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Combined Sewer Temporary Underwater Storage Facility

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

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COMBINED SEWER TEMPORARY
UNDERWATER STORAGE FACILITY

by

Melpar
An American-Standard Company
7700 Arlington Boulevard
Falls Church, Virginia 22046

for the

FEDERAL WATER QUALITY ADMINISTRATION

DEPARTMENT OF THE INTERIOR

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ABSTRACT

A pilot plant underwater storage facility was designed, constructed, operated and evaluated as a method of temporarily storing storm overflow from the combined sewer of the Choptank Avenue drainage basin, Cambridge, Maryland. Combined sewage in excess of the sewer capacity, which would normally be discharged directly into the Choptank River, was intercepted and pumped into a nominal 200,000 gallon flexible underwater storage container located 1300 feet offshore. The stored overflow was later returned from the tank at a rate which could be accommodated by the intercepting sewer and treatment plant.

The facility was tested with overflow both from four naturally occurring rainfalls and using fresh water simulation. The overflow samples were analyzed in a field laboratory for the following characteristics: pH, suspended solids, volatile suspended solids, settleable solids, 5 day biochemical oxygen demand, and chemical oxygen demand.

The pilot plant facility was capable of collecting 96 percent of the average annual overflow from the drainage basin at a cost of less than $1.85 per thousand gallons. The facility could prevent the annual discharge of 7,136 pounds BOD into the Choptank River.

Underwater storage facilities could be used effectively for many combined sewer areas. Site selection, however, has proved to be a critical factor. Care must be exercised to prevent public disturbance, and factors such as land use, tidal conditions, or the types of storms, must also be considered.

This report was submitted in fulfillment of contract number 14-12-133 under the sponsorship of the Federal Water Quality Administration.

Key Words: Storm overflow, combined sewers, underwater storage, hydrology, pumping station.

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SECTION I

PROJECT CONCLUSIONS AND RECOMMENDATIONS

Conclusions

The construction of an offshore underwater temporary storage facility at Cambridge, Maryland, has demonstrated the feasibility of utilizing this concept as a means of abating pollution resulting from storm overflow from a combined sewer. Unfortunately, adverse public reaction to the pilot project limited the contractor's ability to accomplish a thorough evaluation of both the mechanical aspects of the system and those analytical parameters of interest to monitoring the biological and chemical characteristics of the overflow handled by this system.

The impact of public non-acceptance upon projects such as this cannot be dismissed, for Cambridge is typical of many older communities located adjacent to national water resources. Frequently, the sewage treatment facilities are only primary in nature and have not kept pace with the needs of the growing population. Invariably, such systems allow any overflow to discharge directly into the nearest water course. The expense of replacing combined sewers with separate dual systems is generally prohibitive, even if it is physically possible to accomplish such construction.

Public resentment of this demonstration project was such that it became necessary to curtail the operation of the facility prior to satisfactory evaluation, and to completely remove the installation. Had it been possible for the sewage treatment facility at Cambridge to have retained the installation for permanent use, it would certainly have been of long-range public benefit.

During the operation of the facility, it was determined that the use of flumes as metering devices for the discharge of the drainage basin was ineffective, especially for the conditions of the tidal river. The tide caused surcharged conditions within the sewer, thus flooding the measuring devices, and causing inaccurate readings.

Mechanically, the combined sewer temporary underwater storage facility operated as designed. It was capable of storing 200,000 gallons of storm overflow (approximately 4.5% of the daily capacity of the Cambridge Treatment Plant), which could then be pumped into the sewer system at rates up to 48,000 gallons per hour at such time as capacity to treat the stored overflow was once more available.

Recommendations

It is recommended that future demonstration projects of this type place emphasis upon site selection, not only from a practical engineering standpoint, but also with respect to public acceptance giving preference to locations that are not adjacent to residential areas. It would be desirable to accept both the increased facility costs and the inconvenience of rerouting the overflow sewage line to a remote location, rather than to jeopardize the success of the project through public resistance.

Additionally, it is recommended that a number of design changes be incorporated into future systems of this type, as follows:

1. Protective devices should be installed around the eductors of the circulation system in the storage module to prevent damage to the flexible cover.

2. Additional ventilation hose should be added to the flexible cover. Seven ventilators are recommended for the system described in this report. An ell should be installed at the junction of the center hose and flexible cover to prevent kinking of the hose.

3. Better methods for measuring flow rates in sewers should be devised.

4. It is recommended that interface instrumentation be utilized for the synchronization of rainfall-runoff data collection to provide more accurate and precise results and conclusions.

5. The samplers utilized on the project are not recommended for the sampling of sewage from combined sewers. A more advanced and efficient sampling method should be developed for future programs.

SECTION II

INTRODUCTION

Recognition of the Nation's increasing concern over the magnitude of the water pollution problem came in the Water Pollution Control Act of 1948. This act was the first national effort to treat the pollution control problem. In 1956, Congress extended and strengthened the 1948 Act, in part by increasing the effort on pollution research. Subsequent amendments in 1961, 1965, and 1966 were made to strengthen the procedures for abating pollution. Since 1956 an intensified research effort has been enforced nationally to eliminate water pollution and restore our waterways to usable quality standard. Many new and innovative techniques are being evaluated in an effort to achieve this goal. This report concerns a demonstration project to minimize pollution from the discharge from combined sewers during periods of rainfall runoff.

The combined sewer carries sanitary sewage and other waste-water to points of treatment during dry weather. During storm periods this sewer serves as the collector of storm water from streets and other sources and conveys the mixture of sanitary sewage-storm water to points of treatment or discharge. Overflow pipes or channels transmit the excess flow from combined sewers directly to the receiving waters. Sanitary engineers and public health officials long believed that combined sewer overflow water was so dilute that its discharge would not degrade the receiving stream. Today, these officials are installing signs warning of bacterial contamination caused by the discharging of sanitary wastes into receiving waters via the combined sewer outfalls. These discharges adversely affect all known water uses in the receiving water courses. These wastes contain pathogenic organisms that can cause diseases such as typhoid, dysentery, and infectious hepatitis. The wastes provide nutrients, particularly nitrogen and phosphorus, for algal growth and promote eutrophication if discharged in a body of still water. More and more lakes are experiencing ecological unbalances because of the pollutional loads.

A multitude of old cities, both large and small, having combined sewers with overflows discharging in the nearest water course are faced with a requirement for pollution abatement in their community by legislation. These cities are confronted with the complex problem of combined sewers existing in the most fully developed, highly congested parts of the city where land is simply not available. To replace the combined sewers with a separate dual system would be extremely expensive, if not prohibitive, and would produce immeasurable disruption to an entire community. To assist these communities in their development of pollution control, it is necessary to have a national effort for evaluating the feasibility of the many approaches offered; separation within the sewer lines, stilling ponds, and storage tanks are a few of the measures being studied.

The objective of this program was to design, construct, operate and evaluate, and remove (if necessary) a pilot underwater storage facility that would function as a surge tank on a combined sewer system. The purpose of the facility was to retain the flow in excess of the sewer capacity and later return it for processing at a rate which the sewer system and treatment plant could handle. This approach was unique in that the storage reservoir was a flexible tank placed in the adjoining waterway in such a manner that it did not require the use of expensive land in a congested area. The design of the storage tank and its accompanying systems is considered an optimum balance between economics, simplicity, dependability, adaptability to a modular type installation, and future use potential.

This project was installed on the Choptank Avenue drainage basin in Cambridge, Maryland. The drainage basin is 20 acres in size and is sewered by one combined sewer running north down Choptank Avenue to the Choptank River. The basin is fully developed for residential use.

The project was divided into four distinct phases; design, construction, operation and evaluation, and removal, which are described in detail in the subsequent sections of this report. The interception and storage system evaluated in this project consisted of metering manholes, a diversion manhole, wet well, pumping station, transfer lines, and flexible storage module. Except for the storage module the other pieces of hardware were items of standard manufacture. The storage module basically was a steel tank with a rubber diaphragm top which either inverted into the tank or expanded above the tank.

The basic purpose of this pilot project was to assemble the individual components as a single integrated unit to demonstrate that storm overflow from a discharge sewer could be stored for later sewage treatment. Appropriate instrumentation and analytical techniques were also included to allow the evaluation of the characteristics (both quality and quantity) of the water that overflows from a combined sewer system and the changes which might occur during storage.

SECTION III

CHARACTERISTICS OF THE SELECTED SITE

After surveying sites in Virginia, Maryland, Washington, D.C., Pennsylvania, and Delaware, the Choptank Avenue drainage basin in Cambridge, Maryland, was selected as the site for this demonstration project, and approval from all governmental agencies concerned was obtained. Figure 1 shows the geographic location and outline of the drainage basin within the city limits of Cambridge. The basin is 20.0 acres in size and is sewered by one combined sewer running north down Choptank Avenue to the Choptank River. The basin is fully developed for residential use as illustrated in figure 2. The residential development is typical of an older community with the homes built on narrow lots and located close to the street. The imperviousness of the entire drainage basin was estimated and set at 60 percent. The outfall drain, shown in figure 3, is a 18-inch diameter pipe extending 290 feet out into the Choptank River.

At the selected site the Choptank River is approximately 2 miles wide with a narrow channel in the center, which is deep enough to allow the passage of ocean going vessels up to the city harbor. The river is relatively shallow with a gentle slope from the shoreline to the edge of the center channel where the depth is approximately 12 feet.

Prior to the middle nineteen thirties, the entire sewer system of the city of Cambridge discharged into the Choptank River. During the middle nineteen thirties, an interceptor was designed and built to intercept the dry weather flow from the combined sewers along the waterfront to transmit the flow to the municipal sewage treatment plant. Simultaneously, 18 diversionary structures were built with each structure equipped with an overflow weir and tide gate. The Choptank Avenue diversionary structure, with elevations, is shown in figure 4. The diversion manhole on the Choptank Avenue sewer at Hambrooks Avenue is designed so that a maximum flow of 850 gpm is intercepted when the outlet end has a free fall condition. This, however, is not the case when a storm exists because the sewer system upstream contributes to the total flow in the Hambrooks Avenue sewer surcharging the sewer and submerging the Choptank Avenue diversion line. This reduces the discharge capacity of the diversion line to approximately 350 gpm.

A flow of 350 gpm represents the flow equivalent to the maximum dry weather flow (DWF) and the runoff from a rainfall of 0.05 in./hr. This is based on the average dry weather flow of 30 gpm, a maximum daily rate of 2.5 times the DWF and a runoff flow of 54 gpm/0.01 in./hr of rainfall from the Choptank Avenue drainage basin. The Choptank Avenue drainage basin represents about 6 percent of the total area of fully developed land that is served by the city's combined sewer system.

An evaluation of meteorological records was made of several surrounding United States Weather Bureau (USWB) stations. The results of the evaluation established certain criteria that were directed toward the development and sizing of the pilot plant facility for the retention of overflow water from the Choptank Avenue combined sewer system.

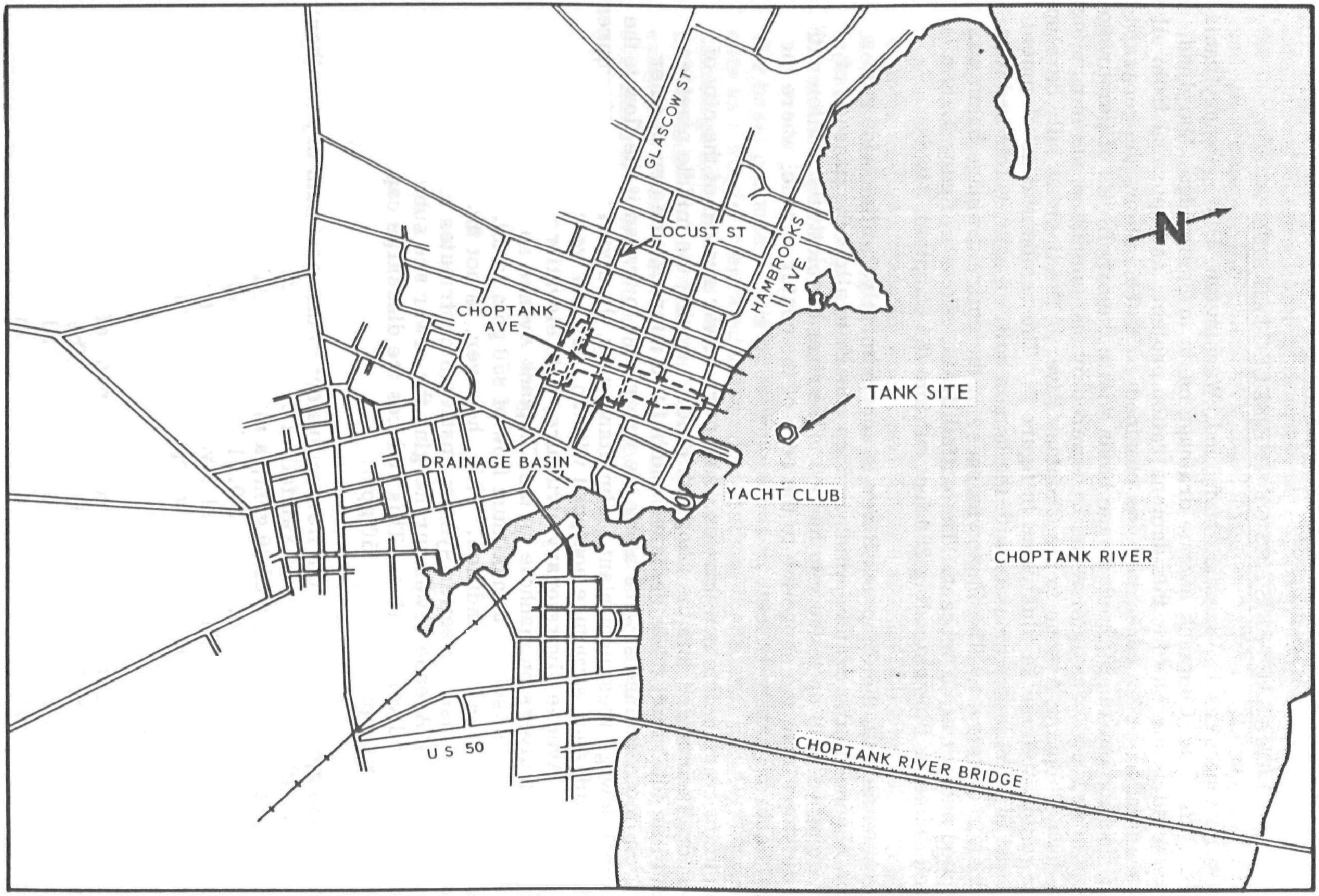


Figure 1. Geographic Location and Outline of Drainage Basin

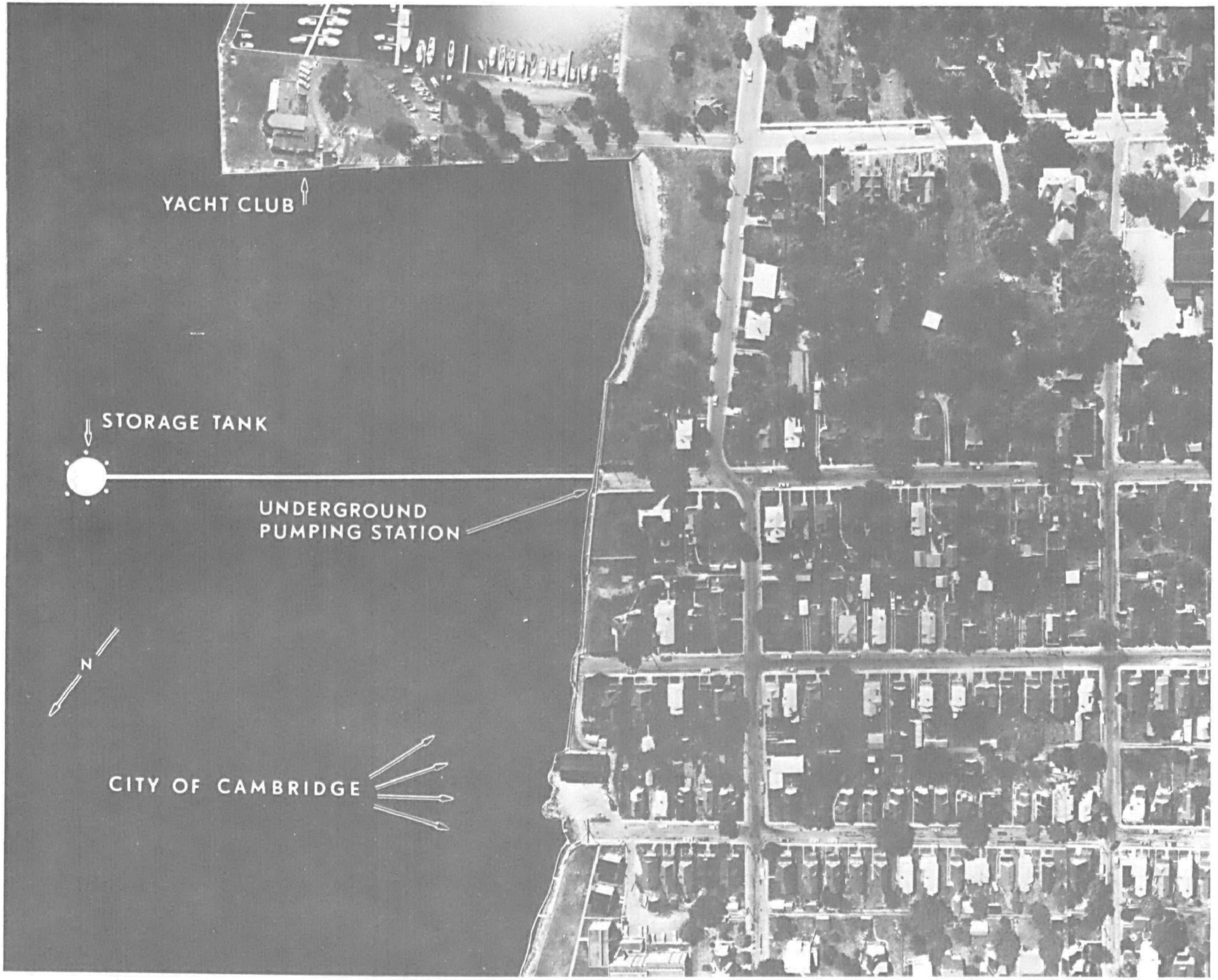


Figure 2. Aerial View, Lower Section Drainage Basin

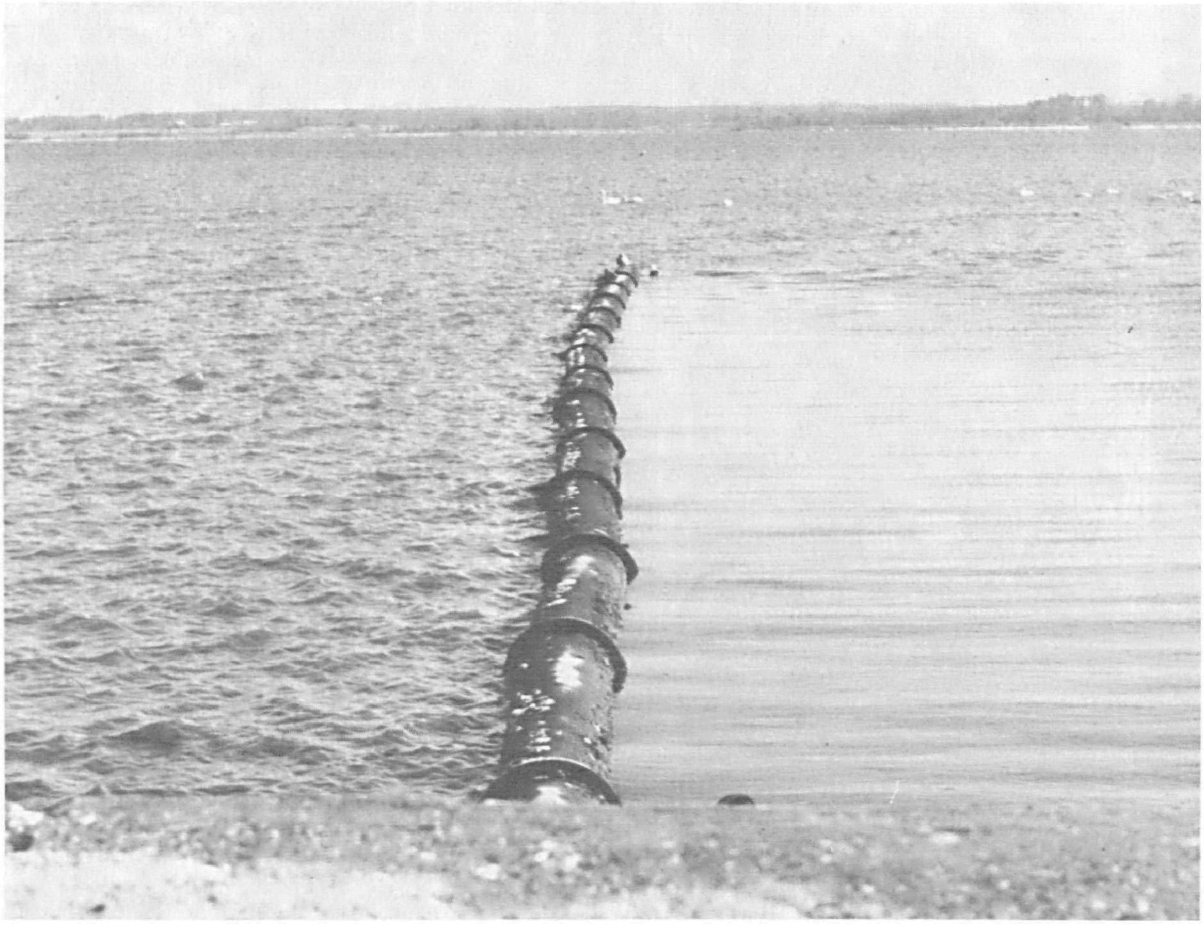


Figure 3. Choptank Avenue Combined Sewer Outfall

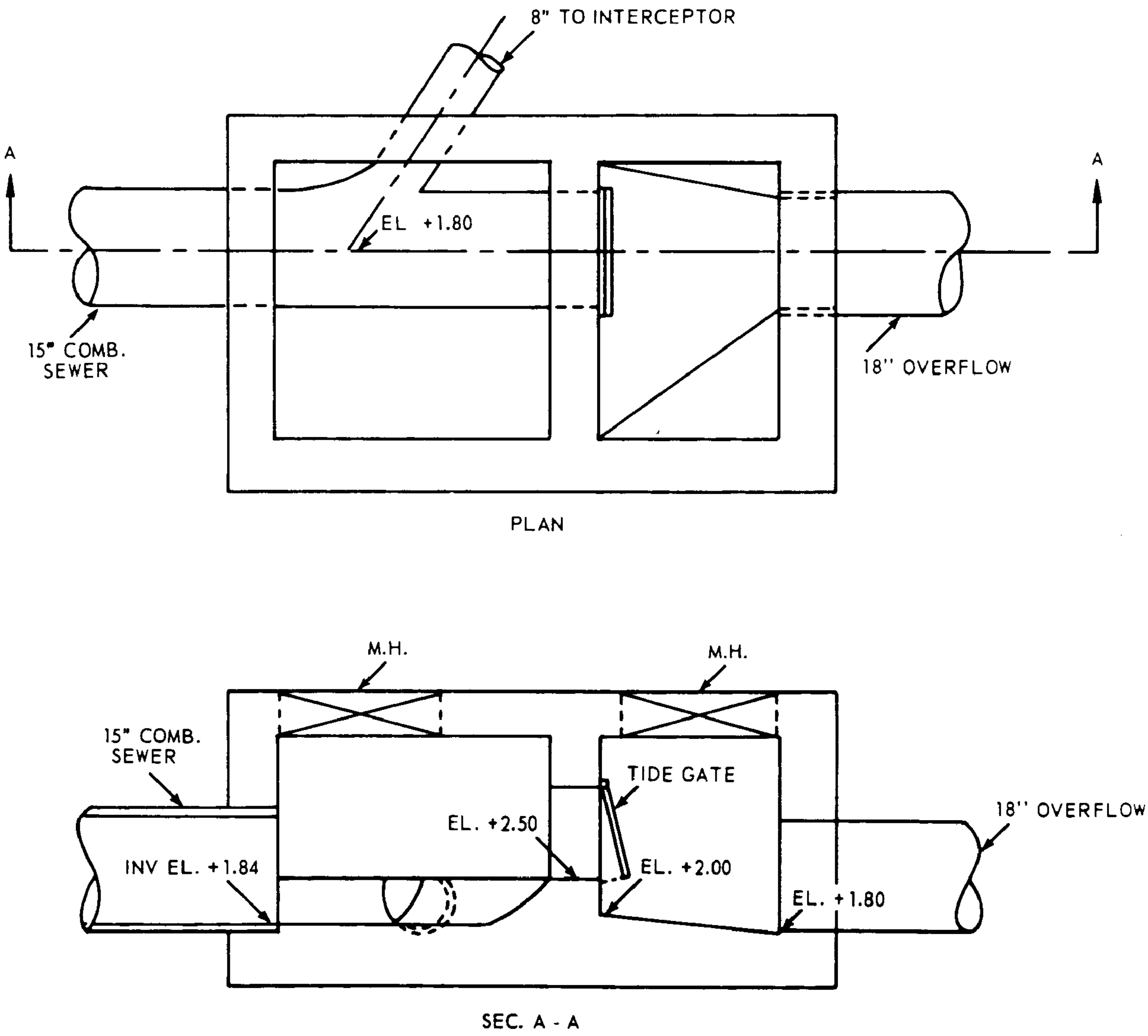


Figure 4. Choptank Avenue Diversionary Structure

Precipitation records were obtained from the Office of Hydrology, USWB. The following stations with reliable periods of record were considered:

Station	Daily	Hourly
Cambridge, Maryland	65 yrs.	---
Washington National	60 yrs.	25 yrs.
Baltimore, Maryland	60 yrs.	25 yrs.
Georgetown, Delaware	13 yrs.	7 yrs.
Leonardtown, Maryland	25 yrs.	12 yrs.

The criterion for selection of the station to be used for analysis of the hourly data was that it be one which most nearly resembled the Cambridge precipitation regime. Consequently, monthly precipitation totals were selected as a description of the regime and correlation coefficients were calculated between the Cambridge monthly totals and the totals for the other nearby stations with the following results:

Station	January	April	July	October
Washington National	0.88	0.92	0.22	0.66
Baltimore	0.81	0.86	0.54	0.33
Georgetown	0.78	0.86	0.59	0.44
Leonardtown	0.90	0.81	0.68	0.78

Leonardtown precipitation showed the highest overall correlation with that at Cambridge, and therefore it was used to evaluate the hourly precipitation data for purposes of relating it to Cambridge.

Ten years of hourly records of Leonardtown data have been analyzed on the basis of storms. A storm is hereafter defined as a period of precipitation during which the total accumulation is 0.10 inches or greater and during which the intensity of precipitation was greater than 0.05 inches per hour for at least one hour. A single storm begins when the intensity is greater than 0.01 inches per hour and ends when the precipitation ceases.

By this definition, the average number of storms will be less than the average number of days per year that precipitation is recorded. In this case, for the last 18 years of record, Cambridge, Maryland, has recorded precipitation on 88 days a year. However, based on the above definition of a storm, there are only an average of 55 storms per year. Since the Hambrooks interceptor, shown in figure 7, will accept the runoff flow from a storm of 0.05 inches per hour or less, by the above definition the overflows resulting from such a storm were considered. In accordance with this criteria there will be an average of 55 overflows per year.

The precipitation for each storm was idealized, and the resultant intensity duration curve for 10 years of Leonardtown data is presented in figure 5. Using the results of this curve and based on a runoff of 54 gpm/0.01 in./hr of rainfall and the interception of the first 0.05 in./hr of rainfall, the anticipated overflow volumes are tabulated and totalized in Table I and shown graphically in figure 6 for storms up to the 18 hours duration. The tabulation and figure indicated that on the averate 19 storms per year will generate an overflow of 200,000 gallons or more. Only the overflow in excess of 200,000 gallons would have to be bypassed around the pilot plant. The amount of runoff, 54 gpm/0.01 in./hr of rainfall, was determined through the use of the "rational formula" (Q=CIA). As previously mentioned a value for the coefficient of imperviousness for the basin was set at 0.6.

On the average, 36 storms per year would be completely contained within a 200,000 gallon container. During an average year an underwater storage system of this size for the Choptank Avenue drainage basin would have been subjected to 55 operational cycles. Of these, 19 overflow conditions would have equaled or exceeded the container: however, the sewer system would be fairly well flushed out and any additional overflow would be of weaker pollutional strength.

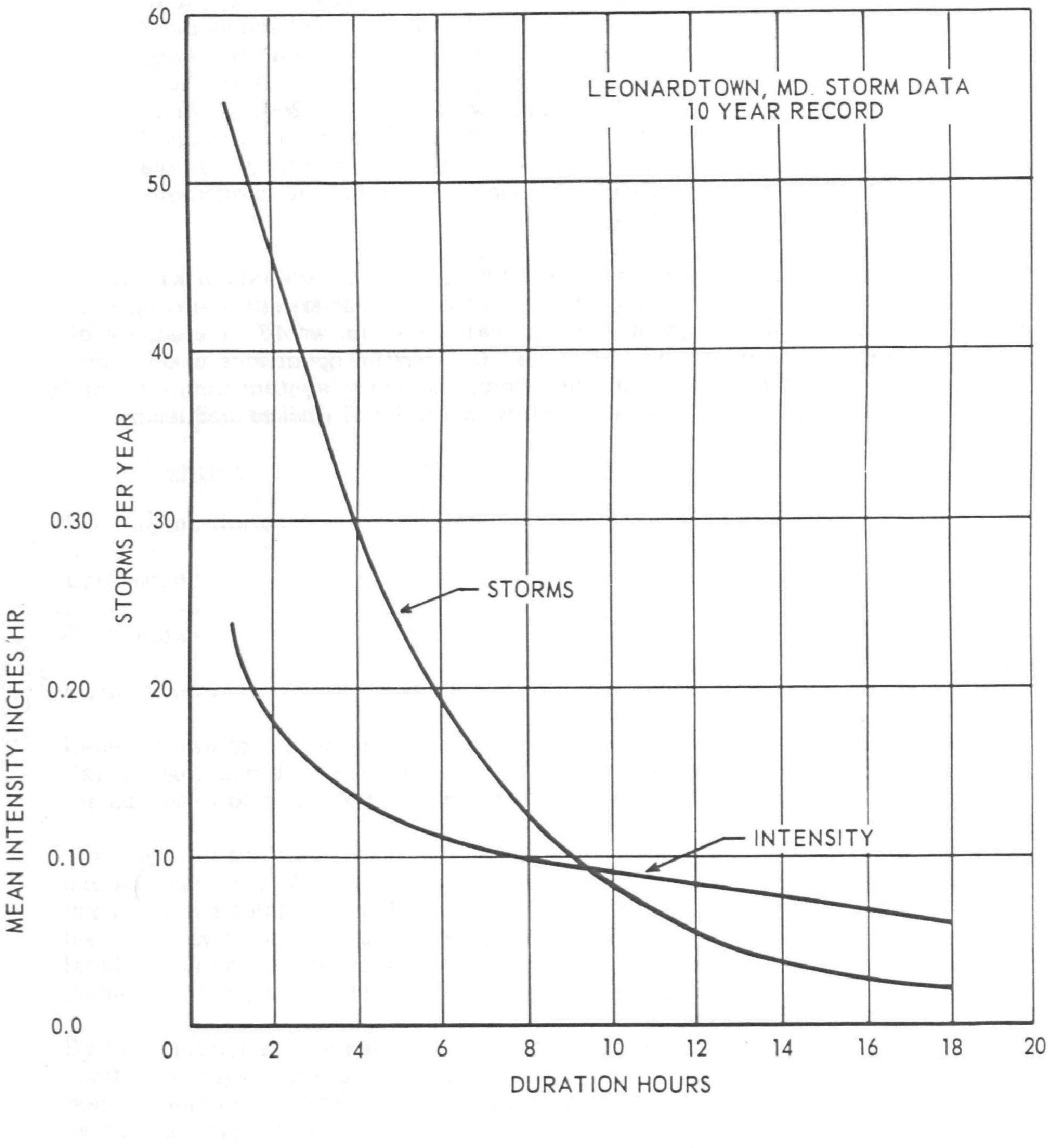


Figure 5. Rainfall Intensity Duration Curve

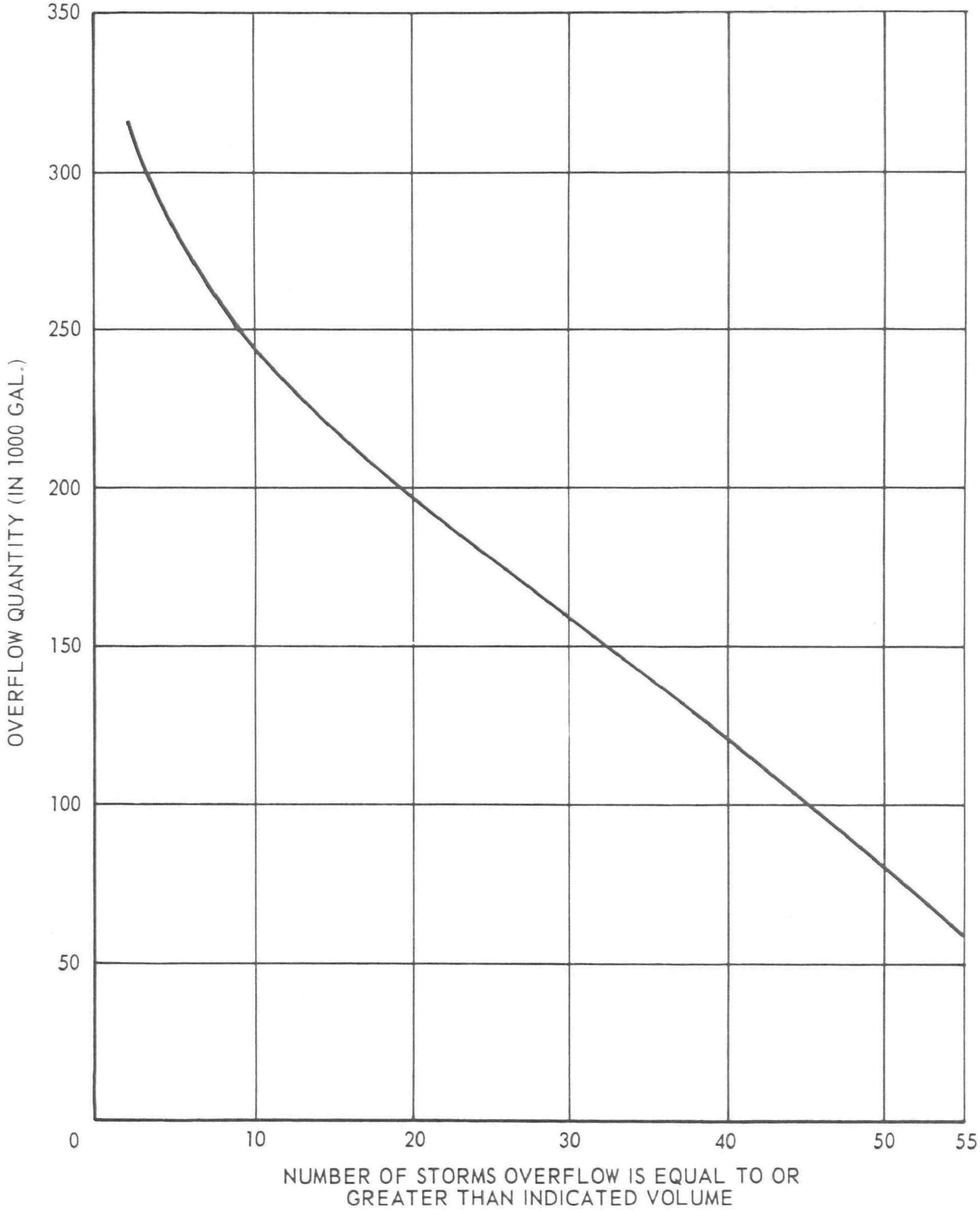


Figure 6. Estimated Overflow Volumes for Choptank Avenue

TABLE 1

ESTIMATED OVERFLOW VOLUMES
FOR
CHOPTANK AVENUE DRAINAGE BASIN

Period of Rainfall hr.	Rainfall Intensity in/hr.	Number of Storms/Yr	Estimated Gallons	Overflow Total to Hour Gallons
1	0.23	55	58,300	58,300
2	0.18	45	42,100	100,400
3	0.15	37	32,400	132,800
4	0.13	30	25,900	158,700
5	0.12	24	22,700	181,400
6	0.11	19	19,400	200,800
7	0.10	15	16,200	217,000
8	0.10	12	16,200	233,200
9	0.09	10	13,000	246,200
10	0.09	8	13,000	259,200
11	0.085	7	11,300	270,500
12	0.08	5	11,300	281,800
13	0.08	4	9,700	291,500
14	0.075	4	8,100	299,600
15	0.07	3	6,500	306,100
16	0.065	3	4,800	310,900
17	0.06	2	3,200	314,100
18	0.06	2	3,200	317,300

SECTION IV

DESIGN PHASE

The pilot plant for the underwater temporary storage of combined sewer overflow was designed to be compatible with a multiplicity of conditions for the particular tidal estuary demonstration site. The purpose of this pilot plant was to provide temporary storage for the excess flow from the 15 inch Choptank Avenue combined sewer, and later return it to the Hambrooks interceptor sewer when the treatment plant was prepared to process it. The key item of the pilot plant was a flexible storage module which was to be located in the Choptank River some 1300 feet offshore where the river was 8 feet deep at mean low tide. When not in use, the storage module would not protrude more than one foot above the existing natural river bottom. The items listed below were required to support the storage module and to transfer the overflow water to it and back to the sewer system.

* The upstream flow meter
* The bar screen, overflow meter, and diversion manholes
* The wet well and pumping station
* The transfer pipe
* The storage module
* The return flow line
* The control trailer

These items are shown on the schematic diagram presented in figure 7.

These individual items can be grouped into two basic installations, the onshore facilities and the offshore facilities. The onshore facilities consist of all the items except the transfer pipe and the storage module. The offshore facilities include the transfer pipe and the storage module, which is in a direct line with Choptank Avenue. The transfer pipe interconnected the onshore and offshore facilities.

A description of the pilot plant is given in the subsequent paragraphs. References are made to appropriate Melpar drawings which are on file at the Federal Water Quality Administration, Storm and Combined Sewer Pollution Control Branch.

The Upstream Flow Meter

A 15 inch Leopold Lagco flume (F.B. Leopold Co., Inc., Zelienople, Pa.) and

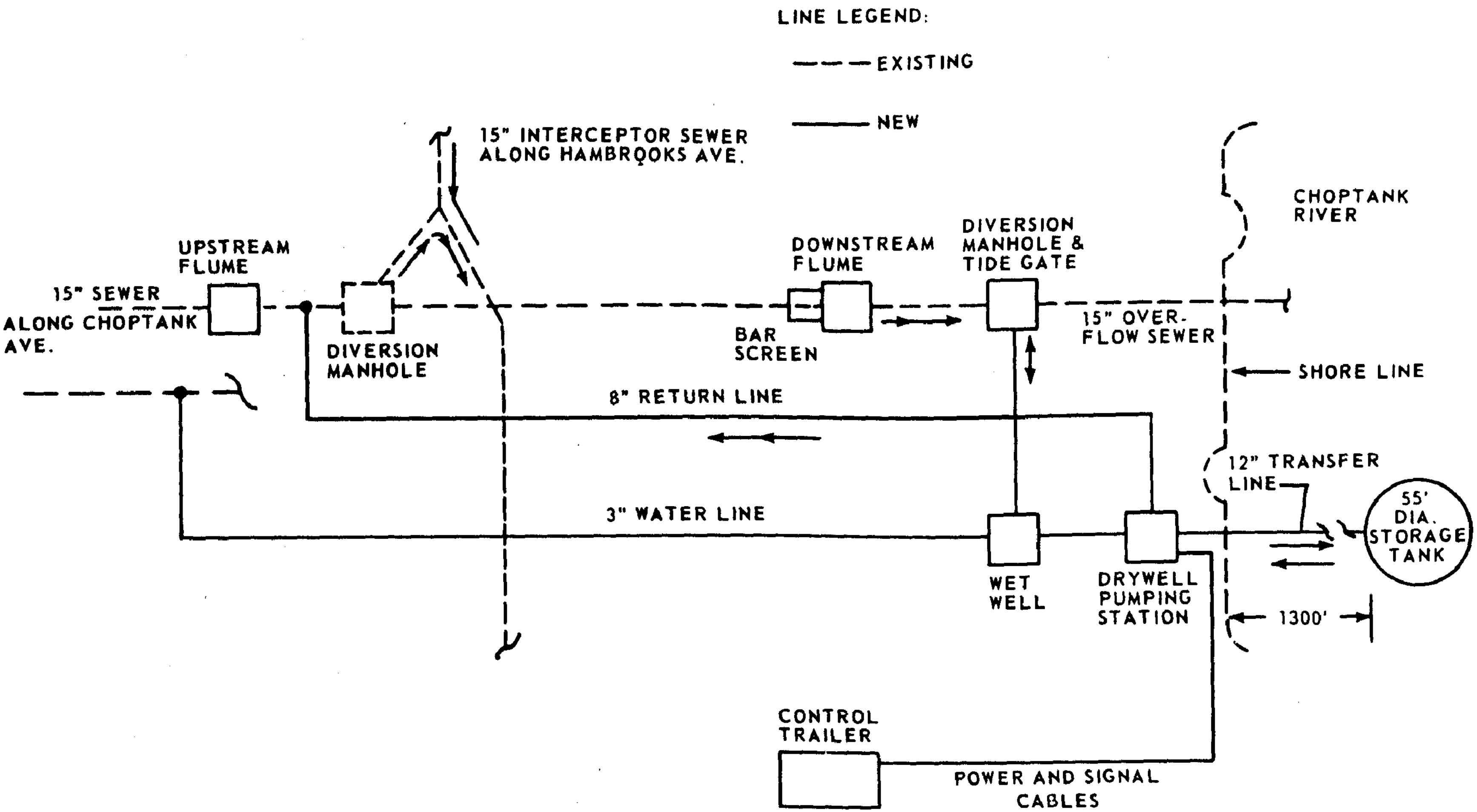


Figure 7. Schematic Diagram of Combined Sewer Underwater Storage Plant

a Stevens Type F water-level recorder (Leopold & Stevens Instruments, Inc., Portland, Ore.) were installed in a test manhole located upstream of the existing city diversion manhole. Figure 8 shows the flume in place, looking downstream. The flume and recorder monitored the total flow generated from the drainage basin before any flow was diverted to the interceptor or to the overflow line.

The flow chart was intended to record the daily flow of sewage and storm water from the drainage basin. Sampling of the flow at this location was intended to allow for the characterization of the entire flow generated in the drainage basin. Surcharging conditions in the sewer eventually eliminated the use of this test manhole.

Bar Screen, Overflow Meter, and Diversion Manhole Bar Screen

A bar screen (Melpar Dwg. No. R453025) was constructed upstream of the overflow meter and diversion manhole. This was a hand cleaned screen used to prevent large objects from clogging the flume and from entering the pilot plant and endangering either the pumps or the storage container's recirculation system. The size and nature of the drainage basin and sewer system did not warrant installation of a mechanically cleaned bar screen or a comminutor.

Overflow Meter

An 18 inch Leopold Lagco flume (F. B. Leopold Co., Inc., Zelienople, Pa.) and a flow transmitter (Style ML, Badger Meter Mfg. Co., Milwaukee, Wisc.) were installed immediately downstream of the bar screen. A receiver and recorder (Style X701E, Badger Meter Mfg. Co., Milwaukee, Wisc.) for the transmitter was located in the remote control panel of the trailer. This flow meter system (Melpar Dwg. No. R453025) which was placed ahead of the diversion structure, monitored all of the overflow water from the drainage basin regardless of whether or not it was taken into the pilot plant.

Diversion Manhole

A new diversion manhole (Melpar Dwg. No. R453136) was constructed just ahead of the seawall. The purpose of this structure was to divert the overflow water from existing outfall sewer into a supply tank (wet well), and then to the pumping and storage facilities. The existing outfall sewer that extended beyond the new diversion manhole served as the pilot plant bypass or overflow line. When the pilot plant was not in operation or had reached its storage capacity, the gate on the diversion line was manually closed and the water flowing in the overflow sewer was directed out the existing outfall pipe. Under plant bypass conditions the overflow sewer functioned in the same fashion as in normal operation.

A tide gate was installed on the downstream side of the bypass port of this diversion structure to ensure that river water would not flow through the outfall pipe and into the pilot plant.

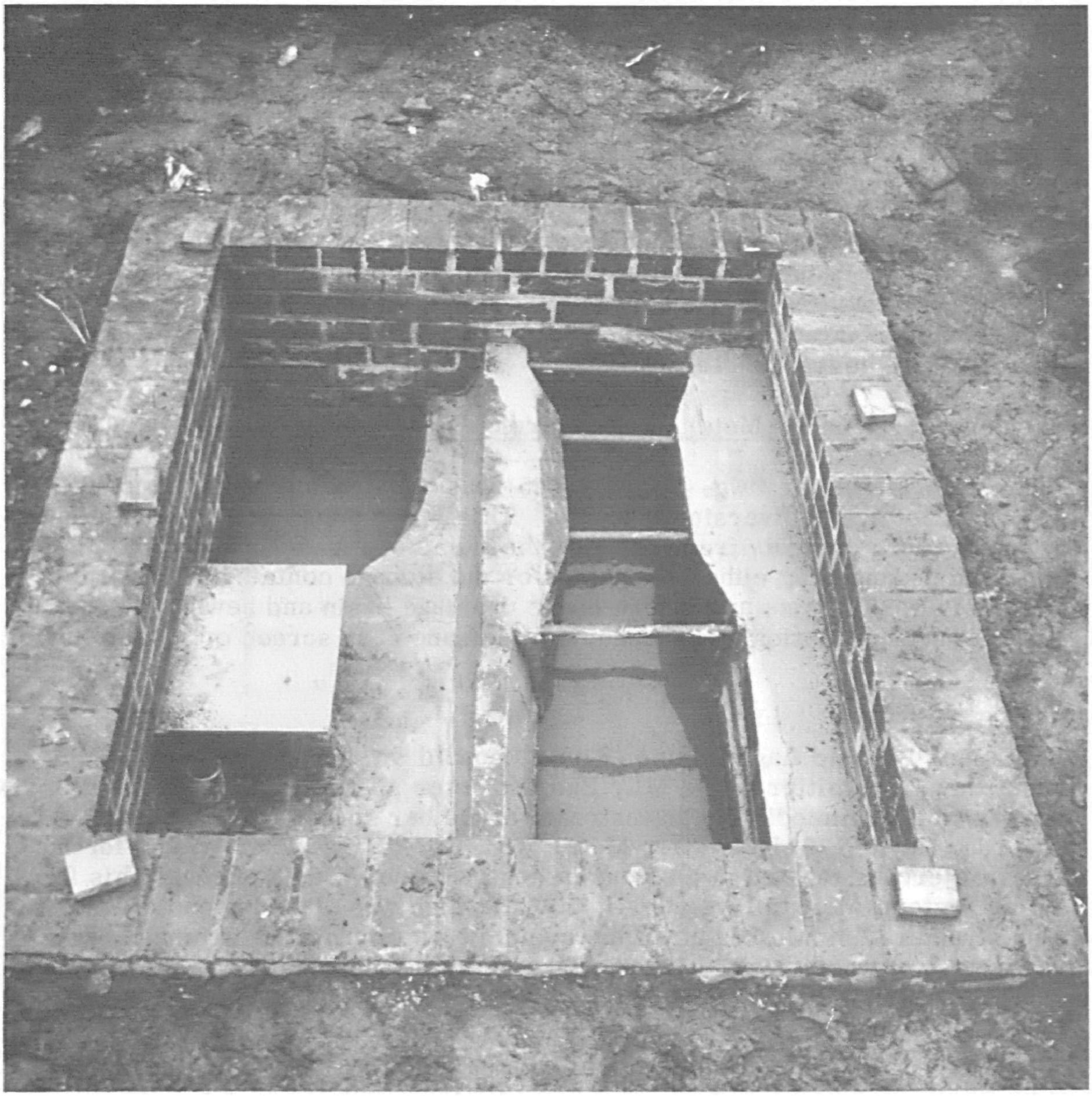


Figure 8. Upstream Flume Manhole

The Wet Well and Pumping Station

The water directed to the pilot plant flowed from the diversion manhole to a wet well just ahead of the pumping facilities. The pumping station was designed to function as the master unit for flow into and out of the storage container and other parts of the pilot plant. These units were built by Schmieg, Richmond, Va., reference Melpar drawing Nos. R452995 and R453034.

The pumping station consisted of two, two-speed (600-1000 gpm) pumps, pump controllers, and the necessary valving. The maximum full flow capacity of the pumping station was 2000 gpm.

On the flow to the storage container cycle, the pumps operated on automatic control (with manual override provisions) that programmed the operation of the pumping stages to match the rate of flow into the wet well from the sewer system. This was accomplished by five liquid level sensors (Type ENP-10, Flygt Corp., Stamford, Conn.) in the wet well. The pumps were stepped up and down automatically as the level in the wet well fluctuated, and they were shut off when the flow into the wet well ceased.

The valving and piping in the pumping station provides for the suction side of the pumps to be connected to the transfer line to the storage container. This also allowed the pumps to function as discharge pumps and to pump the water from the container back to the sewer system. During this operational cycle, the pumps discharged through an eight-inch cast iron pipe that interconnected the pumping station to the interceptor sewer. The normal return flow operation was 600 gpm.

A 3-inch fresh water supply line, with approved backflow devices, was installed on the top of the wet well. This water line provided a source of clean water that was used in a clean-out and flushing operation of the storage container. It also served as a source of water to operate the pilot plant during periods when there was no storm flow.

The Transfer Pipe

The transfer pipe served as the connecting link between the onshore facilities and the storage module. A single 12-inch cast iron pipe was installed in a trench along the river bottom. The power line running to the recirculation pump and the leads running to the pressure sensing elements and warning lamps were laid on top of the pipe. The transfer pipe was used to transfer the liquid wastes to and from the storage tank.

The Storage Module

The storage module included the 200,000 gallon (nominal capacity) container, the inlet and outlet piping, the flushing system, the anchorage system, and the pressure sensing device. The storage container, shown under preliminary test in figure 9, consisted of a lower half of epoxy coated steel and an upper half of neoprene coated nylon fabric material. In a collapsed condition the flexible top folded into the steel bottom section. The storage container had a circular con-

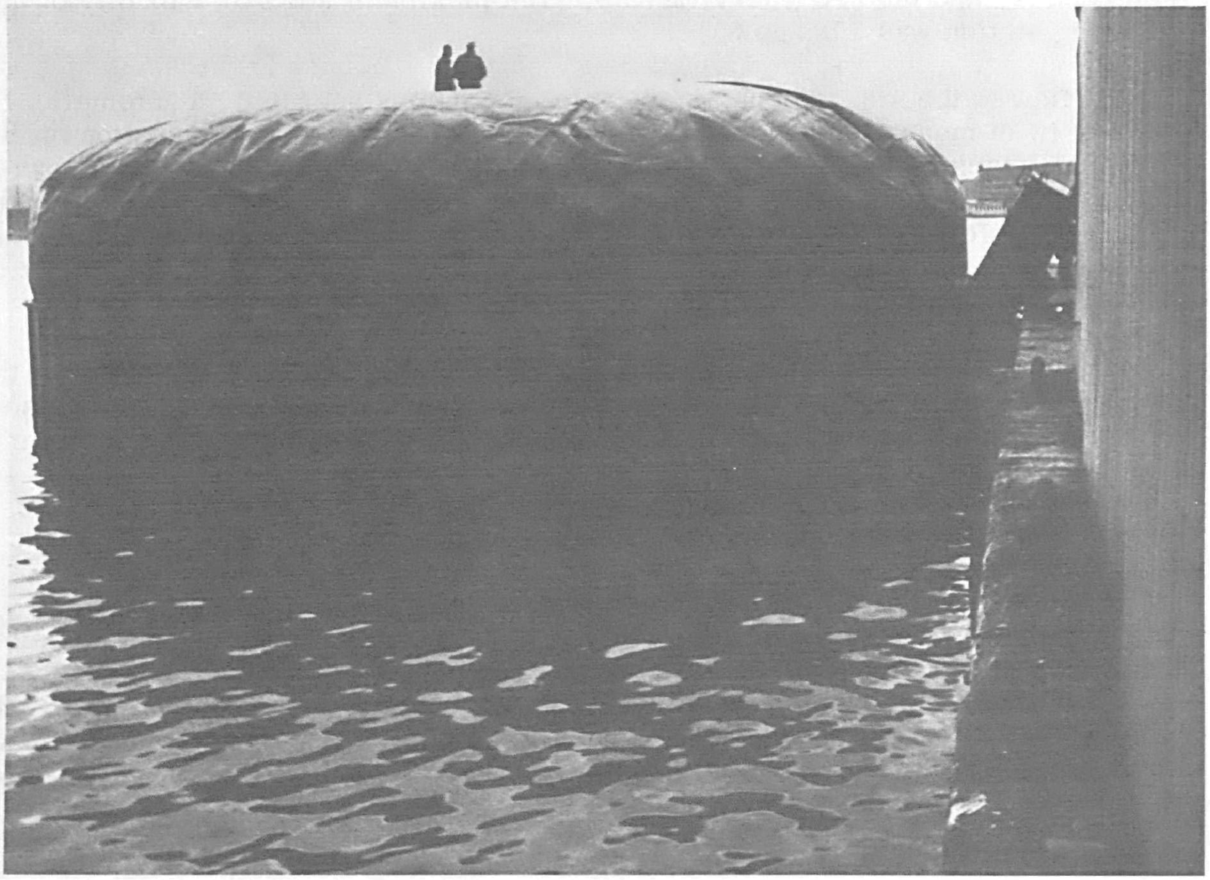


Figure 9. Underwater Storage Container

figuration with a 55 foot diameter and a designed height of 14 feet including both the steel bottom and fabric top. The steel tank was constructed by American Welding Co. Inc., Baltimore, Maryland. The flexible neoprene coated fabric cover was fabricated by Uniroyal Plastics Products, Mishawaka, Indiana. After American Welding Co. installed the flexible cover to the steel tank, the storage module was filled with air by a large air compressor to check for air leaks. See figure 9. Upon completion of the leak testing, the storage container was towed by tugboat from the Baltimore Harbor via the Chesapeake Bay to Cambridge, Md. The storage container was placed in the Choptank River approximately 1300 feet out from shore where the depth was 8 feet at mean low water. The tank rested in a depression excavated in the bottom of the river to a depth of seven feet below the natural river bottom. The elevation of the connection between the steel bottom and the flexible top therefore corresponded to the elevation of the existing river bottom. The flexible top when filled raised an additional seven feet above the top of the steel tank.

The recirculation system consisted of a recirculation pump (Crane-Deming, Type 7360, Burnet Engineering Co., Richmond, Va.) mounted to the steel bottom with necessary intake and discharge piping. The suction line of the recirculation pump was connected to the drain on the storage tank. The discharge header consisted of a pipe running around the entire perimeter of the tank at the bottom of the steel portion of the tank with fifteen circulating tank eductors (Penberthy, Prophestown, Illinois) equally spaced and mounted on the pipe. The recirculating water aided suspension of the solids carried into the tank.

Access to the inside of the tank was gained through a manhole port on the side of the steel tank with a hatch placed at the surface of the river bottom. This provided an access near the bottom of the tank so the position of the flexible top would not be a hindrance or problem.

An air bleed valve on the top of the flexible portion of the storage module provided a means of escape for any gases that might have become trapped in the container. If not released these gases would have reduced the capacity of the tank or resulted in excessive positive pressure on the inside of the tank.

A differential pressure sensing element was installed on the storage tank to monitor the location of the top when in a near full condition. The differential reading between the two levels of the flexible cover would indicate how high the fabric top raised off the steel bottom. It was necessary to measure differential pressure to compensate for the fluctuations in water level due to tides and storm conditions.

<u>The Return Flow Line</u>

The return flow line connected the pumping station with the city diversion manhole for the purpose of returning the water from the pilot plant back to the interceptor sewer. This line discharged into the Choptank Avenue sewer ahead of the original diversion line. By returning the flow upstream of this point, the diversion manhole functioned as a throttle on the return flow to the system. Return flow in excess of the diversion capacity overflowed the existing weir and continued down through the outfall sewer into the wet well. The level monitor in the wet well indicated when the return pumping rate was in excess of the sewer capacity.

<u>The Control Trailer</u>

The control trailer (Coastal Model M-30, Brandywind, Md.) was used as a central location for the control of the pumping station operation and the housing of the meter readouts. An indication panel was installed in the trailer for visual display of the events occurring in the pumping station and wet well. The larger section of the trailer was modified for laboratory space and equipped with the necessary laboratory supplies to perform the chemical analyses of combined sewer overflow samples. The trailer was located nearby in the city park.

SECTION V

CONSTRUCTION PHASE

The objective of this phase was to construct one complete pilot plant for control of water pollution from the Choptank Avenue outfall in the Choptank River at Cambridge, Maryland. The combined storm and sewage overflow from the Choptank Avenue combined sewer was intercepted and stored in a collapsible storage tank located in the river. A pumping station and other necessary diversion and control devices were located onshore at the foot of Choptank Avenue. Primarily, the construction was divided in two parts.

* Offshore Construction

* Onshore Construction

The potential subcontractors were allowed to submit bids on either the offshore or onshore construction or both. The successful bidders were Smith Brothers Pile Driving, Inc., (Galesville, Maryland) for the offshore construction and Norris E. Taylor Contractors, Inc. (Easton, Maryland) for the onshore construction. The work began in September 1968 and was completed in April 1969.

Offshore Construction

The offshore construction consisted of the entire offshore portion of the pilot plant, including:

* Installation of the storage tank

* Installation of the 12-inch cast iron pipe line

* Installation of the electrical equipment

* Installation of the piles and wire rope

Excavation of approximately 1400 cubic yards of the river bottom was required to prepare the offshore site for the storage tank. This step included excavation of the river bottom to the generally required configuration, and placement of the excavated material on the Great Marsh peninsula (about 2500 feet from the site) as fill material. The excavation included a trench in which the transfer pipe and electrical cables were buried near the natural river bottom elevation.

Following the excavation, guide piles were drived around the sides of the excavation. These piles protruded above the surface of the water and were used to locate the storage tank over the excavated area. After the guide piles were in place, the tank was towed into place, sunk, and secured to the guide pilings. After the tank was lowered to the bottom and secured, additional piles were driven around the outer edge and the storage module was secured to these pilings. A total of 6 fifty foot piles were installed.

After the storage tank was installed, the transfer pipe was connected to the tank. In conjunction with this the power cables for the circulation pump and navigation lamps were connected to one of the piles. The cables were terminated in a watertight junction box located on the piling.

A 3/8-inch galvanized wire rope was installed around the periphery of the tank and supported by the piles. The rope was used to support the electric cable that supplied power to the six navigation lamps which were mounted on top of each pile.

Onshore Construction

The onshore construction consisted of the entire onshore portion of the pilot plant. This included the following major subsystems:

* Installation of pumping station and wet well
* Construction of bar screen, flume and transmitter, and diversion manholes
* Installation of 3-inch fresh water line
* Installation of 8-inch return line
* Provisions for necessary utility connections to control trailer
* Electrical installation

The pumping station and wet well were ordered from a manufacturer as package units and were delivered to the site as self contained working modules. The installation of these units required the sheeting and excavation of the proposed area, the pouring of a concrete foundation slab, the installation of the units to the concrete slab, and the connection of the inlet and outlet piping and power lines. Figure 10 shows the construction for the installation of the pumping station and wet well.

The other onshore facilities were built or installed in place. These facilities included the bar screen, flume and transmitter, and diversion manholes. These structures were built independently of the other and the piping connected as they were completed. Figure 11 shows the bar screen and flume manholes being constructed.

After the installation of the pumping station and wet well, the pumping station and the city diversion manhole were interconnected with an eight-inch cast iron pipe line, which served as the return line for the stored overflow waste-water. A three-inch fresh water line was installed between the city's four-inch main and the wet well. The connection of the line to the wet well included a shut-off valve, a city water meter, and a doucle check valve to prevent back flow.

All excavated areas were back filled and the street paved. Figure 12 shows the street after the completion of all construction.

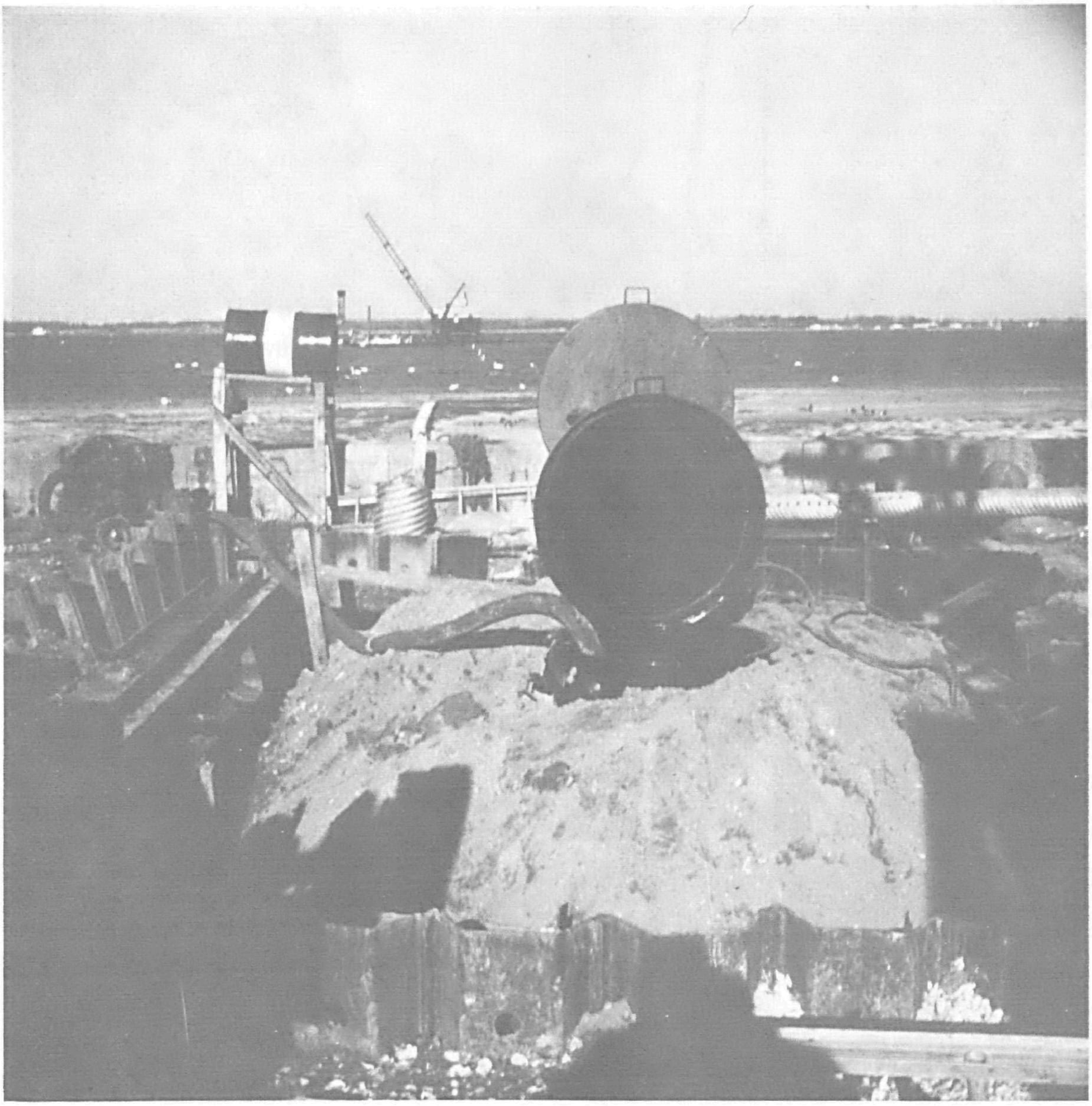


Figure 10. Installation of the Pumping Station and Wet Well

Figure 11. Construction of Bar Screen and Flume Manholes

Figure 12. View of Site After Installation of Facility

Summary

Icy conditions on the river, tidal conditions, and high winds presented the main problems encountered by the subcontractor during the installation of the offshore portion of the facility. Otherwise, the offshore construction continued efficiently and smoothly.

The onshore contractor experienced many difficult situations other than the problem of inclement weather. Problems of the type encountered will occur if the appropriate construction procedures are not properly enforced on the project. For example, during the initial stages of construction, there was the need for full time on-site supervision and better public relations. The excavation for the installation of the pumping station and wet well collapsed and undermined a large area of the street, sidewalks, curbs, and adjacent portions of private properties. The primary reason for the collapse was the weak and insufficient bracing used to support the interlocking steel sheeting. The contractor's small type pumps proved to be inadequate for keeping the excavation area pumped clear of the infiltrating tide water. Eventually adequate pumps and a well point system of sufficient capacity were installed to accomplish the task of dewatering the area of excavation.

SECTION VI

OPERATIONAL PHASE

Operation of Pilot Plant

The operation of the pilot plant was characterized by two basic modes of operation; (1) the normal routine for a rainfall generated overflow condition and (2) a test routine for an artificially generated overflow condition. The latter was necessary to ensure that a sufficient number of mechanical tests could be performed on the pilot plant during the contract period.

Normal Mode of Operation

After each use of the pilot plant, it was returned to a normal mode condition in anticipation of the next storm. This involved opening the sluice gate into the pilot plant at the new diversion manhole, if it was closed to bypass part of the previous storm. The volume of liquid remaining in the storage container during the previous cycle was calculated after each use of the pilot plant. The valves in the pumping station were properly opened and/or closed to direct the pumped flow out to the underwater storage container. The valving arrangement for pumping in and out of the storage container is shown in figure 13.

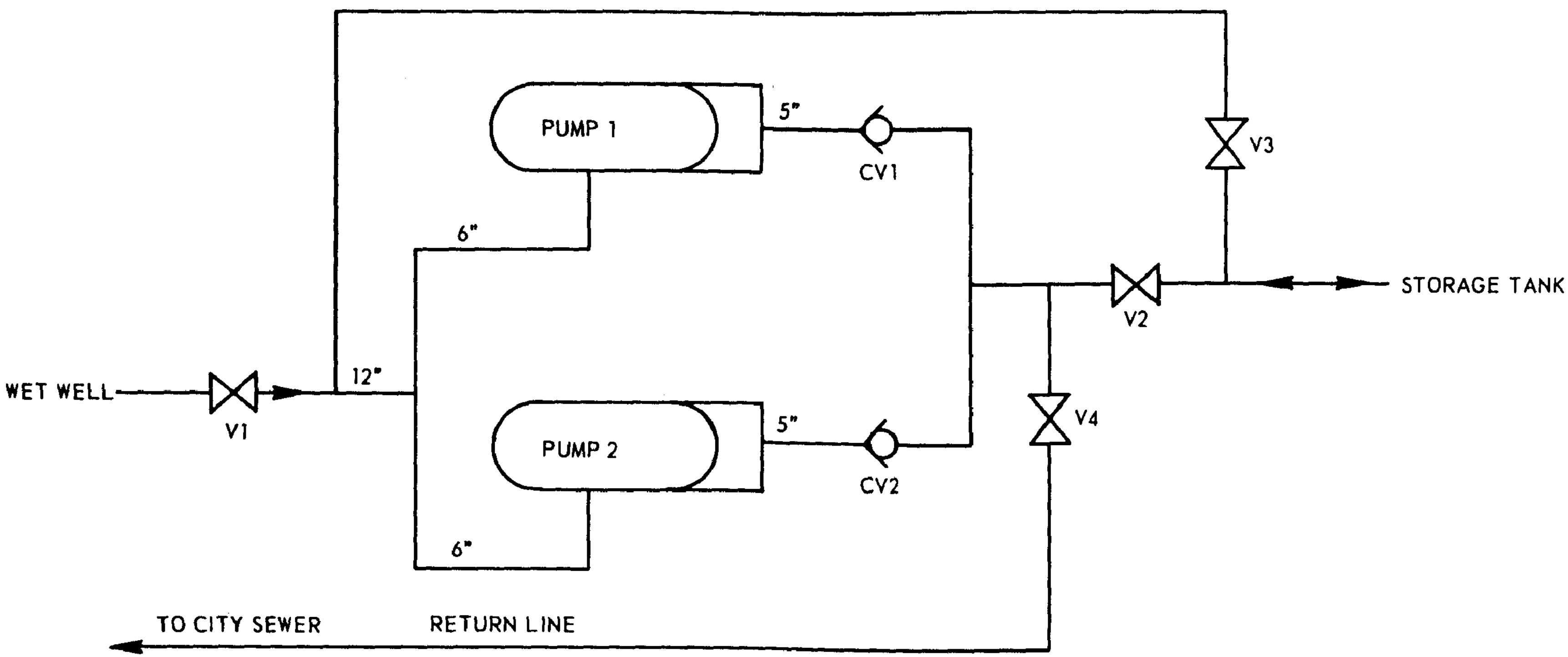
The underwater storage system was now ready to receive any overflow, be it rainfall runoff or just normal surcharging of an overloaded sewer system. When the rain started and the flow in the Choptank Avenue sewer exceeded the height of the diversion dam, the water overflowed the dam and flowed down the outlet pipe toward the Choptank River. The overflow liquid next passed through a manually cleaned bar screen used to protect all downstream units from any large objects. As the flow passed through the Lagco flume it was monitored totally and as a function of time. The flow next came to the new diversion manhole where it was directed away from the overflow sewer to the pilot plant. The flow continued by gravity from the diversion manhole into the wet well.

The wet well contained five mercury switches located at uniform increments of height above the bottom of the well. The switches were enclosed in waterproof weighted bags. These bags were so designed that they changed their attitude in a predetermined manner as water rose up under them. As this occurred, the mercury switch was actuated, signaling that a particular water level had been attained. When any or all of these switches had been actuated, corresponding indicator lamps were illuminated on both the pumping station and trailer control panels; in the automatic mode, they also provided the pump control.

When the pump controls were set for automatic operation, the pumps operated as follows:

* Switch No. 1 (lowest) closed, no pumping action (standby/cutoff)

* Switch No. 2 closed, pump No. 1 starts on low speed (600 gpm)



MODE 1: WET WELL TO STORAGE TANK

V_1 = OPEN
V_2 = OPEN
V_3 = CLOSED
V_4 = CLOSED

MODE 2: STORAGE TANK TO CITY SEWER

V_1 = CLOSED
V_2 = CLOSED
V_3 = OPEN
V_4 = OPEN

Figure 13. Valving in the Pumping Station

recording system is actuated

* Switch No. 3 closed, pump No. 1 starts on high speed (1000gpm)

* Switch No. 4 closed, pump No. 2 starts on low speed (1600gpm)

* Switch No. 5 closed, pump No. 2 starts on high speed (2000gpm)

This sequence was reversed after the runoff subsided and the liquid level in the wet well was being lowered.

As the water filled up the wet well, the level sensors activated the pumps to keep pace with the incoming flow up to the maximum capacity (2000gpm) of the facility. The flow rate into the storage tank from the pumps was sensed by a magnetic flow meter 1800 series (The Foxboro Co., Foxboro, Mass.); this information was transmitted to an associated Foxboro receiver and recorder (9650 series) remotely located in the trailer control panel. A timer was associated with the recorder that, in combination with a mechanical integrating device, determined the total volume flow. This total volume was indicated on a totalizer (a six place digital counter). Attached to this counter was a switch that was set to open when a predetermined total was reached. This switch was normally set to open at the nominal 200,000 gallon capacity of the storage tank. Flow in excess of this 2000gpm rate or in excess of storage tank capacity surged in the upstream sewer until its head exceeded the level of the river and tide at which time it would open the tide gate and discharge into the river. Under a full scale operation this would not occur unless a storm flow in excess of the design storm occurred. All of these events could occur with the station in an unattended condition.

An operator came on duty at the beginning of each storm to monitor the filling operation. These duties included ensuring shutdown if the storage module filled up or shutting off the pilot plant from the overflow sewer should a malfunction occur in the system. The operator placed the sampler in position during an overflow to take periodic samples of the incoming flow. The operator also was responsible for analyzing the samples, changing charts, placing the system in "stand-by" mode, and all other general maintenance.

At the conclusion of a storm event, the operator recorded the totalized flow measurements and prepared the pumping station flow pattern for returning the flow to the sewer system. The recirculation system on the storage module was turned on to keep the solid contents in suspension. After the stored contents were pumped back to the sewer system, the pilot plant was returned to a standby mode.

<u>Simulated Mode of Operation</u>

The simulated test runs followed the same operating procedure with the exception of start up.

To perform a simulated test run, the outlet sewer pipe in the overflow manhole at Hambrooks Avenue was sealed off with an inflatable plug. This diverted all upstream flow to back up and flow through the overflow line to the pilot plant. At no time during these simulated tests was the flow allowed to exceed the plant capacity and result in a discharge into the river. The simulated tests were sampled and monitored in the same manner as the naturally occurring overflows.

The system was debugged and ready for operation on June 19, 1969. Simulated tests were performed with cooperation from the City Water Department. Two simulated tests were performed between the hours of 9 a.m. to 1 p.m. and one simulated test was conducted between the hours of 12 midnight and 1:30 a.m.

After numerous complaints were received from the local residents because of low water pressure during the day and too much noise from the flowing hydrants during night hours, the simulated tests were ceased after July 14, 1969.

From this time, the storage module was operated only during periods of rainfall. The one exception to this was the final fill and draw cycle which was carried out to assist in a full inspection of the diaphragm portion of the storage module. After inspection of the module, the contents were pumped out and the system was permanently isolated from the sewer system.

Sampling of the Combined Sewer Overflow

The required volume per sample was 1020 ml to perform all required analyses. The standard SERCO Model NW-3 Automatic Sampler (Sanitary Engineering Research Co., Minneapolis, Minnesota) would collect approximately 330 ml of sample per bottle when operating with a five-foot lift and 26-inches Hg internal vacuum and an atmospheric pressure of 30-inches Hg. Therefore, it was necessary to fill four bottles at a time (1300 ml) for adequate sample volume. A newly designed and fabricated tripper arm was installed on the SERCO sampler. The tripper arm simultaneously actuated four sampling line switches.

The SERCO units have gearhead provisions for sampling at 5-, 10-, and 15-minute intervals, as well as at hourly intervals. The 15-minute gearhead was utilized for the tests to provide a sampling interval that would not over tax the field laboratory beyond its capacity.

Flushing of the Storage Tank

A three-inch water main was coupled to the wet well and was controlled by a gate valve outside the well. The pilot plant system was set for pumping into the storage tank. The wet well was filled four times with city water and emptied into the tank. This quantity of water was enough to provide sufficient coverage of the tank bottom and to allow the circulating pump to properly rinse the tank sides and bottom. The circulating pump was allowed to operate for several minutes and during the time the tank was pumped out. The water from the tank was returned through the 8-inch return line to the upstream side of the city diversion juncture.

Monitoring of the Rainfall

Precipitation in the Choptank Avenue drainage basin was measured and recorded with a Universal Rain Gauge (No. 5-780, Belfort Instrument Co., Baltimore, Maryland). The instrument was provided with an 8-day chart drive movement and a 12-inch dual-traverse chart. Interfacial instrumentation was not provided for the automatic synchronization of rainfall and runoff data.

Characterization of Combined Sewer Effluent

Samples of effluent from the combined sewer overflow and of sewage as it was pumped back from the storage tank to the city sewer system were analyzed to determine the following characteristics: pH, suspended solids, volatile suspended solids, settleable solids, 5-day biochemical oxygen demand (BOD), and chemical oxygen demand (COD). Analytical techniques employed for the analysis of the samples were in accordance with procedures outlined in the twelfth edition of "Standard Methods for the Examination of Water and Wastewater,"[1] except for the subsequent changes. The procedure for the pH determination was as set forth in "Standard Methods ... I Water," rather than the procedure of "Standard Methods ... III Wastewater," since it was performed at 25°C instead of at the 20°C temperature of the latter procedure. Under field conditions it was easier to maintain the 25°C temperature in the trailer laboratory. In addition, the use of the third (intermediate) buffer, pH = 6.86, provided a greater degree of accuracy and precision. Procedures for suspended solids and volatile suspended solids conformed exactly to "Standard Methods...," except that ignition was for 30 minutes, instead of 15-20 minutes, to more adequately assure the thoroughness with which the volatiles are driven off.

SECTION VII

DISCUSSION

General Testing

After the installation of the pilot plant facility, a debugging phase was employed to test the overall reliability of the facility. Initial tests indicated problems in the magnetic flow metering system and leaks in the offshore system.

The electrical system functioned properly except for the Foxboro flowmetering system that monitored the flow to and from the storage container. The receiver and recorder of the metering system was remotely located on the control panel in the trailer. After exhaustive testing indicated that the system would not function reliably under the remote conditions, the receiver and recorder unit and the associated timer were removed from the trailer control panel and installed in the pumping station, where satisfactory results were achieved.

The offshore system was tested for leaks by pumping a concentrated solution of uranine dye from the wet well into the system. Leaks were detected in the vicinity of the storage tank. An underwater inspection, performed by a diver, revealed three rips in the flexible cover. Each rip was located over an eductor, which protruded inward of the tank. Metal plates were fabricated and coated with gum rubber to conform with the configuration of the damaged area on the flexible rubber cover. These plates, which were installed by two scuba divers, were successfully used to sandwich the ripped areas of the rubber cover. Protective devices, that were fabricated of steel rods, were installed by divers around each eductor. Consequently, no further damage occurred to the rubber cover.

The full scale test runs started with a simulated run on June 19, 1969. During the next month and one-half, four rainfalls were captured and monitored and three additional simulated runs were performed. In addition the facility was mechanically used when water entered the overflow sewer from other than a rainfall or simulated test. During these periods no sampling was carried out. Prior to any simulated run, the tank was pumped relatively clear of its contents.

The characterization of the water pumped in each test run is presented in tables II - IX.

The data from the simulated runs cannot be considered in describing the character of combined sewer overflows because it is made up primarily of fresh water discharged from a fire hydrant. Although it is flushed across the street to the sewer it is concentrated across one path which becomes very clean in a hurry. The higher pH on these runs identifies the source of the water as treated water as opposed to sewage and rainfall runoff water.

One noteworthy observation of the simulated runs is that the first sample is

TABLE II

ANALYTICAL RESULTS OF OVERFLOW DISCHARGE OF JUNE 19, 1969 - SIMULATED OVERFLOW, TEST NO. 1

Time	Temp. °C	pH	Sett. Solids ml/l	T.S.S. mg/l	V.S.S. mg/l	COD mg/l	BOD mg/l
10:55	26.0	8.1	4.0	8.0	6.0	—	300.0
11:10	25.5	8.4	1.0	9.0	4.0	—	190.0
11:25	25.7	8.2	9.0	32.0	23.0	—	275.0
11:40	25.5	8.6	1.0	10.0	10.0	—	80.0
11:55	24.5	8.4	0.5	9.0	9.0	—	50.0
12:10	24.5	8.3	0.1	16.0	10.0	—	25.0

ANALYTICAL RESULTS OF STORED OVERFLOW

*RETURN FLOW JUNE 20, 1969

Sample No.							
1	24.0	7.9	Trace	1.0	1.0	—	100.0
2	24.0	7.9	Trace	1.0	1.0	—	90.0
3	24.0	7.9	Trace	1.0	1.0	—	150.0
4	24.0	7.9	Trace	6.0	4.0	—	130.0

*Return samples taken after every 20,000 gallons

TABLE III

ANALYTICAL RESULTS OF OVERFLOW DISCHARGE OF JUNE 24, 1969 - SIMULATED OVERFLOW, TEST NO. 2

Time	Temp. °C	pH	Sett. Solids ml/l	T.S.S. mg/l	V.S.S. mg/l	COD mg/l	BOD mg/l
11:05	28.0	8.0	9.0	40.0	36.0	568.0	205.0
11:27	26.0	8.2	0.4	4.0	4.0	37.0	15.0
11:42	26.0	8.3	0.4	2.5	2.5	80.0	15.0
11:50	26.0	8.2	0.4	2.5	2.5	43.0	10.0
12:05	26.0	8.2	0.2	2.0	2.0	25.0	15.0
12:20	25.0	8.3	0.1	1.5	1.5	31.0	20.0

ANALYTICAL RESULTS OF STORED OVERFLOW

*RETURN FLOW JUNE 24, 1969

Sample No.							
1	24.5	7.6	Trace	3.0	3.0	37.0	20.0
2	24.5	7.8	Trace	2.5	2.0	37.0	20.0

*Return samples taken: No. 1 after return of 15,700 gallons and No. 2 after 55,000 gallons

TABLE IV

ANALYTICAL RESULTS OF OVERFLOW DISCHARGE OF JUNE 26, 1969 - SIMULATED OVERFLOW, TEST NO. 3

Time	Temp. °C	pH	Sett. Solids ml/l	T.S.S. mg/l	V.S.S. mg/l	COD mg/l	BOD mg/l
11:15	26.5	8.6	21.0	10.5	90.0	728.0	—
11:25	24.5	8.3	3.5	7.0	7.0	31.0	12.0
11:35	25.5	8.3	0.2	6.5	6.5	31.0	20.0
11:50	24.0	8.4	0.2	5.0	4.5	31.0	22.0
12:05	24.5	8.4	0.1	2.5	2.5	12.0	20.0
12:20	23.5	8.3	0.1	2.8	4.0	31.0	23.0

ANALYTICAL RESULTS OF STORED OVERFLOW

*RETURN FLOW JUNE 27, 1969

Sample No.							
1	25.0	8.0	Trace	8.5	6.0	12.0	32.0
2	25.0	8.0	Trace	8.0	6.5	18.0	23.0

*Return samples taken: No. 1 after return of 50,000 gallons and No. 2 after 92,500 gallons

TABLE V

ANALYTICAL RESULTS OF OVERFLOW DISCHARGE OF JULY 5, 1969 - NATURAL OVERFLOW, TEST NO. 4

Time*	Temp. °C	pH	Sett. Solids ml/l	T.S.S. mg/l	V.S.S. mg/l	COD mg/l	BOD mg/l
-	24.5	7.0	3.0	13.5	12.5	446.0	—
-	24.5	7.3	0.8	9.5	9.5	217.0	105.0
-	25.2	7.4	1.3	7.0	5.0	132.0	55.0

*Samples taken at 15 minute intervals

ANALYTICAL RESULTS OF STORED OVERFLOW

*RETURN FLOW JULY 7, 1969

Sample No.							
1	27.0	7.4	1.0	15.5	9.5	157.0	120.0
2	27.0	7.4	2.5	20.0	14.5	290.0	105.0

*Return samples taken No. 1 after return of 3,000 gallons and No. 2 after 6,000 gallons.

TABLE VI

ANALYTICAL RESULTS OF OVERFLOW DISCHARGE OF JULY 7, 1969 - NATURAL OVERFLOW, TEST NO. 5

Time	Temp. °C	pH	Sett. Solids ml/l	T.S.S. mg/l	V.S.S. mg/l	COD mg/l	BOD mg/l
11:25	25.5	7.4	1.5	1.0	1.0	120.0	280.0
11:40	26.0	7.4	1.2	4.5	4.5	102.0	220.0
11:55	25.5	7.4	0.7	8.5	1.0	102.0	215.0
12:10	24.5	7.4	0.5	6.5	4.5	84.0	200.0
12:25	25.5	7.5	0.3	4.0	4.0	96.0	195.0
12:40	26.0	7.7	Trace	4.5	4.5	108.0	235.0

ANALYTICAL RESULTS OF STORED OVERFLOW

RETURN FLOW JULY 8, 1969

Sample No.							
1	26.0	7.0	0.5	21.0	12.0	147.0	235.0
2	26.0	7.0	0.5	15.0	10.5	235.0	215.0

TABLE VII

ANALYTICAL RESULTS OVERFLOW DISCHARGE OF JULY 10, 1969 - SIMULATED OVERFLOW, TEST NO. 6

Time	Temp. °C	pH	Sett. Solids ml/l	T.S.S. mg/l	V.S.S. mg/l	COD mg/l	BOD mg/l
0039	25.0	7.8	28.0	40.0	37.0	200.0	—
0054	23.5	8.1	0.2	7.0	7.0	40.0	—
0109	23.0	8.2	0.1	6.5	6.5	17.0	35.0
0124	23.0	8.2	Trace	5.0	5.0	17.0	—

ANALYTICAL RESULTS OF STORED OVERFLOW

*RETURN FLOW JULY 10, 1969

Sample No.							
1	25.0	7.7	1.0	26.0	13.0	130.0	—
2	25.0	7.7	Trace	15.5	11.0	40.0	40.0

*Samples taken: No. 1 after return of 17,000 gallons and No. 2 after 35,000 gallons

TABLE VIII

ANALYTICAL RESULTS OF OVERFLOW DISCHARGE OF JULY 12, 1969 - NATURAL OVERFLOW, TEST NO. 7

Time	Temp. °C	pH	Sett. Solids ml/l	T.S.S. mg/l	V.S.S. mg/l	COD mg/l	BOD mg/l
	25.5	6.9	1.0	8.5	6.5	71.0	—
	26.0	6.9	6.0	14.0	10.5	118.0	90.0
	25.0	7.2	12.0	7.0	2.0	71.0	65.0
	24.0	6.9	2.0	5.0	5.0	82.0	60.0
	24.0	7.1	2.0	5.0	4.5	53.0	45.0
	24.0	7.3	0.1	2.5	2.5	71.0	60.0

RETURN FLOW - NO RETURN SAMPLES

TABLE IX

ANALYTICAL RESULTS OF OVERFLOW DISCHARGE OF JULY 28, 1969 - NATURAL OVERFLOW, TEST NO. 8

Time*	Temp. °C	pH	Sett. Solids ml/l	T.S.S. mg/l	V.S.S. mg/l	COD mg/l	BOD mg/l
1700	24.0	6.6	7.0	16.0	13.0	—	90.0
1705	24.0	6.6	7.0	15.0	13.0	—	35.0
1745	24.0	7.0	2.5	18.0	15.0	—	35.0
1800	26.0	7.3	Trace	15.0	13.0	—	45.0
1830	26.0	7.3	Trace	15.5	11.0	—	50.0

*Manual actuation of sampler

RETURN FLOW - NO RETURN SAMPLES

comparable to the storm flow. Once the sewer has been flushed however, the monitored values quickly drop to minimum levels in terms of pollution sources.

The four natural occurrence overflows were runs No. 4, 5, 7, and 8. The first one, No. 4, was a very short rainfall that generated an insignificant quantity of overflow water. Even then a decrease in all of the measured paramters was evident. This decay indicates that the sewage in the line has been flushed out and the storm flow has taken over as the major contributor. The return flow samples are close to averaging out the values for the incoming flow. This means that once in the tank the water has been homogeneously mixed and has not measurably changed its state.

The second rain was of a longer duration and therefore allowed the collection of samples over a 1-1/2 hour period. In contrast to the previous rain all the measured parameters remained relatively constant during this storm period. The initial first flush was not as high, but this was due to the fact that a previous storm and a simulated run on the two previous days had cleaned out the sewer. Generally, the dry-weather sewer sludge accumulations are scoured from the combined sewer by higher velocities and turbulences of flow during rain storms or simulated storm events. A summary of the rainfall and overflow data are presented in Table X.

The use of flumes as metering devices for the discharge of storm overflow from the drainage basin was ineffective, especially for the conditions of the tidal river. For each natural overflow occurrence during the operational period, the tide caused surcharged conditions within the sewer, thus flooding the measuring devices, and causing inaccurate readings; therefore, discharge hydrographs are not available.

In terms of biochemical oxygen demand (BOD), chemical oxygen demand (COD), and settleable solids, the return flow from the storage tank appears unchanged from the incoming flow of sewage. However, the total solids and volatile fraction did increase significantly. This would indicate that some previously deposited material was scoured up by the large turnover of liquid and was flushed out. This material was not of sufficient quantity however to have a noticeable effect on the BOD.

The next rainfall was run No. 7. This runoff followed the pattern of the first storm with the samples showing a decay in pollutional strength with time. The level of strength was comparable to run No. 5 in terms of COD and total suspended solids (T.S.S.) but was only half the strength in terms of BOD. No samples were available on the return flow.

The fourth rain storm, run No. 8, indicated a first flush in terms of BOD but not in terms of T.S.S. In this case the T.S.S. remained at a level comparable to the high point for previous storms. The BOD however started at the level of the previous storm and then showed a significant reduction.

In an overall view the BOD and COD levels characterized a flow comparable to typical sewage.

TABLE X

SUMMARY OF RAINFALL AND OVERFLOW

Date	Activity (storm)	Total Rainfall (inches)	Overflow Pumped to Storage (gallons)	Overflow Pumped From Storage (gallons)
6/19	simulated	-	91,700	96,700
6/24	simulated	-	111,400	119,600
6/26	simulated	-	108,100	113,500
7/5	natural	0.45	20,600	20,600
7/7	natural	0.75	44,300	47,700
7/10	simulated	-	43,400	47,000
7/12	natural	1.15	55,200	58,800
7/28	natural	0.85	121,000	-
7/29	simulated*	-	184,000	-

*This simulation was conducted to fill the storage tank to maximum capacity for visual inspection.

In contrast the total suspended solids were consistently low. With the limited amount of data it is impossible to determine if this was due to analytical procedures or was actually a characteristic of the runoff from this particular basin. Regardless of what form it was in, the runoff did carry with it a significant pollutional load in terms of oxygen demanding material. The flow when allowed to be regularly discharged will have a noticeable effect in the quality of the waterway.

The volumes of runoff water generated from this small basin indicate that efforts to contain the total combined sewer flow in any single container may be close to impossible; however, multiple tank installations could be used to provide sufficient capacity. During the short operational period, what might be considered typical storms for the area were experienced. These storms generated overflows in excess of the pilot plant's pumping capacity of 2000 gpm and in excess of the capacity of 18" overflow line. A projection to an overall full scale operation for the entire city of Cambridge would require a reservoir system capable of storing approximately 3.5 million gallons, to handle the major portion of the polluting material.

Public Attitude and Acceptance

One of the criteria for a successful program was the development of a system which was compatible with the surrounding buildings and which would not create a nuisance condition to the neighborhood. In terms of engineering knowledge it is believed that this had been accomplished. The onshore facility, once installed, was visible only as two low rising manholes. The offshore facility was marked by buoy lights to warn boaters of its underwater presence, but otherwise it was not visible. The local boating community did not see this as an invasion of their waterway, but on the other hand they were curious and interested observers. One unfavorable aspect of this program which manifested itself from the beginning of the work, but for which we were unable to fully plan ahead was the repercussion this project had on the neighborhood.

After the selection of the Choptank Avenue site, approval was sought and obtained from all concerned governmental agencies. This included the City Council of Cambridge, Maryland, the State Health Department of Maryland, the State Water Resources Commission of Maryland, the State Board of Public Works of Maryland including the Chesapeake Bay Affairs Commission, and the U.S. Army Corps of Engineers. After approvals for the installation and operation of the proposed project at the selected site was received from the agencies, a public hearing was held in Cambridge. The City Council approved the project and granted permission to Melpar to continue work on the project. During the same meeting, Melpar made the following concessions: relocation of the trailer and a power pole and lowering of the pumping station and wet well manholes, in an attempt to promote harmony.

The attitude of the residents immediately adjacent to the demonstration project indicated a feeling that any sewer problems which might be experienced were a direct result of having the facility located there.

An exceptionally large quantity of sea lettuce was present in the harbor and on the shoreline of the park. After a storm sufficiently intense to cause combined sewer overflow, raw sewage discharged near the shoreline of the park and bulk head near the Yacht Club on Water Street and further out into the water at West End Street. The outfalls in this area should be in deeper water where the more favorable current conditions prevail. The receiving waters at the outfall near the park have always remained polluted for a period of one to five days after each rainfall. The sewage from the outfalls adjacent to and including the Choptank outfall decomposed and provided nutrients to the obviously large quantities of algae which eventually washed ashore. Highly objectionable odors did emanate from the putrid sewage and sea lettuce. Public complaints concerning the foul odor were understandable: although, it was sometimes difficult to explain and convince people that the causes were not from the demonstration project.

As background to the reactions of the residents it must be remembered that Cambridge is an old seashore fishing village. Life is slow moving and quiet in a tranquil environment. The families have lived in the same homes for generations and any change is a disruption of their way of life. Out of this backdrop came such comments as "we fully support your idea and the need to stop pollution, but why can't you do it on the next street?" This background possibly explains why the running of an open fire hydrant in the night made too much noise in the neighborhood and caused them to call city officials requesting that the water be turned off. This water was being used to simulate a rainfall. The operation of the pumping station and storage facility itself did not apparently disturb the neighborhood. This is encouraging because in a normal application the facility itself would not be a disturbance to the neighborhood. It was, however, these research related disturbances which started to generate a community environment that was best handled by a discontinuance of any further tests.

SECTION VIII

REMOVAL OF PILOT PLANT FACILITY

The project site was restored to an appearance compatible with the surrounding area as it existed prior to construction of the pilot plant facility. Both the offshore and onshore work was performed under contract with Smith Brothers Pile Driving, Inc. (Galesville, Maryland). The cost for removal and site restoration was $28,670.00.

The collapsible offshore storage tank and all its appurtenances were removed from the Choptank River. After the removal of the storage tank from the river bottom, the flexible cover was removed and an inspection was made of the tank. Figure 14 shows a close-up view of the side of the steel tank. Numerous marine crustaceans were attached to the tank's steel bottom (and also the flexible cover). No apparent damage was observed as a result of the presence of marine life.

The U-clamps, which were used to secure the storage tank to the wood pilings, had rusted because they were not coated with a protective vinyl, an obvious oversight before the installation of the tank. One of the clamps is shown in figure 14.

The inside of the steel tank was relatively free of sludge deposits. This indicates that the flushing procedure and circulation system was effective in keeping the tank clear of settleable solids. Figure 15 shows an internal view of the storage tank. The silt that was observed in one area of the tank was the result of the discharge of external sedimentation from the flexible top into the tank during the removal of the flexible cover.

The submersible pump and gate valve are shown in figure 16. The gate valve and the mounting plate for the pump were damaged during the removal of the storage tank. The deposits, both marine crustaceans and rust, could have a deleterious effect on the heat transfer characteristics of the submersible pump. During the course of the program, no problems were encountered with the operation of the pump nor with the protective heater element.

In general, the flexible cover was believed to be in good condition. After the initial repairs, damages were not observed on the flexible cover as a result of system performance and the presence of saline water and marine crustaceans.

The patches were intact on the torn areas of the cover. The metal plates of the patches did not cause damage to the cover. Hard rubber plates should be substituted for the metal plates and assembled with plastic nuts and bolts. The presence of saline water was obviously detrimental to the metal components of the patches used in repairing the cover; however, time and the urgency to begin program testing did not allow for time consuming design, selection of materials, and testing.

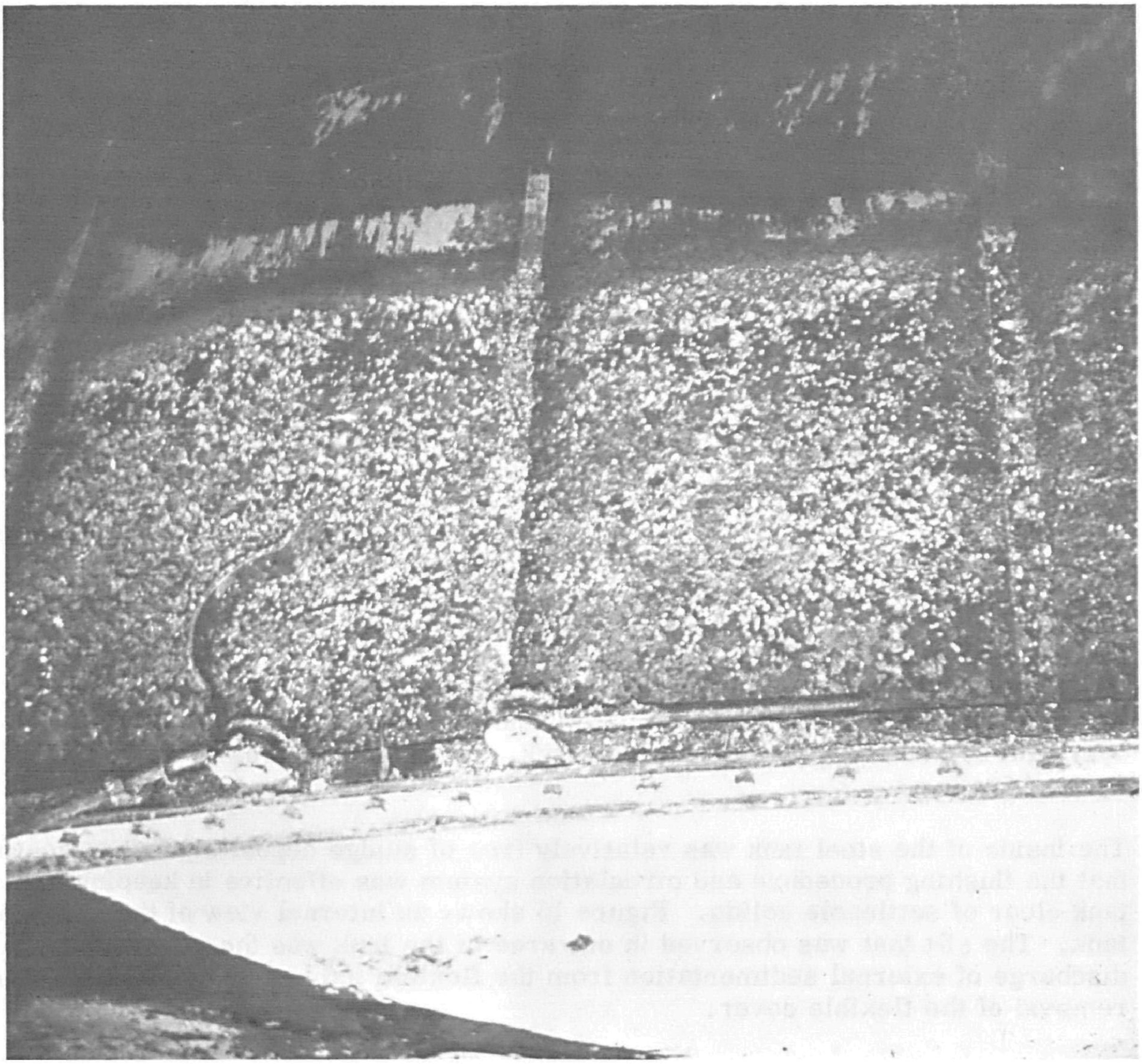


Figure 14. Close-up View of Storage Tank After Removal

Figure 15. Internal View of Storage Tank After Removal

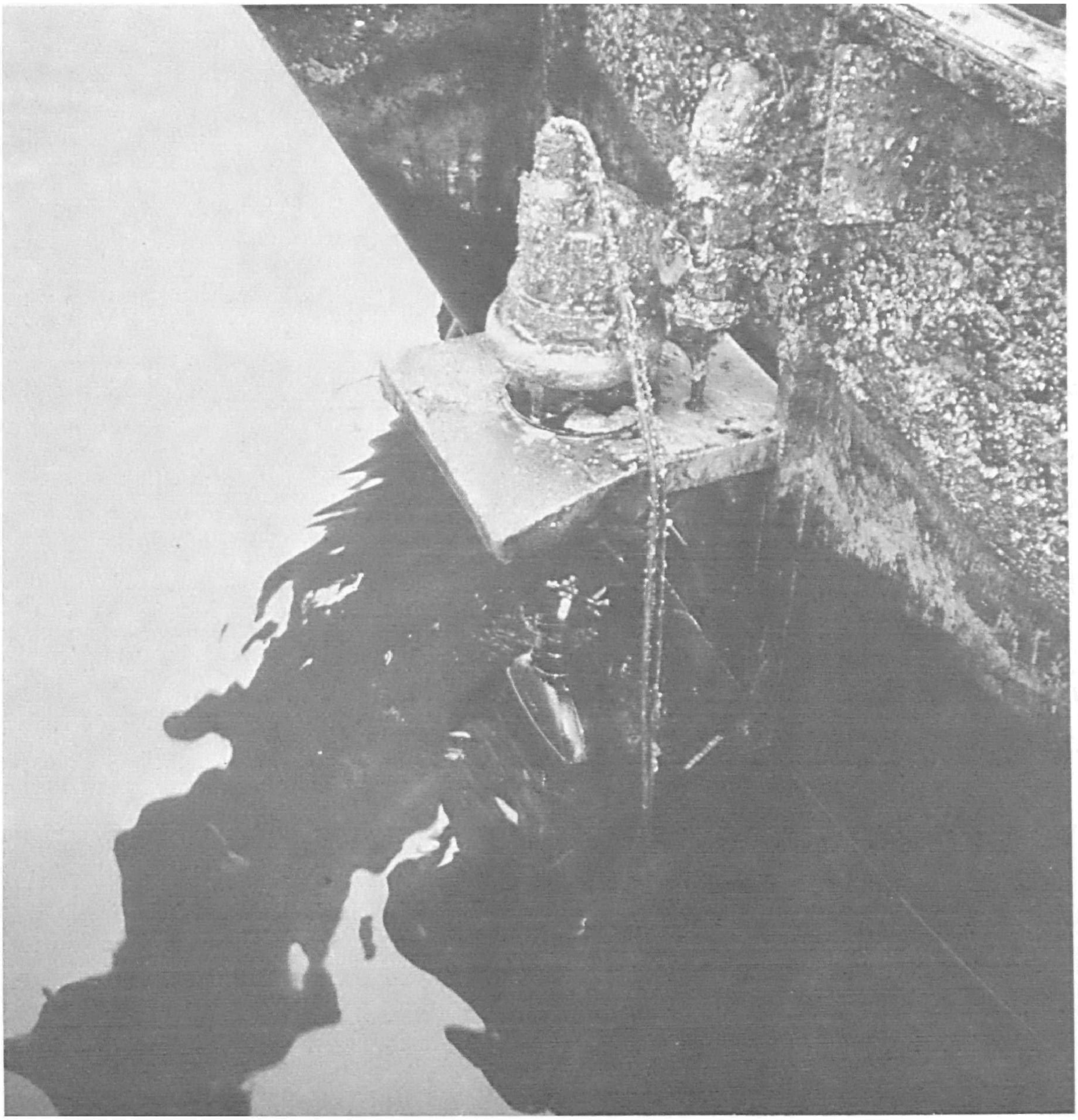


Figure 16. Submersible Pump and Gate Valve After Removal

The onshore restoration was performed as smoothly and efficiently as the off-shore restoration. Inspection of the onshore equipment did not reveal any damage, other than normal wear, except to the upstream recorder. The recorder was damaged by water that infiltrated in the unit during a surcharged sewer condition.

SECTION IX

COSTS

The costs for the pilot plant facility to store combined sewer overflow from the Choptank Avenue drainage basin are summarized in Table XI. The onshore and offshore subsystems were fabricated and constructed by subcontractors. The value given in the table for the construction and installation of the offshore system also includes the cost for pilings (6), 12-inch cast iron pipe (1300 ft.) and electrical cables (1300 ft.). Similarly for the onshore elements of the system, the sewer pipe, manhole materials, electrical cables and conduits, and the 8-inch cast iron pipe were furnished by the onshore subcontractor and these costs are included in the value given for the onshore construction.

A municipality could expect to duplicate the pilot plant facility at a similar site, for slightly more than $150,000. The total cost depends upon the obvious variables of local labor rates, the distance of the storage tank from the shore line, and individual onshore construction requirements. In addition, the associated costs of site surveys, detailed design, and inspection could increase the total cost of such a facility. These costs are not included in Table XI, for municipalities would frequently assign these functions to the normal work load of existing offices; i.e., City Engineer, Sanitary Engineer, Planning, etc.

The annual operation and maintenance costs for the pilot plant facility are tabulated in Table XII. The costs are based on labor and supervision supplied by the city only during overflow conditions and costs realized during the performance of this demonstration project. Maintenance costs include two underwater inspections of the storage module and general maintenance of the equipment.

The cost of replacing the combined sewer in the Choptank Avenue drainage basin with a complete separate dual system would be $262,000. This figure is based on the cost data published by the U.S. Department of Interior.[2]

The nominal design capacity of the pilot plant underwater storage facility was 200,000 gallons. The flexible cover actually contained a larger volume than expected, resulting in a total capacity of 248,000 gallons. The storage capacity of the pilot plant, then, was sufficient to allow total containment of the runoff from more than 40 of the 55 storms which are anticipated during a given year (see Table I). The facility could totally contain some 75 percent of the individual rainfall events.

The average annual runoff from the Choptank Avenue drainage basin is estimated to be 8.9 million gallons. The pilot plant facility could contain 8.6 million gallons of this runoff, or approximately 96 percent of the total yearly overflow. The storage and treatment of this quantity of combined sewer overflow would prevent the annual discharge of 7,136 pounds BOD, based upon an average five day BOD of 100 ppm. The discharge of all but 299 pounds BOD would be prevented.

TABLE XI

TOTAL COST FOR COMBINED SEWER OVERFLOW FACILITY

Task			Total Costs
OFFSHORE			
Subsystems and Supplies			
Steel tank		$	27,175.00
Flexible cover			13,254.00
Installation of flexible cover to tank			2,039.00
Miscellaneous supplies			3,368.00
	Subtotal	$	45,836.00
Construction		$	46,830.00
	Subtotal	$	92,666.00
ONSHORE			
Subsystem and Supplies			
Pumping station and wet well		$	22,550.00
Remote control panel			6,200.00
Miscellaneous supplies			3,097.00
	Subtotal	$	31,847.00
Construction		$	31,488.00
	Subtotal	$	63,335.00
MAINTENANCE - REPAIRS		$	865.00
LABOR (OPERATING, SAMPLE COLLECTION, WATER QUALITY ANALYSES)		$	2,167.00
TOTAL		$	159,033.00

TABLE XII

ESTIMATE OF ANNUAL OPERATION AND MAINTENANCE COSTS

Labor and Supervision	$4,500
Maintenance	1,100
Power	1,400
Materials and Supplies	300
Total Annual Cost	$7,300

The approximate cost of storage and return per one thousand gallons is $1.85 based upon prorating the facility cost over a ten year life expectancy for the system; increasing to $2.71 when the estimated annual operation and maintenance costs are added to the installation cost for the same ten year period. The approximate costs per pound of BOD are $2.22 and $3.25, respectively, on the same basis. The ten year life expectancy is considered to be a conservative estimate, and the annual operational costs may well be a part of other sewage department labor assignments. The actual cost of storage and return could be less than $1.50 per thousand gallons, with a cost per pound of BOD less than $2.00, if the underwater storage facility was an integral part of an overall sewage treatment system.

SECTION X

ACKNOWLEDGEMENTS

Melpar wishes to express its appreciation to the several organizations and many individuals who made extensive contributions during the course of this program. Melpar is especially indebted to the City of Cambridge, Maryland and to the Honorable Mayor Osvrey C. Pritchett and the City Council. Mr. Robert L. Dodd, Director of the Department of Public Works, rendered valuable assistance and furnished valuable data and information relative to the program. Melpar is indebted also to Mr. Bernard W. Dahl, Consulting Engineer, Rockville, Maryland, for his capable and valuable assistance and consultation. The service provided by the following subcontractors was of considerable value to the conduct of this project:

American Welding Co., Baltimore, Md., Steel Tank

Schmieg, Division of Sydnor Hydrodynamics, Inc., Richmond, Va., Wet Well and Pumping Station

Uniroyal Plastics Products, Mishawaka, Ind., Rubber Cover

Smith Brothers Pile Driving, Inc., Galesville, Md., Offshore Construction

Norris E. Taylor Contractors, Inc., Easton, Md., Onshore Construction

B. C. Langley

T. P. Meloy

SECTION XI

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2. "Problems of Combined Sewer Facilities and Overflows 1967." U. S. Department of Interior, FWPCA, Publ. WP-20-11, U. S. Government Printing Office, Washington, D.C. (1967).

SECTION XII

APPENDIX

Choptank Avenue and Park

Flexible Cover Fabrication

Fabric Back-up Plate

Storage Tank

Storage Tank Sub-Foundation

Storage Tank Installation

PAGE NOT

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ACCESSION NO.

KEY WORDS:

Storm overflow

Combined sewers

Underwater storage

Hydrology

Pumping station

BIBLIOGRAPHIC:

Melpar, an American-Standard Company, Combined Sewer Temporary Underwater Storage Facility, Program No. 11022DPP, Contract No. 14-12-133, October 1970.

ABSTRACT

A pilot plant underwater storage facility was designed, constructed, operated and evaluated as a method of temporarily storing storm overflow from the combined sewer of the Choptank Avenue drainage basin, Cambridge, Maryland. Combined sewage in excess of the sewer capacity, which would normally be discharged directly into the Choptank River, was intercepted and pumped into a nominal 200,000 gallon flexible underwater storage container located 1300 feet offshore. The stored overflow was later returned from the tank at a rate which could be accommodated by the intercepting sewer and treatment plant.

The facility was tested with overflow both from four naturally occurring rainfalls and using fresh water simulation. The overflow samples were analyzed in a field laboratory for the following characteristics: pH, suspended solids, volatile suspended solids, settleable solids, 5 day biochemical oxygen demand, and chemical oxygen demand.

The pilot plant facility was capable of collecting 96 percent of the average annual overflow from the drainage basin at a cost of less than $1.85 per thousand gallons. The facility could prevent the annual discharge of 7,136 pounds BOD into the Choptank River.

Underwater storage facilities could be used effectively for many combined sewer areas. Site selection, however, has been proved to be a critical factor. Care must be exercised to prevent public disturbance, and factors such as land use, tidal conditions, or the types of storms, must also be considered.

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5 *Organization*

MELPAR, an American-Standard Company
7700 Arlington Boulevard
Falls Church, Virginia 22046

6 *Title*

COMBINED SEWER TEMPORARY UNDERWATER STORAGE FACILITY

10 Author(s)	11 Date	12 Pages	15 Contract Number
Meloy, T. P. Langley, B. C.	October 1970	79	14-12-133
	16 Project Number		21 Note

22 *Citation*

23 *Descriptors (Starred First)*

storage overflow
combined sewers
underwater storage
hydrology
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25 *Identifiers (Starred First)*

27 *Abstract*

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Abstractor T. P. Meloy

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