require essentially the same communication means. An operator at a control center could not generally tell whether controls are handled by a softwired computer or a hardwired conventional supervisory control system. There are a number of ways in which these two classes of equipment may be implemented. Four general techniques to consider are:

1. Conventional Supervisory;
2. Computer On-Line Control;
3. Conventional Supervisory with Off-Line Computer Monitoring; and
4. Computer On-Line with Conventional Supervisory Standby.

Hardwired conventional supervisory control equipment is a type that many operators have used for years. It has the advantage of having a low initial cost and central control may be implemented in step-by-step stages of development.

Softwired digital computer control can not only take over and perform all of the functions, but can also accomplish additional tasks beyond the ability of the conventional supervisory control.

The digital computer has no prewired control capability. Therefore a program must be written to customize the computer to a particular application before it is used in a control system. Programming the computer for control of remote stations requires special skills. A combination of waste water practice experience and computer application technique is required. Conceivably, the programming expense could exceed the cost of the conventional supervisory control system.

The value of the computer system lies in the additional benefits that fall into two general areas: (1) The flexibility of modifying and expanding the control system; and (2) the capability for data processing.

The value to the waste water management of the flexibility of modification and system expansion alone are not of sufficient importance to influence the selection of a computer. The control system for any specific remote station will normally remain static and unchanged after the initial requirements have been determined and successfully established, except for remote station expansion which could generally be taken into consideration during the initial implementation of the equipment.

4.7.2 Advantages of Data Processing

The capability for data processing, can be a most important benefit. These benefits alone may justify the higher cost and problems of program writing. Some of the benefits to be realized by data processing could be as follows:

1. Programmed data logging at periodic intervals;
2. Selective data logging, for example high repetition log on a particular variable for engineering study;
3. Computation and logging of quantities for selected time intervals;
4. Computation and logging of the summation of flows;
5. Computation and logging of inventory storage facilities;
6. Rate of change alarms on flows, levels, pressures, etc;
7. Deviation checking for off-normal conditions with alarms;
8. Computation and logging of equipment, operative status and running time;
9. Programmed or selected display of variables;
10. Programmed printout of instructions to operators;
11. Computation of load and system studies;
12. Storage of information and computation for billing purposes;
13. Preparation of data for storage in the form of punched card, punched paper tape, or magnetic tape;
14. Economic computations for operation guidance; and
15. Capability of actually performing as an element of the control scheme; that is, closing a control loop by performing its own control action.

Although the initial programming may be a relatively costly item, it should be a non-recurrent expense, and updating and changing to conform with system growth or altered data manipulation should be accomplished inexpensively.

4.7.3 Disadvantages of Data Processing

The disadvantages of each system are:

1. The conventional supervisory equipment:
   a. Must be customized for each particular application;
   b. Requires rewiring to change control patterns;
   c. Has limited capability of data handling;
   d. Lacks flexibility; and
   e. Maintenance costs may be high due to large number of components.
2. The digital computer control equipment:
   a. Costs more initially;
   b. Requires costly programming;
   c. Compounds the maintenance and service problem by requiring technical know-how in two additional technologies, one, the electronics of the computer and another, programming techniques, in addition to the electronics of the fixed wired conventional systems used at the remote stations.

4.7.4 Combining the Advantages of Supervisory Control and Digital Computer Control

In comparing the two techniques, the conventional supervisory control and the digital computer control, it is easily concluded that there would be considerable merit in initiating a construction program in which the advantages of both could be realized. This necessitates the use of both classes of equipment in either of two arrangements:

1. The conventional hardwired supervisory for remote control supplemented with a computer for monitoring and data handling;
2. The computer for on-line control and data handling, supplemented with the conventional supervisory equipment as standby for use when the computer is in down-time.

This second arrangement could be an outgrowth of a successful experience with the first. In this approach the hardwired conventional supervisory control would be purchased and installed in increments amenable to any overall program for converting the outlying station from manned to unmanned remotely controlled facilities. As the control system is expanded so are the computer facilities for monitoring and data handling, together or separately as the conditions and circumstances govern.

Advantages of the on-line availability of the supervisory system, together with the data handling capability of the computer, may then be realized in a gradual manner without an initial full commitment to either. This arrangement also has the advantage of allowing the computer to be taken off-line for program maintenance, problem solving or other uses for which the computer can be applied without disrupting the control of the remote stations.

The operations control center attendant would initially generate all control function messages to remote stations by manipulation of switches or push-buttons at his dispatcher's panel or console. The computer could monitor all messages going out to remote stations and all remote equipment status and remove measurement data being received from remote stations. The computer could therefore "listen-in" and be instructed to interrogate, check for limits and off-normal conditions, process and log the measurement data and cause whatever printouts including alarming and instructions the programming diagnostics would provide. Ultimately the computer might be programmed and interfaced with the supervisory equipment to exercise a certain amount of logical and routine control. This latter degree of sophistication would be approaching the case of computer on-line control with the manual supervisory as standby.

For the management of sewerage systems, it seems appropriate to place emphasis upon the long-range establishment of an operations control center. The plan of development should not require, however, the initial investment for the procurement of supervisory or data handling capacity for functions that are not expected to be required for several years nor for those partially known or unknown requirements of the future.

System growth should not be projected too far into the future. Changes, not only in the design and technique of control and data handling but also of the system affect future requirements. Rather than forecasting and perhaps freezing the requirements, it is better to arrange for considerable flexibility and the possibility of gradual expansion.

Consoles and panels, for expample, should be procured for only those control and metering functions initially required without panel cutouts or blanked panel space for the indefinite future. Console and panel arrangements should be such that additional sections or modules can be procured and installed when required. Nor should the initial equipment be designed to preclude the feasibility of replacement or rearrangement of certain console or panel sections or operational groups to meet future changes of the controlled system or even the layout and arrangement of the operations control center.

If the ultimate needs for the control center were known, it would seem most logical to choose one class of equipment designed for the combined requirements of control and data handling. This is not feasible. Plans should be prepared for the gradual implementation of both the conventional supervisory control equipment for remote control and the digital computer for monitoring and data processing.

4.8 Activation Equipment

Activation equipment refers to the power operators for regulator devices. Self-powered regulators such as floats and hydraulic cylinders, have been described in 2.7, 2.9, 2.10 and 2.11.

Electric motor operating regulating devices have been quite successfully employed on open-close type regulators and have been arranged for inching controls for intermediate positioning. Intermediate positioning control schemes have been arranged by using a series of limit switches and timers. In addition to the problems encountered with electrical equipment, the problem of loss of power is always of concern since this is most likely to occur during a storm which is precisely the period during which the regulator is most likely to function.

Recent development of electric motor operators for modulating or throttling control of valves, bring to the user a new and more appropriate drive unit for regulators. These drive units use low voltage, direct current motors with direct current power derived from silicon controlled rectifiers powered from the standard alternating current power sources. Small direct current batteries, similar to car or truck batteries, may now be used as standby power for regulator operation during power failures. These new motor designs can be controlled directly by presently available electronic instruments.

In the past, reversing contactors were required for throttling control of the standard motor. It was the reversing contactor, not the motor, that could not withstand continued reversing duty. The new motor designs eliminate the reversing contactor and continuous regulator control is possible. Gates, for example, can be controlled automatically to maintain a certain water level; they can be made to "hunt" in direct or indirect relation to the rise and fall of water level within a given band width; or they may be remotely positioned at any point within their full travel by a remote set point signal from a central control station. Gate position and water level, can be telemetered to the control station over the same control line. New motor operators and electronic instruments are small, allowing them to be placed in enclosures which protect them from exposure to adverse conditions. This new equipment, after it has been placed in service should prove to be of considerably less trouble and prove, to be less expensive to maintain. However, its initial cost will be more than the cost of conventional equipment.

An example of a design problem using total system control is presented in Section 7.

SECTION 5

PRACTICES FOR IMPROVED OPERATION AND MAINTENANCE OF REGULATORS AND THEIR APPURTENANCES

CONTENTS

5.1 General

Satisfactory operation of combined sewer overflow regulator facilities depends to a large extent on adequate, regular inspection and maintenance. The purpose of this is twofold: First, to locate and correct any operational failures; and second, to prevent or reduce the probability of such failures.

5.2 Common Causes of Failure

Some of the factors causing failure as related to regulator types are as follows:

1. Static regulators
   a. Clogging
   b. Silting
2. Dynamic-semi-automatic
   a. Clogging
   b. Silting
   c. Sticking (lack of lubrication)
   d. Parts failure
   e. Corrosion
3. Dynamic-automatic
   a. Clogging
   b. Sticking (lack of lubrication)
   c. Parts failure
   d. Power failure
   e. Water pressure failure in float-actuated, hydraulically operated units

Of these factors, clogging is the principal offender. It affects all types of regulators to some extent and is practically impossible to eliminate entirely. Silting affects most regulators, but it can be controlled by regular flushing of the regulator station by the maintenance crew. The other factors apply to mechanical, electrical, or hydraulic types of regulators and their effects can be eliminated or minimized by proper inspection and maintenance.

5.3 Frequency Of Inspection Required

The susceptibility to clogging, to an important degree, determines the required frequency of regulator inspection. Several things affect this susceptibility. They include the type and size of regulator, the size of the combined sewer, the size of the connection to the interceptor, and the quantity and quality of the combined sewage.

Experience has shown that regulators of the orifice type, or which depend on an orifice for operation, particularly the horizontal orifice and leaping weir, readily clog, especially when the orifice is small.

Horizontal orifices, or drop inlets, protected by grates appear to be more subject to frequent clogging than any other type of regulator. Even daily inspection and cleaning may not be adequate to insure proper operation of this type of regulator.

Practice in the jurisdictions included in the National Investigation varies from daily or more frequent inspections to as few as three per year. The average number of inspections is approximately 70 per year.

Inspection must be as frequent as required to keep the regulators in as continuously operable condition as practicable. In general this will require an inspection schedule of at least each week and after every storm. Small orifices and drop inlets with grates will require more frequent inspection. It is recommended, however, that no regulator be inspected less frequently than twice per month and after each storm.

Experience will indicate which regulators require more frequent attention than others. The schedule should be adjusted to meet local or changing conditions. In this way maximum efficiency of operation will be achieved with minimum use of personnel.

5.4 Recommended Maintenance Program

The regulator chamber should be cleaned after every storm and more frequently if necessary to maintain satisfactory working conditions in the chamber.

For all types of regulators, each visit should include a visual inspection of the regulator and removal of any debris preventing or tending to prevent its operation. Preventive maintenance programs recommended for the various types of regulators are as follows:

1. Static Regulators
   a. Orifices

      Orifices clog frequently. Maintenance equipment should include hooks, sewer rods, and scoops, so that as much clearing of debris as possible may be effected from the ground surface. The regulator chamber may then be flushed out, also from ground surface.

   b. Drop Inlets and Leaping Weirs

      Grates protecting drop inlets should be checked to be sure they have not been damaged or weakened by corrosion. Inlets, particularly large ones without gratings should be fitted with grates or guard rails for the protection of the maintenance crew, as well as prevention of the entrance of large objects. Where blockages are excessive the gratings should be replaced by ones with larger openings. Leaping weir plates must be lubricated and adjusted semi-annually to prevent "freezing."

c. Side-Spill Weirs

Weir crests should be inspected for damage and be repaired promptly.

d. Manually-Operated Gates

Manually operated gates should be operated on a regular basis through the full range from the open to closed position and reset at the proper opening. The floor stand, if any, operating stem and guides should be freed of corrosion and be well lubricated.

2. Dynamic Regulators

a. Float-Operated Gates

If possible float-operated gates should be operated through a complete cycle. Float wells should be cleared of deposits of sand or sludge and all accumulations of debris should be removed from the float and float well. Chains and gears should be cleaned of rust and other deposits and thoroughly lubricated. All parts of the mechanism should be examined for wear or corrosion and, if necessary, promptly replaced or repaired.

b. Tipping Gates

Tipping gates should be checked to be sure they move freely on the pivot shaft. Adequate lubrication of the bearings is essential. In some cases it may be advisable to replace existing shafts and bearings with stainless steel shafts and bronze bearings.

c. Motor-Operated Gates

Motor-operated gates should be operated through the full range from open to closed position and reset at the proper opening. Where remote control is provided, the cycle should be run through, using the remote controls while the maintenance crew observes the operation and checks the final setting of the gate. Water level indicating and transmitting equipment should be checked to insure that it is functioning properly and measuring water levels accurately. All equipment should be inspected for signs of wear or corrosion, repaired or replaced, if necessary, and properly lubricated.

d. Cylinder-Operated Gates

If possible, cylinder-operated gates should be operated through a complete cycle. Float wells should be cleared of deposits and all accumulations of material should be removed from the float and float well. Strainers on the water supply line should be cleaned and inspected. Water supply lines, valves and cylinders should be inspected for leakage; and the pressure available at the cylinder under all operating conditions should be checked. Water level sensing equipment on hydraulically-oil-operated gates should be checked to insure that it is functioning properly and accurately. All equipment should be inspected for signs of wear or corrosion, repaired or replaced, if necessary, and properly lubricated. Cross-connection hazards should be detected and corrected to prevent pollution of public water supplies.

5.5 Personnel Requirements

Maintenance of regulators should be carried out by crews of three to five men, depending on the type and complexity of the regulators used. A minimum crew of three men is recommended in order that one man may remain on the surface while two men enter the chamber.

For simple regulators, three-man crews with one foreman to direct several crews should be satisfactory. Where remote controls, water level sensing devices and motor-operated gates are used, a crew of five men, including a technician and foreman may be required.

The number of crews required will depend on the number and types of regulators and the frequency with which they must be inspected.

5.6 Equipment Needs

Adequate equipment should be provided for the safety and efficiency of activities of the maintenance crew. The following items of equipment are considered necessary:

a. A 1½-ton panel truck with a two-way radio, winch and A frame

b. 110-220 volt portable generator

c. 1 and 2-hp submersible pumps

d. One 1½-hp blower unit

e. Various chains, ropes, hoses, ladders, pike poles, sewer hooks, sewer rods, chain jacks, tool kits, etc.

f. One oxygen deficiency meter, one explosimeter, and one H_2 detection meter, safety equipment, helmets, harnesses, first aid kits, danger flags, signs, barricades, life jackets, flares, gas masks or air packs, gas detector lamps, fire extinguishers, extension cords, rubber jackets, pants, boots, waders, etc.

g. Spare parts.

5.7 Safety Precautions

Precautions necessary to protect the maintenance crews from the hazards in the sewers and regulator chambers and from traffic are relatively uniform throughout the country. The following instructions, issued to the crews in Philadelphia, are typical:

1. Truck should be parked so as not to obstruct traffic but, if possible, should be used to protect men working near the open manhole. If truck is used for this purpose, suitable flashing lights must be used on the truck.
2. Warning cones, flags, signs, and lights should be used to make areas safe for both vehicle and pedestrian.
3. Manhole cover should be raised with a safe tool and bar placed under it so that it can be rolled to one side.
4. A manhole guard should be placed around the open manhole.
5. The air in both sewer and chamber should be checked for explosive mixtures and hydrogen sulfide. Oxygen deficiency should be checked.
7. If there is an indication of gases, the portable blower should be used to clear the area.

In addition to the above safety precautions it is considered essential that at least one man remain at the surface at all times to summon or render assistance in the event of an accident.

5.8 Maintenance Costs

The cost of maintaining sewer regulators, as reported in the National Survey, varies widely. In most cases the reported expenditures are probably not adequate to maintain the regulators in completely satisfactory condition. The annual cost per regulator required to conduct a satisfactory maintenance program is estimated to be as follows:

Description	Annual Cost per Regulator*
Vertical orifice or siphon	$ 600 – 800
Leaping weir	700 – 900
Drop inlet	1200 –1500
Side-spill weir	400 – 500
Manually operated gate	900 –1100
Float-operated gate	1100 –1200
Tipping gate	1100 –1300
Cylinder operated gate	1200 –1300

* From Report, Section 5

5.9 Records and Data Analysis

Complete records should be kept of all inspection and maintenance work. The time and date of each inspection should be recorded together with a description of the condition of the regulator and the work performed. The number of man-hours spent at each regulator should be noted.

The data obtained should be tabulated for each regulator and summarized for each type of regulator. These records will quickly reveal any regulators which require excessive maintenance or are out of operation with unusual frequency. The causes should then be investigated and, if possible, corrected.

The records also will provide the data needed to compare the cost and efficiency of different types of regulators for guidance in the design of new regulators or the remodeling of existing regulators.

SECTION 6

DESIGN AND LAYOUT, AS INFLUENCED BY OPERATION AND MAINTENANCE – TYPICAL CRITERIA AND DETAILS WITH RESPECT TO OPERATIONS AND MAINTENANCE

CONTENTS

6.1 Typical Layout Criteria

6.1.1 General

The layout of a regulator station should meet two criteria: The first, based on the hydraulic requirements; and the second, based on the operation and maintenance requirements. Very often the designer gives little attention to operating and maintenance requirements, with the result that the regulator fails to function properly due to lack of adequate maintenance. Hydraulic design principles have been outlined in Section 2. This Section considers the layout from the viewpoint of operation and maintenance.

6.1.2 Location

The designer may have little choice with regard to the location of the regulator. Regulators often must be located on existing combined sewers. Where the area is highly developed and there is no choice except to place the regulator station within the street right-of-way. If the street is a cul-de-sac and terminates at the water's edge it may be possible to fence off the end of the street for the regulator site. Where areas are partially developed, it may be possible to relocate the combined sewer so as to place the regulator on a site other than the traffic right-of-way. For maintenance purposes and for operator safety this is most desirable. If this is not possible, access to the facility should preferably be from the rear of the curb in order not to block traffic while routine maintenance is being performed.

6.1.3 Access

Separate access should be provided to each chamber of the regulator station. This is necessary when the chambers are small or the maximum level of the sewage in the chamber is near the ground surface.

In large chambers where the depth permits, access to the diversion and tide gate chambers may be combined.

Access to regulators located in streets is made through conventional manhole shafts with a vertical ladder or with manhole steps set 12 inches on centers. If the regulator is deep, stairs should be used at a reasonable level below the ground surface.

When the regulator is located off the street, consideration should be given to alternate means of access rather than the standard manhole. If there is objection to a structure extending above ground, access can be provided by a floor door or hatch and a ship's ladder. A ship's ladder is a fixed inclined ladder with an angle to the horizontal of beteen 40 and 56 degrees. The minimum width of tread between stringers should be two feet. Ladders steeper than 56 degrees should not be used.

Where there is no objection to a superstructure, stairs can be provided. This increases the length of superstructure opening to 11 feet and requires a superstructure approximately 13 feet long, 5 feet wide, and 8 feet high. Details of such a stairwell are shown in Fig. 6.1.3. If a superstructure is required for electrical equipment the stairwell superstructure may be built adjacent to its exterior wall.

Spiral stairs should be considered where space is not available for standard stairs. These are constructed in various diameters, from 4 feet, considered a minimum for ease of access, to 6 feet or more depending on the requirement. They are usually constructed with either 12 or 16 treads to a complete circle. On a 12-tread circle, 9-inch risers will provide 6 feet 9 inches of headroom, and on a 16-tread circle 7-inch risers will produce 7 feet of headroom when calculated on the basis that three-fourths of the vertical height of a full circle is required for free passage.

6.1.4 Light, Heat and Ventilation

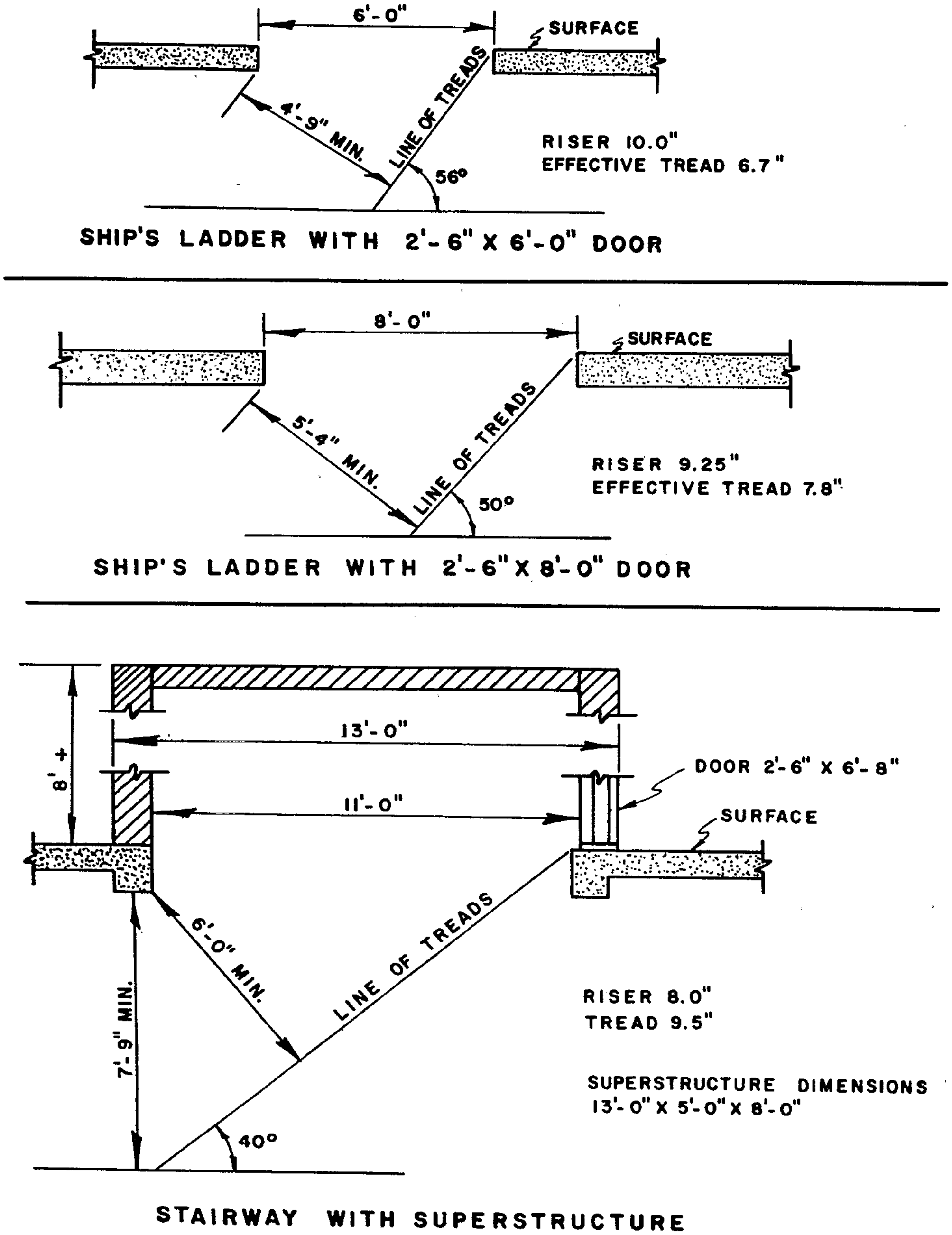
As a rule, insufficient consideration is given to light, heat and ventilation in regulator chambers of average size, particularly when no electric power is required for operation of the regulating device.

Portable lamps are usually used for lighting purposes. When the chambers are large, at least two manhole openings should be provided for each chamber for adequate light and ventilation. If the regulator is located below the point of access and is properly fenced, roof gratings can be used to increase light and ventilation. Whenever electric power is available at the regulator chamber, provision for electrical lighting in all parts of the regulator station is normal.

Heating is not considered necessary in underground chambers for the comfort of personnel. However, when dehumidifiers are used to prevent corrosion of electrical equipment, heating is sometimes provided to prevent freezing of the dehumidifiers and to reduce the relative humidity. Superstructures, if used, should be heated.

Shallow regulator stations using static regulator devices requiring little attention can be adequately ventilated by use of portable blowers. Such chambers should be tested for flammable gas or oxygen deficiency before entering. In deep chambers tests also should be made for the presence of carbon monoxide and hydrogen sulfide. Where mechanical equipment is used consideration should be given to the installation of ventilating equipment. Such ventilating equipment is considered essential for wet wells in sewage pumping stations. Present

FIGURE 6.1.3



ACCESS STAIRS

recommendations for such use by health authorities ("Recommended Standards for Sewage Works." Great Lakes-Upper Mississippi River Board of State Sanitary Engineers, 1968) include: (1) Provide at least 30 complete air changes per hour for intermittent ventilation; (2) interconnect intermittently operated ventilating equipment with the lighting system; and (3) introduce fresh air into the wet well by mechanical means. The recommendations for pumping station wet wells, noted above, should be equally applicable to regulator chambers containing mechanical equipment and requiring frequent inspection.

6.1.5 General Features

The designer should visualize all possible activities of maintenance personnel and attempt to provide adequate and convenient facilities in which they can perform their duties. Walks or platforms, preferably above the maximum sewage level, should be provided so that all parts of the regulator station can be reached and so that the inlet and outlet sewers can be observed and are readily accessible. Benches for access landings should have a minimum width of 1½ feet. Headroom should be at least 6½ feet.

Guard rails should be provided around all openings or sudden drops. However, officials of one city interviewed in the National investigation stated that they generally do not use protective railings in the regulator chambers since the staff feels that reliance on a railing which may fail due to corrosion is more hazardous than the omission of the railing. However, this city uses railings of structural steel, encased in concrete, to prevent failure due to corrosion at regulator stations that are particularly hazardous and where a fall might mean death or serious injury.

Cast iron stop plank guides should be provided to shut off or divert flow to, or from a channel when required for maintenance purposes.

6.1.6 Gates

The sluice sizes available from one manufacturer are shown in Table 6.1.6. Normally the smallest size used in a regulator chamber should be 12 by 12 inches.

When a cylinder-operated gate is activated by water pressure the gate size is generally limited to a maximum of nine square feet. The size of gate when oil pressure is used is not limited.

A shaft and ground surface opening should be provided over each gate. The dimensions of this opening should be at least one foot greater in each direction than the overall dimensions of the gate, to allow quick, problem-free operation or removal.

6.1.7 Float Well

The float well should have minimum dimensions of three feet square. The bottom of the well is set above the invert of the adjacent channel and connected thereto with a 12-inch wide passage or telltale. The telltale should be aimed downstream from the float well to prevent floating solids from entering the well or plugging the telltale.

6.2 Materials

6.2.1 Metals

The best of the bronzes for corrosion resistance and strength is silicon bronze. This is a very high copper, zinc-free bronze. Manganese bronze castings and extrusions wear well. They are used for valve seats and operator stem nuts for this reason. Among the stainless steels, the 18-8 chromium-nickel content percentage respectively endures best. Types 303, 304, and 305 are used for valve stems, studs, nuts and bolts. Type 326 stainless steel is especially good in sea water and less costly than monel which also gives excellent service in salt water. Heavy body castings are usually grey iron castings, conforming with ASTM specification A126, Class B. However, in highly corrosive applications, Ni-Resist Type 1A has been used successfully. This is a trade name of International Nickel Company for an iron-casting with the following percentage composition: 14% nickel, 6% copper, 2½% chromium. Both cast iron and wrought iron are customarily coated with a hot coal-tar enamel in accordance with AWWA Specification C203. Bronze, stainless steel and monel are not usually coated.

Sluice gates generally should conform to AWWA Standard C501. This standard also covers sluice gate operating mechanisms of the manual, electric-motor and hydraulic-cylinder types. The latter type is oil operated at 2000 psi working pressure. The Suma Standard, Sect. 103, states that the purchaser may specify other operating media or pressures, as desired. One manufacturer has suggested four combinations of materials for sluice gates for use under various conditions. These combinations are shown in Table 6.2. Combination No. 1 which meets AWWA Standard C501 is not considered satisfactory for use in raw sewage service because it contains naval bronze. De-zincification, where zinc is dissolved from bronze leaving a weak, porous material, can occur with naval bronze in contact with either acid or alkali fluids. The low zinc content of phosphor or silicon bronze enables these alloys to resist de-zincification. Material Combination No. 3 would provide

TABLE 6.1.6

SLUICE GATE SIZES

Size of Gate Inches		Area of Clear Opening Square Feet	Size of Gate Inches		Area of Clear Opening Square Feet	Size of Gate Inches		Area of Clear Opening Square Feet
Rectangular Width X Height	Circular Diameter		Rectangular Width X Height	Circular Diameter		Rectangular Width X Height	Circular Diameter	
	6	.1964	36 x 18		4.50	66 x 72		33.00
6 x 6		.2500	36 x 24		6.00		72	28.27
	8	.3491	36 x 28		7.00	72 x 42		21.00
8 x 8		.4444	36 x 30		7.50	72 x 48		24.00
	10	.5454	36 x 36		9.00	72 x 60		30.00
10 x 10		.6944	36 x 42		10.50	72 x 72		36.00
	12	.7855	36 x 48		12.00	72 x 84		42.00
12 x 12		1.000	36 x 60		15.00	72 x 96		48.00
12 x 18		1.500	36 x 72		18.00		78	33.18
12 x 20		1.667	36 x 84		21.00	78 x 78		42.25
12 x 24		2.000		42	9.62	78 x 96		52.00
	14	1.069	42 x 24		7.00		84	38.48
14 x 14		1.361	42 x 27		7.88	84 x 36		21.00
	15	1.227	42 x 30		8.75	84 x 60		35.00
15 x 15		1.562	42 x 36		10.50	84 x 72		42.00
	16	1.396	42 x 42		12.25	84 x 84		49.00
16 x 16		1.778	42 x 48		14.00	84 x 96		56.00
16 x 24		2.667	42 x 60		17.50	84 x 108		63.00
	18	1.767	42 x 72		21.00		88	42.20
18 x 12		1.500		48	12.57		90	44.18
18 x 18		2.250	48 x 27		9.00		96	50.26
18 x 24		3.000	48 x 30		10.00	96 x 48		32.00
18 x 36		4.500	48 x 36		12.00	96 x 60		40.00
	20	2.182	48 x 48		16.00	96 x 72		48.00
20 x 20		2.778	48 x 54		18.00	96 x 84		56.00
20 x 24		3.334	48 x 60		20.00	96 x 96		64.00
20 x 36		5.001	48 x 72		24.00	92 x 120		80.00
	21	2.405	48 x 84		28.00	96 x 144		96.00
	24	3.142	48 x 96		32.00		102	56.75
24 x 12		2.000	48 x 108		36.00		108	63.62
24 x 18		3.000		54	15.90	108 x 48		36.00
24 x 24		4.000	54 x 48		18.00	108 x 60		45.00
24 x 30		5.000	54 x 54		20.25	108 x 108		81.00
24 x 36		6.000	54 x 72		27.00		120	78.55
24 x 48		8.000		60	19.64	120 x 42		35.00
24 x 60		10.000	60 x 36		15.00	120 x 72		60.00
	30	4.909	60 x 48		20.00	120 x 84		70.00
30 x 24		5.000	60 x 60		25.00	120 x 96		80.00
30 x 30		6.250	60 x 72		30.00	120 x 120		100.00
30 x 36		7.500	60 x 84		35.00	120 x 190		158.33
30 x 48		10.000	60 x 96		40.00	144 x 48		48.00
30 x 60		12.500		66	23.76	144 x 144		144.00
	36	7.068	66 x 66		30.25			

Courtesy Rodney Hunt Co.

TABLE 6.2

SLUICE GATE MATERIALS

Material Specifications:

CAST IRON	A 126, Class B or C
AUSTENITIC GRAY IRON CASTING (Ni-Resist)	A 436, Type 2 or 2b
STAINLESS STEEL (Seating Faces and Anchors)	A 276, Type 302 or 304
MONEL (Faces and Fasteners)	B 164, Class A or B
MANGANESE BRONZE (Lift Nuts and Wedges)	B 147, Alloy 8A
NAVAL BRONZE (Faces and Stems)	B 21, Alloy B
PHOSPHOR BRONZE (Faces)	B 139, Alloy A
SILICON BRONZE (Fasteners)	B 98, Alloy A, B or D
SILICON BRONZE (Castings)	B 198, Alloy 12A
STAINLESS STEEL (Fasteners)	A 320, Grades B8 or B8F (Bolts) A 194, Grades 8 or 8F (Nuts)
STAINLESS STEEL (Stems and Anchors)	A 582, Type 303

Material Combinations:

Gate Part or Item of Assembly	#1 Material Combination	#2 Material Combination	#3 Material Combination	#4 Material Combination
WALL THIMBLE	Cast Iron	Cast Iron	Cast Iron	Ni-Resist
GATE ASSEMBLY				
FRAME AND SLIDE	Cast Iron	Cast Iron	Cast Iron	Ni-Resist
SEATING FACES	Naval Bronze	Stainless Steel	Phosphor Bronze	Monel
YOKE (NRS ONLY)	Cast Iron	Cast Iron	Cast Iron	Ni-Resist
SIDE WEDGE BLOCKS	Cast Iron	Cast Iron	Cast Iron	Ni-Resist
CONTACT FACES	Naval Bronze	Stainless Steel	Phosphor Bronze	Monel
SIDE WEDGES	Manganese Bronze	Stainless Steel	Silicon Bronze	Monel
TOP AND BOTTOM WEDGES	Manganese Bronze	Stainless Steel	Silicon Bronze	Monel
FASTENERS	Silicon Bronze	Stainless Steel	Silicon Bronze	Monel
FLUSH BOTTOM ASSEMBLY				
STOP PLATE AND RETAINER	Cast Iron	Cast Iron	Cast Iron	Ni-Resist
STEM ASSEMBLY				
STEM	Stainless Steel	Stainless Steel	Stainless Steel	Monel
STEM BLOCK	Manganese Bronze	Ni-Resist	Silicon Bronze	Ni-Resist
STEM SPLICE	Stainless Steel	Stainless Steel	Stainless Steel	Monel

compatible materials for this condition. Combination No. 2 would also satisfy this condition.

Ammonia-bearing fluids such as raw sewage may cause stress-corrosion cracking to which copper based alloys are extremely susceptible. Since the 300 Series of stainless steel is not affected by stress corrosion cracking, the No. 2 Material Combination replaces with stainless steel all bronze gate parts subject to high stress. Combination No. 4 is intended only for salt or brackish water installations and combines the most corrosion-resistant materials.

Piping for air, water or oil should be corrosion-resistant. A suitable pipe for this purpose is seamless red brass pipe meeting ASTM specification B43. Suitable fittings for this pipe are copper-base alloys meeting ASTM specification B30, Alloy 4B.

Manhole steps usually are made of cast iron or aluminum. Vertical ladders usually are made of galvanized steel or aluminum. Ship's ladders can be fabricated with either galvanized steel or aluminum stringers and either cast iron or cast aluminum abrasive treads. Spiral stairs also can be obtained with cast iron or cast aluminum treads. Normally the center column is a 3 to 4-inch-diameter steel pipe; for regulator use the center column should be stainless steel. Aluminum used in the foregoing is usually specified to be 6061-T6, 6063-T5, 6063-T6 and 6063-T832.

6.2.3 Elastomers and Gasket Materials

The most commonly used elastomer is Neoprene. Neoprene is a copolymer of butadiene and acrylic nitrile. It has good resistance to hydrocarbons and ozone and resists air-hardening. Nitrile and a blend of nitrile and polyvinyl chloride also have good resistance to the sewage atmosphere. Natural rubber deteriorates in sewerage applications and is not recommended. Gaskets and packing should be made of asbestos, teflon coated asbestos or tallow lubricated flax.

6.2.4 Electrical

Motors located in underground chambers or in above-ground chambers into which sewer atmosphere may escape should be explosion-proof conforming to National Electric Code Article 500 for Class I, Group D, Division I locations. Wires should be in conduits of corrosion-resistant materials such as polyvinyl chloride.

6.2.5 Plastics

Although plastics and plastic-coated metals have not been used to any appreciable extent in combined sewage regulator installations, they offer considerable promise for the future. Coatings such as epoxy, vinyl, nylon, and celloplosic applied by the fluidized bed process all endure well in sewage service. They are quite abrasion-resistant, an important consideration as combined sewage contains much grit. These coatings applied to steel and aluminum offer maximum corrosion resistance, coupled with good strength characteristics.

SECTION 7

EXAMPLE OF SYSTEMS CONTROL THROUGH INSTRUMENTATION

CONTENTS

7.1 General

The following example of total system control of a hypothetical combined sewer system through the use of instrumentation, automation, and data control devices, has been developed to provide a practical demonstration of the application methodology required for its implementation. The intent of the section is to examine how an effective control system may be developed.

7.2 Description of Project Problem

CENTROPOLIS
PROJECT NO. 5468
CONTRACT 13
REGULATING STATION NO. 5

General This will be located near Eight-Mile creek at approximately Station 28+00 on the Eight-Mile interceptor just west of the west right-of-way line of Highway 6 out of Worden. Ground elevation is approximately 885. The City has entered into a contract with the adjacent Minneola Sanitary District to divert part of the combined sewage from this interceptor to a new system which will allow the postponement of treatment plant expansion for several years. The structure will be similar in appearance to Station No. 2 for Appenouse which was completed this spring. A downstream regulated weir will be part of another contract.

Head Conditions Maximum hydraulic gradient at the regulating station influent will be EL. 870. Minimum gradient is assumed to be El. 845. Minimum gradient of the connection to the Minneola interceptor will be El. 820 and maximum gradient may approach El. 862.

Flow Limitations The present contract limits the maximum rate of diversion under any condition to 500 gpm. In addition, the maximum rate of diversion from 3:00 p.m. to 9:00 p.m. on any day when the temperature is 90°F or higher is limited to the average hourly diversion rate during the 24 hour period ending at 9:00 a.m. on such date.

Equipment The station will contain the following major items of equipment:

1. One throttling gate. This gate normally shall remain fully open for gravity flows up to the amounts limited by the contract. For higher flows the gate shall then be automatically throttled to maintain the flow within the contracted amounts.
2. One mechanically operated check gate to automatically prevent backflow through the check gate. This will require no control or electric equipment.
3. Two 125-horsepower variable speed pumping units, each sized to deliver up to the contracted maximum quantity under any imposed head condition. Either pumping unit may be selected as the lead pump with the other serving automatically as a standby pump if the lead pump fails. Pumps shall always start at a minimum speed.
4. Each variable speed pump will have a discharge valve of a particularly low allowable working pressure. It will, therefore, be necessary to design the control to prevent a differential pressure across the valve in excess of 10 psi.
5. Influent level shall be measured at Station 43+00, approximately 7500 feet upstream from the diversion regulating station. Control shall be such that at a certain adjustable low level the regulating station will shut down to prevent any diversion from the Eight-Mile interceptor.
6. The regulation station effluent flow shall be metered and used for feed-back to the control. The control and instrument designer shall investigate and determine the type of metering. The effluent conduit will be 36-inch concrete pipe.
7. The Minneola Sewer District plant influent screen chamber wet well level shall be measured and telemetered to the regulating station. At an adjustable plant influent high level, the regulating station shall be shut down.
8. Provision shall be made for two incoming 480-volt, 3-phase, 60-hp. services. Only one service is to be installed originally with space for the second service in the future.
9. Gate and valve operators shall be electric or pneumatic with independent opening and closing speed adjustments.
10. Pumps, motors and controls shall be housed indoors, separated from sewers. The building shall be force-ventilated.

Sequence Of Operation The station controls shall be such that the flow diverted from the Centropolis Eight-Mile interceptor will be maintained at a set-point rate, adjustable between 200 gpm and 1200 gpm. The sequence should be such that first the throttling gate opens slowly when Eight-Mile interceptor reaches Elevation 1.5 at level measuring Station 43+00. If the hydraulic conditions are such that set-point flow can be maintained by gravity, the throttling gate shall be automatically positioned to effectively operate as a rate-controller. As the level rises and set-point is maintained by the gate no pumping is required.

If the hydraulic conditions are such that the throttling gate is open, the level remains above 0.5 feet at the level measuring station, and the diverted flow is less than set-point quantity, the lead pumping unit should start at minimum speed and rise in speed slowly to deliver set-point flow. In this case the pumping unit shall continually vary in speed to effectively operate as a rate-controller. The pumping unit discharge valve should not commence to open, however, until the pump discharge pressure is greater than the station discharge pressure. During the sequence of putting the pumping unit on line, the speed also should be such as to limit the differential pressure across the pump discharge valve to a maximum of 10 psi. It may be decided not to start the lead pump until the level reaches approximately 2.0 feet at Station 43+00. If, due to malfunction, the lead pump should fail to start, the standby pump should be started in a similar pump and discharge valve control sequence whenever the Eight-Mile interceptor level rises to Elevation 2.25. In this case the standby pump is to effectively operate as a rate controller to maintain set-point diversion rate-of-flow. The two pumps shall never operate in parallel.

Pumping, once commenced, shall be maintained until the Eight-Mile interceptor level falls to 1.0 foot, in which case the pump speed shall be reduced to minimum, the discharge valve slowly closed, and finally, the pumping unit stopped.

High water level at the Minneola Sewer District Plant No. 1 shall override all controls and shut down the regulating station by stopping the pumps and closing the throttling gate.

In case of power failure, an over-riding control shall slowly close the throttling gate and pump discharge valves.

Arrangements shall be made for the future monitoring of station equipment status and alarm conditions at the proposed Operations Control Center whenever it is designed.

Miscellaneous During the first years of operation it is anticipated that the required quantity of diverted flow by gravity through the regulating station will sometimes, under certain hydraulic conditions, not be of sufficient quantity to fill the interconnecting conduit during normal contracted flow rate. Therefore the flow meter must not be of a design requiring a filled pipe. Flushing water will not be available for several years.

The final selection of the throttling gate and check gate has not yet been determined.

The plans and specifications must be completed in early June to be eligible for appropriations approved for this project. Plans and specifications are 80 percent complete. Rather than delay the project, certain detailed features of equipment may have to be modified and covered by change order after award of contract.

7.3 Project Requirements

This is a common situation. The project must meet a certain deadline. The problem is defined. The solution is somewhat hazy, however, depending upon the ingenuity of the control and instrumentation designer together with those of each of the other disciplines involved.

At this point the designer will begin to conceive the parts and their relationship to the whole. These parts or subsystems will be reduced to functional block diagrams and schematics. These subsystems will then be put into greater systems until all are combined into one system.

It is not feasible or practicable in many cases to wait until all of the necessary details are resolved before proceeding with the scheme of control. In this case, it is recognized that some suitable means for metering flow must be determined, and that this may be a real hurdle but for the purpose of starting the control schematic this is assumed possible and the development of the scheme is taken from the point of a metering signal representing flow. The control designer can make other assumptions too, such as the means of varying the speed of the pumps. In this case it can be assumed that some type of slip coupling can be used for speed regulation; then, if some other means of variable speed drive is selected the control scheme can be modified to conform to that particular drive circuitry.

It may be, as in this case, that the throttling valve and check gate may not be definitely determined as to type, manufacturer, etc.; however, the control designer must assume that such equipment will be found and that it can be driven by standard type operators. Again, the control designer may have some misgivings regarding the hydraulics, as defined in the memorandum; however, he must proceed upon the assumption that such information is reasonably correct or will be corrected. It also must be assumed that suitable means of communications links, such as leased telephone lines will be made available, the details of which can be ascertained later. All of this is simply to show that the instrumentation and control scheme can be commenced early in design stage, as it certainly should be, since its development points out more clearly all of the special requirements of the system equipment.

The control designer will choose from his past experience whether to use as a basic control media pneumatic, hydraulic, or electric devices, or perhaps a combination of such instruments. In this particular case, he chooses to use both pneumatic and electric systems with some hydraulic devices for timing control.

For those who are interested in examination of yet more detail that must be conceived, an example of the functional block diagrams and schematics for this hypothetical project is shown in the following figures together with a written description of the actual performance of each particular element employed.

The block chart is a typical exercise in the logic that must be conceived and employed in the design development of a control scheme. The diagram is used to describe the instrumentation, whereas schematics are used to describe the electrical circuits. By reference to these diagrams while reading the following steps, one can follow the design logic used. No two designers would necessarily arrive at the same means of solution. For example, whereas this solution has used a considerable number of pneumatic control devices, the problem could be solved using almost all electric and electronic equipment.

It has been assumed that the station would best be designed to function automatically with local controls, then to superimpose the necessary remote control features after this local automatic mode of control is fully explored and developed. This is reasonable since the station must be constructed to be operable locally for such contingencies as loss of remote control facilities, or even to be operated prior to the construction of the Central Operations Center. Details, such as time of day that certain rates might be allowed are also disregarded at this stage of the design, since they too can be assumed to be superimposed on the basic scheme.

The following describes the step-by-step features for both the local-automatic and the local-manual control modes:

Automatic Control

1. The Eight-Mile interceptor waste water level, measured over a five-foot control range for control accuracy only regardless of overall depth, is telemetered from the measuring station to a telemetering receiver at the regulating station. The telemetering receiver is equipped with adjustable contacts for control circuit switches, as follows:

   EM-1 Normally open contact, closes on rising level at 0.5 feet and remains closed above. This contact prevents pumping when the interceptor level is below 0.5-feet.

   EM-2 Normally open contact, closes on rising level at 1.0-foot and remains closed above. This contact prevents pumping when the interceptor level is below 1.0-foot.

   EM-3 Normally open contact, closes on rising level at 1.5-feet and remains closed above. This contact prevents the regulating station beginning operation until the interceptor level rises to 1.5-feet.

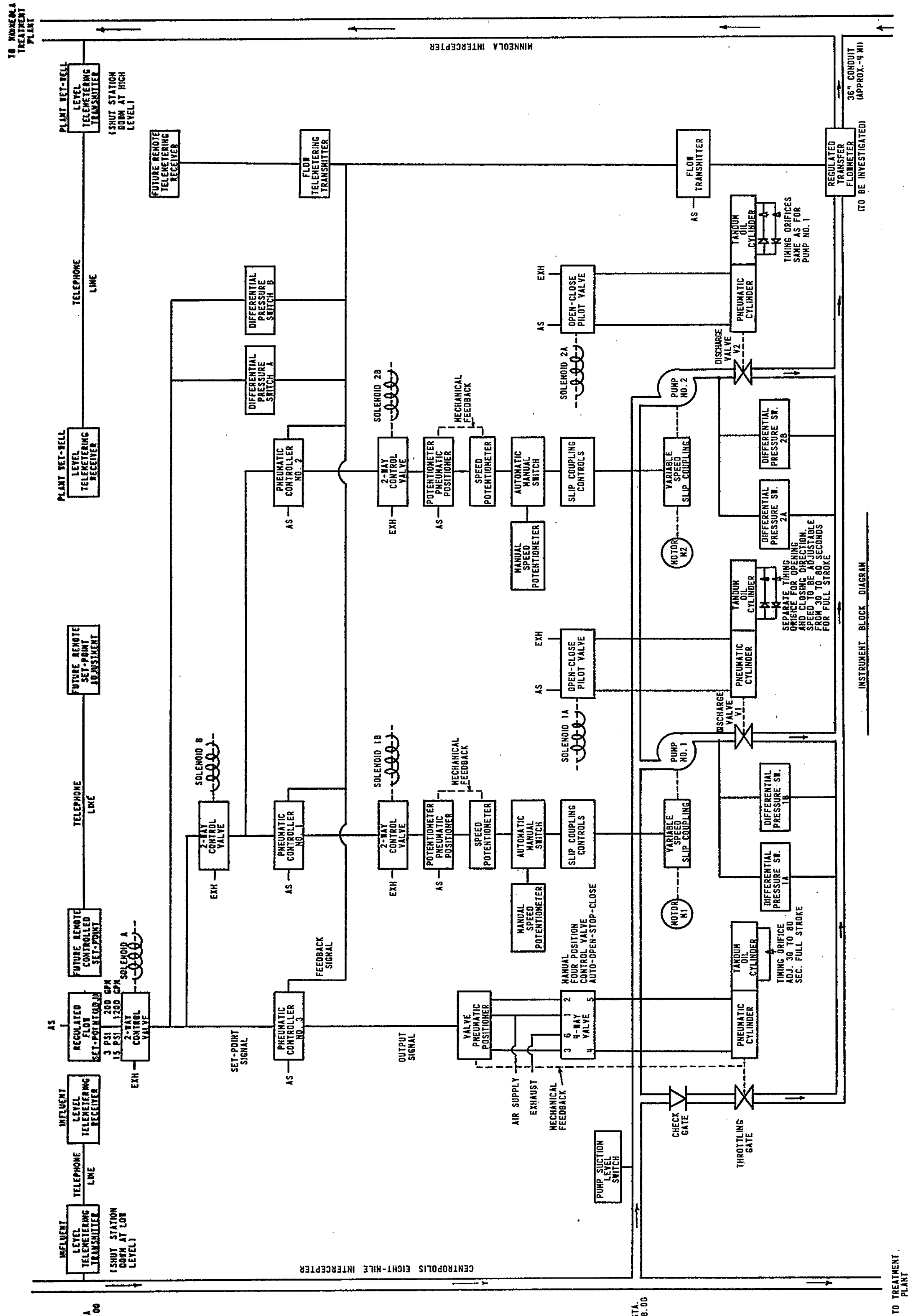
   EM-4 Normally open contact, closes on rising level at 2.0-feet and remains closed above. This contact prevents the beginning of any pumping to commence until the level rises to 2.0-feet.

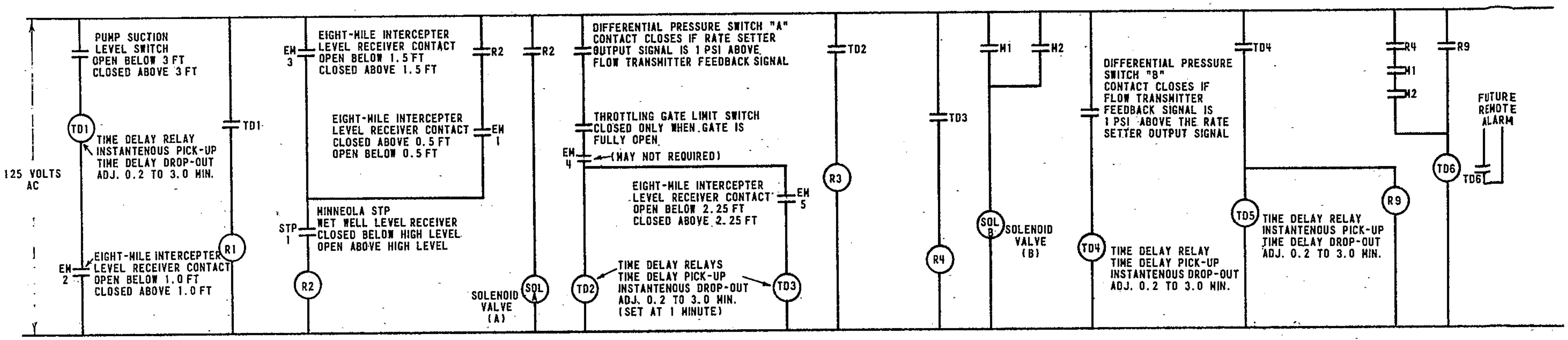
   EM-5 Normally open contact, closes on rising level at 2.25-feet and remains closed above. This contact prevents any standby pumping to commence until the level rises to 2.25-feet.

   EM-6 Normally closed contact, open on rising level at 0.05-feet and remains open above. This contact can be used for monitoring and alarm of the level measuring equipment by sensing an essentially zero level measurement.

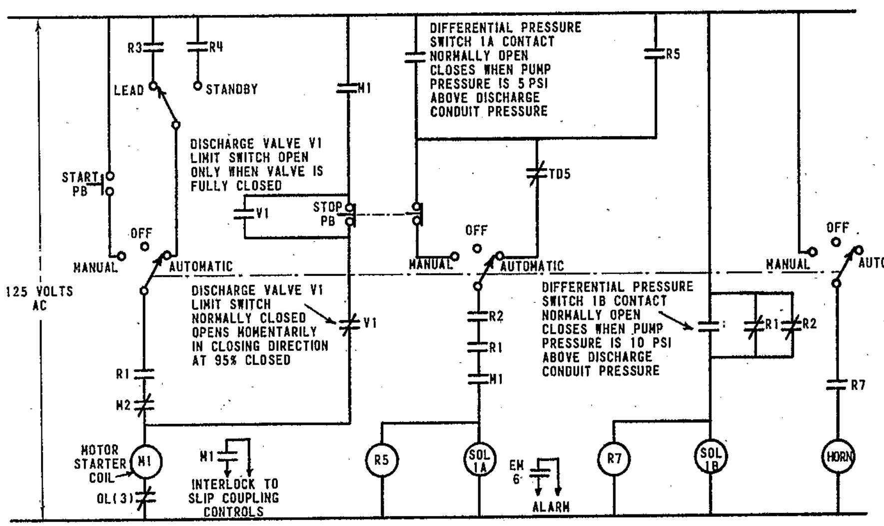
2. The Minneola sewage treatment plant influent wet well level is measured and telemetered to the regulating station. The regulating station is not to operate during the time that this plant is over-loaded. Contacts are therefore provided in the plant level telemetering receiver for use in the control circuits. Only one contact, STP 1, which is normally closed and opens on rising level at some high level setting is required.

3. A rise in waste water level on Eight-Mile interceptor to 1.5-feet closes receiver contact

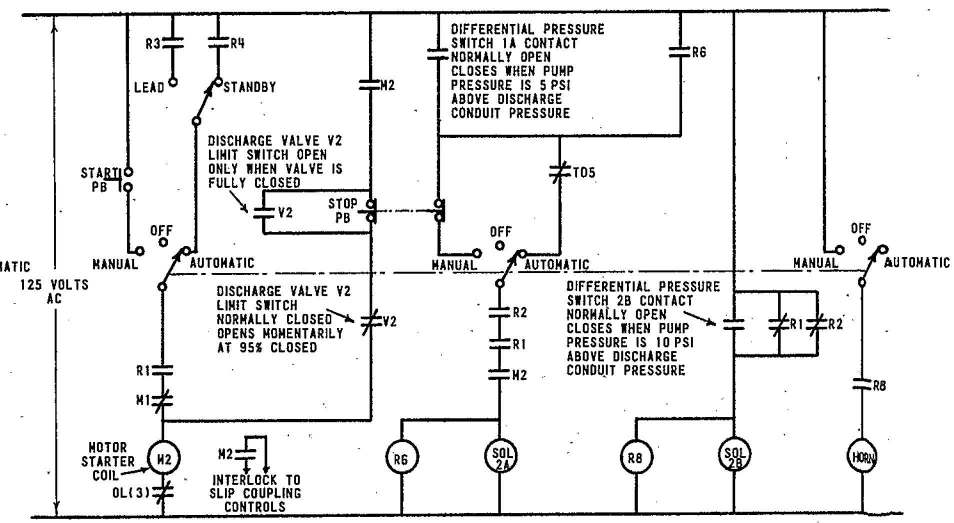

INSTRUMENT BLOCK DIAGRAM
TO MINNEOLA TREATMENT PLANT
MINNEOLA INTERCEPTER
CENTROPOLIS EIGHT-MILE INTERCEPTER
TO TREATMENT PLANT
STA. 43.00
STA. 28.00
36" CONDUIT (APPROX.-4 MI)
REGULATED TRANSFER FLOWMETER
(TO BE INVESTIGATED)
PLANT WET-WELL LEVEL TELEMETERING TRANSMITTER
(SHUT STATION DOWN AT HIGH LEVEL)
PLANT WET-WELL LEVEL TELEMETERING RECEIVER
INFLUENT LEVEL TELEMETERING TRANSMITTER
(SHUT STATION DOWN AT LOW LEVEL)
INFLUENT LEVEL TELEMETERING RECEIVER
TELEPHONE LINE
FUTURE REMOTE TELEMETERING RECEIVER
FUTURE REMOTE SET-POINT ADJUSTMENT
FUTURE REMOTE CONTROLLED SET-POINT
REGULATED FLOW SET-POINT DIAL
3 PSI 200 GPM
15 PSI 1200 GPM
2-WAY CONTROL VALVE
SOLENOID A
SOLENOID B
SOLENOID 1A
SOLENOID 1B
SOLENOID 2A
SOLENOID 2B
FLOW TELEMETERING TRANSMITTER
FLOW TRANSMITTER
DIFFERENTIAL PRESSURE SWITCH A
DIFFERENTIAL PRESSURE SWITCH B
PNEUMATIC CONTROLLER NO. 1
PNEUMATIC CONTROLLER NO. 2
PNEUMATIC CONTROLLER NO. 3
SET-POINT SIGNAL
FEEDBACK SIGNAL
OUTPUT SIGNAL
POTENTIOMETER PNEUMATIC POSITIONER
MECHANICAL FEEDBACK
SPEED POTENTIOMETER
AUTOMATIC MANUAL SWITCH
MANUAL SPEED POTENTIOMETER
SLIP COUPLING CONTROLS
VARIABLE SPEED SLIP COUPLING
MOTOR M1
MOTOR M2
PUMP NO. 1
PUMP NO. 2
DISCHARGE VALVE V1
DISCHARGE VALVE V2
OPEN-CLOSE PILOT VALVE
PNEUMATIC CYLINDER
TANDUM OIL CYLINDER
TIMING ORIFICES SAME AS FOR PUMP NO. 1
SEPARATE TIMING ORIFICE FOR OPENING AND CLOSING DIRECTION. SPEED TO BE ADJUSTABLE FROM 30 TO 80 SECONDS FOR FULL STROKE
DIFFERENTIAL PRESSURE SW. 1A
DIFFERENTIAL PRESSURE SW. 1B
DIFFERENTIAL PRESSURE SW. 2A
DIFFERENTIAL PRESSURE SW. 2B
VALVE PNEUMATIC POSITIONER
MANUAL FOUR POSITION CONTROL VALVE AUTO-OPEN-STOP-CLOSE
4-WAY VALVE
AIR SUPPLY
EXHAUST
TIMING ORIFICE ADJ. 30 TO 80 SEC. FULL STROKE
CHECK GATE
THROTTLING GATE
PUMP SUCTION LEVEL SWITCH
AS
EXH



COMMON CONTROL SCHEMATIC



PUMP NO. 1 MOTOR CONTROL SCHEMATIC



PUMP NO. 2 MOTOR CONTROL SCHEMATIC

EM-3, which is in series with the Minneola plant level receiver contact STP-1, to complete a control circuit through relay R2 to energize relay R2.

4. When relay R2 is picked up (energized) it seals itself in through one of its own contacts R2 so that it will remain energized until power to the relay is interrupted by either a fall in the intercepter level to 0.5 feet causing the level contact EM-1 to open or high level at the plant to cause contact STP-1 to open. It should be noted that relay R2 has a number of contacts which will be called upon to perform various functions and that relay R2 will act as sort of a master switch for placing the station into service and for taking it out of service.

5. One of the relay R2 contacts will close when the relay is energized to energize solenoid valve "A" that connects the station rate setter output signal to rate controller No. 3 which in turn controls the position of the throttling gate allowing flow by gravity through the station at a controlled or regulated rate as required by the rate setter. Relay R2 contacts also partially enable circuits, that is they help to set up circuits, for solenoid valves 1A, 1B, 2A and 2B, on pumping units No. 1 and No. 2 respectively, should their operation become required.

6. The throttling gate, the flowmeter and the controller function as a rate controller under this condition to maintain the set-point flow rate as determined by the rate setter. The rate setter develops an output signal representing the desired rate of flow. Controller No. 3 receives this signal as its input and compares it with the feedback signal which represents actual flow. The output of controller No. 3 automatically varies as required to position the throttling gate to that position necessary to establish an actual flow signal equal to the set-point signal. Under this condition the actual fl w is equal to the desired or set point flow. This is referred to as closed control loop with feedback.

7. If the Eight-Mile interceptor level falls to a level below 0.5 feet, relay R2 is de-energized, switching the controller input set-point signal line to exhaust (zero flow signal) causing the throttling gate to close to satisfy the zero flow signal. This, in effect, shuts down the station, after it normally has functioned by gravity flow for a period, without having entered a pumping stage.

8. If the desired (set-point) flow cannot be maintained by gravity flow through the throttling gate, the gate obviously will have reached the fully open position, since it is positioned to try to satisfy controller No. 3. In this situation the diverted flow signal pressure will be less than the rate setter (set-point) signal pressure. By sensing the occurrence of this condition, the sensor can be used to initiate pumping. This sensor is differential pressure switch "A".

9. Differential pressure switch "A" shall be set to close a switch contact whenever the rate setter output signal exceeds the flow transmitter output signal by approximately 1.0 psi.

10. The switch contact of differential pressure switch "A" connected in series with a throttling gate limit switch, which closes only when the gate is fully open, energizes the time delay relay TD2. This time delay relay is to prevent starting of a pump unless this condition lasts for a reasonable period.

11. The time delay relay TD2 is adjustable from 0.2 to 3 minutes. If the throttling gate is open and the rate of flow through the regulating station is less than the desired (set-point) rate for a period equal to the time delay relay setting (one minute, for example), the time delay relay contact TD2 closes to pick up relay R3.

12. Relay contacts R3 close to complete the starting circuits of both pumps. It will not complete the starting circuit on the pump selected as "standby"; however, it will complete the starting circuit for the pump selected as "lead" provided there is sufficient suction level and that the second pumping unit is not running (a condition which could occur, for example, if it has been started under manual control). These conditions are imposed on the starter circuits by the series connection of contacts R1 and M2 in Pump No. 1 and contacts R1 and M1 in Pump No. 2.

13. To insure that sufficient water level on the influent to the regulating station exists for pumping, a level switch installed on the suction piping closes its switch contact at a measured water depth of 3-feet and above. This switch energizes time delay relay TD1.

14. The time delay relay TD1 is adjustable from 0.2 to 3 minutes. This relay, when energized closes its contact TD1 instantaneously but when de-energized the contact will delay opening for a period equal to the time delay relay setting (one minute, for example). Thus, loss of influent water level must be sustained for a reasonable

period to cause the time delay relay TD1 contact to open. This prevents stopping the pump upon a transient fall in water level during pump starting that is not sustained after pumping is commenced.

15. Time delay relay TD1 contact closes to pick up relay R1.

16. Relay R1 contacts energize each pump starting circuit, discharge valve control circuit, circuits to control the pump speed. These contacts are used in such a manner that, if a pump is running, loss of water level on the suction for a definite period de-energizes relay R1; causing the pump to reduce speed to minimum, the pump discharge valves to close, and the pump motor to stop.

17. Thus with sufficient suction water level available, closure of relay R3 contacts as stated above, starts the "lead" pump.

18. The pump starts at minimum speed. The set-point signal pressure to the pump speed controller is connected to exhaust (minimum speed signal) through solenold "B". Note at this point that solenoid "B" is still de-energized.

19. When the motor starts, its motor starter holding coil (M1 or M2) is sealed by respective auxiliary (M1 or M2) starter contacts (M1 or M2) in series with a "Stop" push-button contact and a knee action limit switch on the pump discharge valve, thus eliminating any further effect of time delay relay TD2 and relay R3.

20. The closure of either pump motor starter auxiliary contact (M1 or M2) picks up solenoid "B", which switches the pneumatic set-point signal of the speed controllers (No. 1 and No. 2) from exhaust pressure (zero flow signal) to the rate setter signal pressure. This forces the pumping unit always to start from minimum speed.

21. The pump discharge valve is not to open until the developed pumping head exceeds any possible station discharge pressure by 5 psi. Differential pressure switches "1A" and "2A" on pumps No. 1 and No. 2, respectively, close their contacts as soon as the pump pressure exceeds the station discharge pressure by 5 psi.

22. Closure of the differential pressure switch contacts (1A or 2A) completes the control circuit through time delay relay TD5 contact, the automatic position of the pump "Manual–Off–Auto" switch, relay contact R2, relay contact R1, and the respective motor starter auxiliary switches (M1 or M2) to the pump discharge valve (V1 or V2) control solenoid (1A or 2A) and relay (R5 or R6) on pump No. 1 or No.2, respectively, to initiate discharge valve opening.

23. When the pump discharge valve control solenoid (1A or 1B) becomes energized, it is immediately sealed by its associated relay (R5 or R6) contact so that the pump discharge valve continues to travel to the fully open position.

24. Also, during the starting and stopping of a pumping unit, the pump speed never is allowed to cause a differential pressure across the discharge valve in excess of 10 psi. To sense this condition, differential pressure switch (1B or 2B) across valve (V1 or V2) on pump No. 1 or No. 2; respectively, closes its contact when the developed pump pressure exceeds the station discharge pressure by 10 psi.

25. Closure of the differential pressure switch (1B or 2B) energizes solenoid (1B or 2B), respectively, which switches the pneumatic control signal to the positioner on the pump speed potentiometer to exhaust (zero speed) pressure, causing the positioner to start to operate in the direction of speed reduction. As speed is reduced to the point where the differential across the valve is within 10 psi, the solenoid (1B or 2B) is de-energized, thus connecting the controller output back to the positioner for continuation of speed control.

26. When the pump discharge valve is completely open, the variable speed pump together with the flowmeter and pneumatic controller associated with that pump then function as a closed control loop to regulate flow at the desired rate setting.

27. If the lead pump should fail to start and the Eight-Mile interceptor level rises to 2.0 feet, time delay relay TD3, which is adjustable from 0.2 to 3 minutes, will pick up relay R4 if this condition persists for a period equivalent to the relay time setting (one minute, for example).

28. Relay 4 contacts would then close to start the standby pump. The starting sequence of the standby pump would be as described above for starting of the lead pump. Note, however, that the standby pump will not start if the lead pump is running because of the interlocking starter auxiliary switches (M1 or M2).

29. As the hydraulic conditions vary, the variable speed pump may go to full speed if necessary to try to maintain the rate-set flow.

30. Also as the hydraulic conditions vary, the variable speed pump may reduce speed to a

minimum, yet the flow through the station might then exceed the rate-set (set-point) flow. When this condition develops, the station obviously could perform by gravity, without the pumping units. In this case the flow transmitter output signal pressure will exceed the rate setter output signal pressure. This condition is sensed by differential pressure switch "B" which closes its switch contact whenever the flow transmitter signal exceeds the rate setter signal by one psi.

31. Differential pressure switch "B" contact then closes to energize time delay relay TD4, which must be energized for a period (adjustable from 0.2 to 3 minutes) before its contact closes; hence, station flow greater than the required set-point flow for one minute, for example, will cause relay TD4 contact to close to pick up time delay relay TD5.

32. Time delay relay TD5 is one which has contacts that open immediately when energized and remain open after being de-energized for an adjustable period of 0.2 to 3 minutes.

33. The normally closed time delay relay TD5 contacts will then open to de-energize the control solenoid (1A or 2A) which causes the pump discharge valve (V1 or V2) to close. The purpose for the delay of the TD5 contacts is to allow sufficient time for the pump discharge valve to fully close. This relay should be set for a delay period slightly greater than the time it takes for the discharge valve to close.

34. As the discharge valve closes, the differential switch (1B or 2B) will prevent speed increase to occur to the point where the differential across the valve exceeds 10 psi, as explained previously, should the flow through the station fall below the set-point flow thus causing the pump speed controller to try to cause an increase of pump speed.

35. As the pump discharge valve closes the pump bypass check gate will commence to open to allow flow through the throttling gate by gravity.

36. As the pump discharge valve moves in its closing direction a knee action limit switch mounted on the valve will momentarily open the holding circuit of the motor starter as the discharge valve passes its 95 percent closed position, to stop the pump motor. The knee action switch on the valve is adjustable from 90 percent to 97 percent port closure. (The knee action limit switch on the discharge valve is normally closed and stays closed while the valve opens and during valve closure with the exception of opening momentarily during the closing stroke at approximately the position of 5 percent valve port opening.

37. As soon as the motor starter is de-energized auxiliary motor starter contact (M1 or M2) opens to de-energize solenoid "B" which disconnects the rate setter signal from the pump speed controllers, causing the set-point signal pressure to the pump speed controlers (No. 1 and No. 2) to drop to exhaust pressure (zero speed signal pressure). This in turn causes the pump speed control potentiometers to be returned to zero speed position.

38. Controller No. 3 is then in control of the rate of flow through the throttling gate, acting again as a rate control system for gravity flow through the station.

39. The station then automatically returns to pumping if required or continually regulates flow through the throttling gate or, if the Eight-Mile interceptor water level falls to below 0.5 feet a contact in the Eight-Mile level receiver will open to break the seal-in circuit on relay R2, thus de-energizing relay R2.

40. When relay R2 is de-energized, the solenoid "A" in the station rate setter output signal line is de-energized, which switches the set-point signal line to the flow control system to exhaust, (or zero signal pressure) causing the throttling gate to close, thus shutting down the station.

41. Anytime during operation, if the Minneola Treatment Plant wet well level becomes too high, telemetering level receiver contact STP-1 opens to de-energize relay R2 to shut down the regulating station.

Manual Control

Manual control is provided mainly for checking the individual station components and for control whenever the Eight-Mile interceptor level telemetering receiver controller is out of service.

1. Under gravity flow conditions, the throttling gate may be throttled to any desired position and stopped and held in that position by means of the four-position "Automatic-Open-Stop-Close" manually operated pneumatic selector switch. In this case an attendant is required at the station to maintain the flow within the prescribed limits by watching the flowmeter. (Any time that the station is on automatic rate control this selector switch must be in the "Automatic" position.) This manual means of control is not necessary for local control of the station if pneumatic controller No. 3 is operative, since the controller

also can be switched to a manual loading station or set-point signal on the controller.

2. Under gravity flow conditions, if controller No. 3 is operative, the attendant may switch this controller from "Automatic" to "Manual" and adjust the controller output to obtain the desired flow.

3. If pumping is required, the attendant must switch the pump motor starter controls from "Automatic" to "Manual", and switch the pump speed control to a manually operated speed control potentiometer. (The speed control potentiometer should be set at its minimum speed position before starting the pump.)

4. The pumping unit is started by depressing the "Start" push-button. This closes the pump starter circuit and also starts the slip coupling to operate at the speed determined by the setting of the manually operated speed control potentiometer

5. The pump starter seals itself through its auxiliary contact (M1 or M2), the "Stop" push-button parallel with the valve limit switches, (V1 or V2), and the knee action switches (V1 or V2). (The purpose of parallel limit switches V1 or V2 is to permit stopping the pump motor if, for some reason, the discharge valve fails to open during the initial starting period.)

6. When the pump motor starts, the pump speed must be increased slowly until the pump discharge pressure is 5 psi above the station discharge pressure, to cause the pump discharge valve to commence to open. The differential pressure switch (1A or 2A) senses the 5 psi differential and energizes the pump discharge valve control solenoid (1A or 2A) and relay (R5 or R6) to seal the control solenoid circuit to drive the discharge valve fully open.

7. The attendant must raise the speed of the pumping unit gradually during the pump discharge valve opening cycle to prevent the development of greater than a 10 psi differential across the discharge valve.

8. If the pressure across the discharge valve exceeds 10 psi, the relay (R7 or R8) will pick up and sound an alarm. (This will notify the attendant to reduce or at least not to increase speed until the valve opens farther.)

9. When the discharge valve is fully open, the speed of the pumping unit is manually controlled to maintain the flow of the station at the required stage by watching the flowmeter and adjusting the manual speed control potentiometer accordingly.

10. The pumping unit is shut down by depressing the "Stop" push-button.

11. As the valve begins to close the speed should be reduced to maintain less than 5 psi differential across the valve.

12. When the valve is 97 percent closed, the pump motor will stop.

Alarms and Instruments for Future Operations Control Center.

1. If relay R4, which actuates the standby pump, is energized and neither pump is running, the time delay relay TD6 is energized.
2. If relay R9, which indicates that the station flow exceeds set-point flow, is energized, the time delay relay TD6 is energized.
3. Time delay relay TD6, when energized for an adjustable period of 0.2 to 3 minutes, will close its contact to transmit an alarm tone via leased telephone line to the Operations Control Center.
4. The diverted flow rate is telemetered to the Operations Control Center. The recording receiver has an alarm contact which closes if the flow exceeds the contracted allowable limit.
5. As noted on the block diagram, it is planned that remote rate setting equipment will be installed in the future. Obviously when this is done a transfer switch "Local-Remote" should be installed to allow the use of either the original local rate setter or the remote rate setter.

The next step is to simplify the system as much as possible. This particular problem is an example of just how complex a system might become by seemingly relatively simple conditions existing when determining the design requirements. Limiting the working pressure across the discharge valves, for example, has introduced a number of controls, whereas, the selection of a different type of valve might have significantly simplified the control scheme. As the design progresses, it behooves the persons in each professional discipline involved to be aware of the complications they may be introducing by their particular design or selection of equipment and to attempt to simplify the control requirements.

Once the system has been simplified and established the scaling or calibration of the devices is necessary. This will involve the selection of the most suitable ranges for measurement with respect to control performance. In some cases separate metering for control is advisable, such as the measurement of water level. In this instance, one measurement is advisable for metering of zero to maximum level; another for level measurement of the narrow band over which control is to be exercised.

Another item that must be considered is use of "live" or "dead" zero output signals from instruments, as with time-impulse telemetering equipment. Arguments can be presented for both; but, for a particular application one will be more suitable than the other.

The designer must be concerned with accuracy, reliability, and maintenance requirements; must provide for clean, dry air for pneumatic systems as well as for voltage regulated within prescribed limits, for electrical circuits, together with over-voltage protective devices.

Although nothing has been mentioned herein regarding the telephone lines or communication facilities, it must be emphasized that regardless of the degree of excellence of the control system or equipment, the success or failure of the system will depend upon the quality and reliability of the communication facilities.

7.4 Comments

The customer must be extremely careful about what is required from the control and instrument designer. Almost any system control problem can be solved but the cost of implementation and operation must not outweigh the benefits to the total system.

In the development of an Operation Control Center, the approach is somewhat similiar to that mentioned herein for the design of a single station control scheme. Here the complete picture may be represented again in block diagram form with each block representing a certain controlled station, telephone exchange, and control center, with the interconnecting lines representing the communications channels. The degree of success will depend almost entirely upon the extent of understanding that each and every one has about the whole system and all of its component parts. This must include management, all of the professional services involved in design, equipment suppliers, contractor, and telephone and power utility personnel.

SECTION 8

ACKNOWLEDGEMENTS

The American Public Works Association is deeply indebted to the following persons and their organizations for the services they rendered to the APWA Research Foundation in carrying out this study for the 25 local governmental jurisdictions and the Federal Water Quality Administration who co-sponsored the study. Without their cooperation and assistance the study would not have been possible. The cooperation of the American Society of Civil Engineering (ASCE) and the Water Pollution Control Federation (WPCF) is acknowledged for their participation on the project Steering Committee.

Steering Committee

Arthur D. Caster (WPCF)	Walter A. Hurtley
William Dobbins (ASCE)	Peter F. Mattei, Chairman
George T. Gray	Ed Susong
Carmen Guarino (WPCF)	Harvey Wilke (ASCE)

Consultants

Dr. Morris M. Cohn, Consulting Engineer
Ray Lawrence, Black & Veatch, Consulting Engineers
M. D. R. Riddell, Greeley and Hansen, Consulting Engineers
Morris H. Klegerman, Alexander Potter Associates, Consulting Engineers
James J. Anderson, Watermation, Incorporated

Federal Water Quality Administration

Darwin R. Wright, Project Officer
William A. Rosenkranz, Chief, Storm and Combined
Sewer Pollution Control Branch, Division of Applied
Science and Technology.

Manufacturers Advisory Panel

Vernon F. Brown	Badger Meter Manufacturing Co.
Peter A. Freeman	Bowles Fluidics Corporation
R. E. Gerhard	Allis-Chalmers Company
R. W. Henderson	Rodney Hunt Company
Karl E. Jasper	American Chain & Cable Co., Inc.
Louis F. Lemond	Coldwell-Wilcox Company
Charles Prange	Rockwell Manufacturing Co.
Milton Spiegel, Chairman	FMC Corporation
Jack D. Stickley	Honeywell, Inc.
E. P. Webb	Firestone Coated Fabrics Co.
Leon W. Weinberger	Zurn Industries, Inc.

ACKNOWLEDGEMENTS (Continued)

Advisory Committee

Vinton Bacon	The Metropolitan Sanitary District of Greater Chicago, Illinois
Donald Bee	City of Muncie, Indiana
Philip Blunck	Municipality of Metropolitan Seattle, Washington
C. A. Boeke	City of Middletown, Ohio
C. A. Boileau	City of Montreal, Quebec, Canada
Ron Bonar	City of Fort Wayne, Indiana
Richard J. Durgin	City of Alexandria, Virginia
Paul Ehrenfest	City of Cleveland, Ohio
John F. Flaherty	City of Boston, Massachusetts
George T. Gray	Allegheny County Sanitary Authority, Pittsburgh, Pennsylvania
Allison C. Hayes	Metropolitan District Commission, Boston, Massachusetts
Robert S. Hopson	City of Richmond, Virginia
Walter A. Hurtley	City of St. Paul, Minnesota
Roy L. Jackson	City of Kansas City, Missouri
Gene E. Jordan	City of Omaha, Nebraska
Robert E. Lawrence	Metropolitan Government of Nashville & Davidson County
O. H. Manuel	City of Charlottetown, P.E.I., Canada
Peter F. Mattei	Metropolitan St. Louis Sewer District, Missouri
Hugh McKinley	City of Eugene, Oregon
George J. Moorehead	Washington, District of Columbia
J. D. Near	City of Toronto, Ontario, Canada
Richard W. Respress	City of Atlanta, Georgia
Max N. Rhoads	City of Owensboro, Kentucky
Harry E. Rook	City of Syracuse, New York
Ben Sosowitz	The Metropolitan Sanitary District of Greater Chicago, Illinois
Ed Susong	City of Akron, Ohio

BIBLIOGRAPHIC: American Public Works Association, Research Foundation. Combined Sewer Regulations and Management — Manual of Practice FWQA Publication No. 11022DMU08/70

ABSTRACT: Design application operation and maintenance of combined sewer overflow regulator facilities are detailed in this Manual of Practice, developed in conjunction with a report prepared on combined sewer overlfow regulators.
Design calculations are given for various types of regulators and tide gates. A sample regulator facility control program is given to illustrate the development of a control system. Operation and maintenance guidelines are also given. Thirty-eight sketches and photographs are included.
This manual and accompanying report were submitted in fulfillmennt of Contract 14-12-456 between the Federal Water Quality Administration, twenty-five local jurisdictions and the APWA Research Foundation.

KEY WORDS

Combined Sewers
Overflows
Regulators
Design
Operation
Maintenance
System Control
Quantity of Overflow
Quality of Overflow
Tide Gates

BIBLIOGRAPHIC: American Public Works Association, Research Foundation. Combined Sewer Regulations and Management — Manual of Practice FWQA Publication No. 11022DMU08/70

ABSTRACT: Design application operation and maintenance of combined sewer overflow regulator facilities are detailed in this Manual of Practice, developed in conjunction with a report prepared on combined sewer overlfow regulators.
Design calculations are given for various types of regulators and tide gates. A sample regulator facility control program is given to illustrate the development of a control system. Operation and maintenance guidelines are also given. Thirty-eight sketches and photographs are included.
This manual and accompanying report were submitted in fulfillmennt of Contract 14-12-456 between the Federal Water Quality Administration, twenty-five local jurisdictions and the APWA Research Foundation.

KEY WORDS

Combined Sewers
Overflows
Regulators
Design
Operation
Maintenance
System Control
Quantity of Overflow
Quality of Overflow
Tide Gates

BIBLIOGRAPHIC: American Public Works Association, Research Foundation. Combined Sewer Regulations and Management — Manual of Practice FWQA Publication No. 11022DMU08/70

ABSTRACT: Design application operation and maintenance of combined sewer overflow regulator facilities are detailed in this Manual of Practice, developed in conjunction with a report prepared on combined sewer overlfow regulators.
Design calculations are given for various types of regulators and tide gates. A sample regulator facility control program is given to illustrate the development of a control system. Operation and maintenance guidelines are also given. Thirty-eight sketches and photographs are included.
This manual and accompanying report were submitted in fulfillmennt of Contract 14-12-456 between the Federal Water Quality Administration, twenty-five local jurisdictions and the APWA Research Foundation.

KEY WORDS

Combined Sewers
Overflows
Regulators
Design
Operation
Maintenance
System Control
Quantity of Overflow
Quality of Overflow
Tide Gates

1 Accession Number	2 Subject Field & Group	**SELECTED WATER RESOURCES ABSTRACTS** INPUT TRANSACTION FORM	

5 Organization
The American Public Works Association, 1313 E. 60th Street
Chicago, Illinois 60637

6 Title
COMBINED SEWER REGULATION AND MANAGEMENT

10 Author(s): The American Public Works Association	11 Date: June 25, 1970	12 Pages	15 Contract Number: 14-12-456
	16 Project Number: APWA 68-01		21 Note

22 Citation

23 Descriptors (Starred First)
*Overflow, *Regulation, design, operations, maintenance

25 Identifiers (Starred First)
*Combined sewers, system control, quantity of overflow, quality of overflow, tide gates

27 Abstract

Design application, operation and maintenance of combined sewer overflow regulator facilities are detailed in this Manual of Practice, developed in conjunction with a report prepared on combined sewer overflow regulators.

Design calculations are given for various types of regulators and tide gates. A sample regulator facility control program is given to illustrate the development of a control system. Operation and maintenance guidelines are also given. Thirty-eight sketches and photographs are included.

This manual and accompanying report were submitted in fulfillment of Contract 14-12-456 between the Federal Water Quality Administration, twenty-five local jurisdictions and the American Public Works Research Foundation.

Abstractor: Richard H. Sullivan
Institution: APWA Research Foundation

WR:102 (REV. OCT. 1968)
WRSIC

SEND TO: WATER RESOURCES SCIENTIFIC INFORMATION CENTER
U.S. DEPARTMENT OF THE INTERIOR
WASHINGTON, D.C. 20240

As the Nation's principal conservation agency, the Department of the Interior has basic responsibilities for water, fish, wildlife, mineral, land, park, and recreational resources. Indian and Territorial affairs are other major concerns of America's "Department of Natural Resources."

The Department works to assure the wisest choice in managing all our resources so each will make its full contribution to a better United States—now and in the future.