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Control of Pollution by Underwater Storage

U.S. DEPARTMENT OF THE INTERIOR ● FEDERAL WATER POLLUTION CONTROL ADMINISTRATION

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CONTROL OF POLLUTION

BY

UNDERWATER STORAGE

Feasibility of providing temporary underwater storage of storm overflow from a combined sewer system.

FEDERAL WATER POLLUTION CONTROL ADMINISTRATION
DEPARTMENT OF THE INTERIOR

by

UNDERWATER STORAGE, INC.
SILVER, SCHWARTZ, LTD.
JOINT VENTURE

1028 Connecticut Avenue, N.W.
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Program No. 11022 DWF
Contract No. 14-12-139

December, 1969

F.W.P.C.A. Review Notice

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

ABSTRACT

A pilot plant was designed, constructed and operated to assess the feasibility of providing a facility for the collection, treatment, storage and final disposition of a portion of the storm overflow from a combined sewer system serving a thirty-acre drainage area in Washington, D.C.

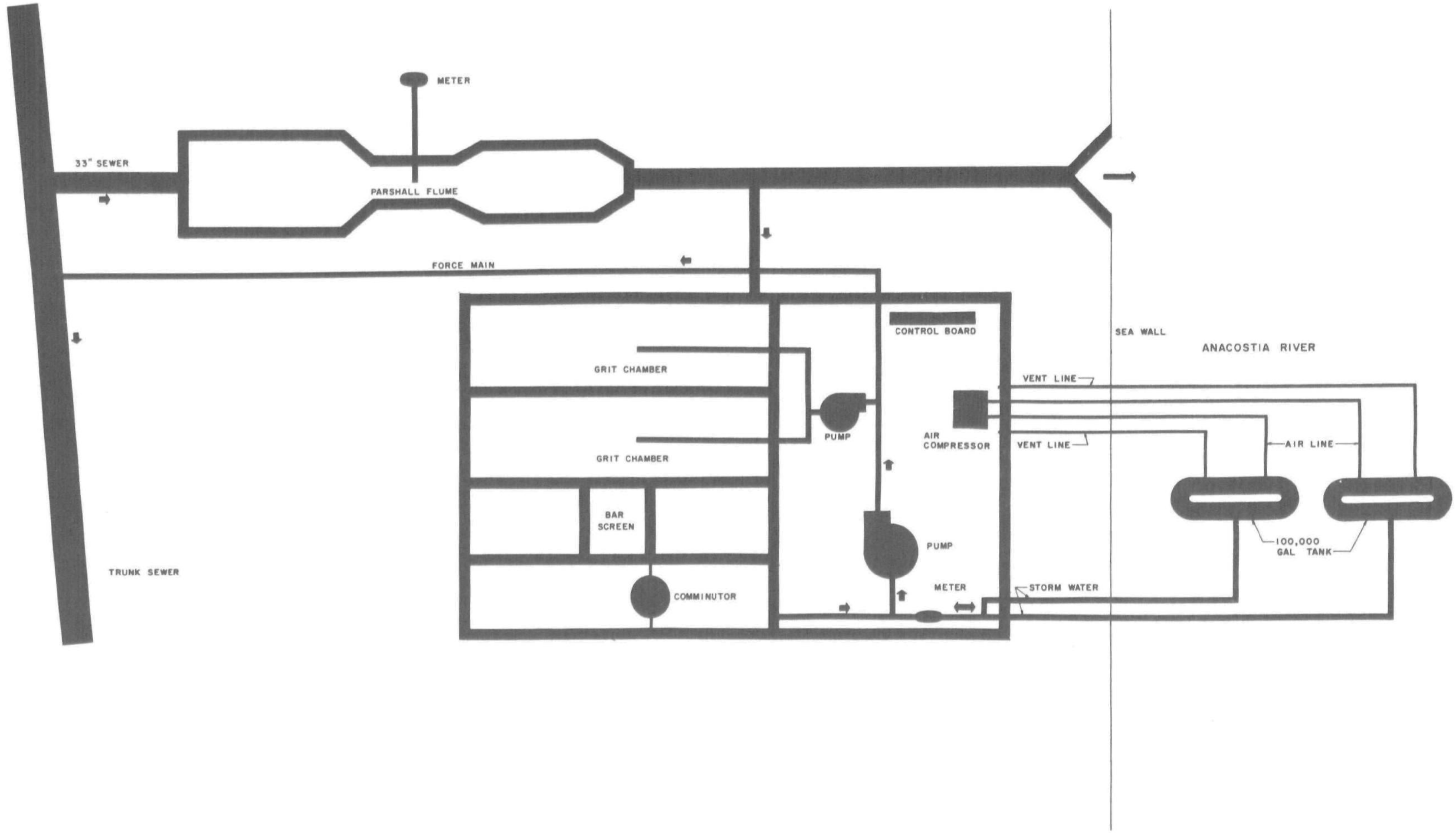
A Parshall flume was installed in the overflow line for measurement of flow rates and determination of total overflow volume. A portion of the overflow was diverted to the pilot plant through grit chambers and a comminutor. Flow was stored in two 100,000-gallon underwater bags fabricated of nylon reinforced synthetic rubber and fastened to the river bed by a system of patented anchors. During the period of storage, compressed air was delivered to the tanks for agitation of the solids. Following cessation of the storm, contents of the bags were pumped to the interceptor sewer for delivery to the District of Columbia Sewage Treatment Plant at Blue Plains. Flow into and out of each underwater storage tank was metered and recorded. Samples of the combined sewage overflow discharged to the bags and pumped discharge from the bags were collected and subjected to laboratory analyses.

During the operation period from January through September, 1969, a total of 1,600,000-gallons of diverted overflow from 38-storms was stored in the tanks. In addition, 600,000-gallons of river water was pumped into the underwater storage tanks for testing during dry weather periods. The total amount stored was pumped to the interceptor sewer in 26-separate pump out periods.

The cost of the pilot plant was $341,480.00, or $1.70 per gallon of storage. This included facilities for testing, samples and flow measurement. Estimates for larger installations, without these special requirements range from 28.2¢ to 14.6¢ per gallon for plants with storage from two to twenty million gallons.

The project demonstrated that temporary storage of overflow from combined sewers in underwater rubber storage tanks is feasible and may, under suitable conditions, be effective in eliminating direct, untreated discharge of combined sewage into surface waters during storm periods. Drainage area to be served, land use, nature of storm events,and other factors must be considered when planning an underwater storage facility.

This report is submitted in fulfillment of Contract No. 14-12-139 between Federal Water Pollution Control Administration and Underwater Storage, Inc., Silver, Schwartz, Ltd., Joint Venture.



FLOW DIAGRAM

v

Fig. 1

STORM OVERFLOW PROJECT

UNDERWATER STORAGE INC. & SILVER, SCHWARTZ LTD. ——— JOINT VENTURE

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FIGURES

FIGURES

TABLES

SECTION 1

CONCLUSIONS

A holding facility eliminates many of the problems encountered with other means of controlling water pollution from overflow sewers. The holding facility can be above ground, below ground, or under water. The underwater holding facility lends itself, under certain conditions, in meeting the problem for the following reasons:

1. Since overflow is always near a river, stream or lake, space is either available or can be made available by dredging to install an underwater holding facility.

2. Land area along a river bank or other body of water is frequently inadequate for an above ground holding facility, and acquisition of shore line real estate for such use is generally costly and time consuming.

3. There are no significant odor problems in underwater installations.

4. Beautification along our waterways is greatly enhanced by the avoidance of large land structures. Storage facilities are out of sight, permitting unhindered development of shore line property for industrial, residential or public use.

5. Underwater storage offers an economical solution for control of pollution. The maintenance of the system is minimal. The system components remain serviceable for years with little upkeep.

A further advantage of underwater holding tanks is found in construction flexibility in the fact that tanks can be installed in multiple units at each overflow, thereby permitting work to be accomplished in phases. This further allows expansion of the system as required, permitting capacity to keep pace with municipal growth. In addition, the storage tanks are portable, floatable, replaceable, collapsible, washable, odor free and flexible.

The construction cost of the project included many items not required for an actual operating installation and is, therefore, not indicative of actual anticipated construction costs. Construction estimates, based on ENR Construction Cost Index of 1300, from two to twenty million gallons storage, as shown on Table I are more realistic for actual requirements. These estimates are based on the use of 500,000-gallon and 1,000,000-gallon underwater storage tanks in lieu of 100,000-gallon tanks as utilized (costs for the larger tanks are considerably less than costs for equal capacity of smaller tanks) in the demonstration project. These estimates include grit chambers, comminutor, pump house, metering station, pumps, piping, air compressor, Parshall flume, instrumentation, electrical work, earthwork and all underwater work of tanks, installation and dredging. These estimates assume certain ground conditions that may not require extensive costs in ground work, excavation, sheet piling, dewatering, etc., as was encountered in the pilot plant. Even with all these unknown variables, it is seen in Figure 2 that as the storage capacity of the project increases, the cost per gallon of storage will level off at approximately twelve cents per gallon. This cost does not include automated computerized control.

Reference is made to Section 8, Construction Costs, page 51 for explanation of the various line items of cost indicated in the Estimates of Table I.

As a result of this pilot project, it was determined that many cost savings could be realized in such areas as in the use of tank flotation systems, installation of quick-connect fittings for underwater work, replacement of underwater bolting with shackles, installation of nets for hold-down of tanks and change in venting system on tanks. It was also determined that the cost per gallon of underwater storage tanks decreased as the capacity increased. The cost of a 500,000-gallon tank is less than ten cents per gallon and the cost of a 1,000,000-gallon tank is estimated at eight cents per gallon, whereas the cost of a 100,000-gallon tank is 14.4 cents per gallon.

TABLE I

CONSTRUCTION COST ESTIMATES

Storage Capacity	2 M Gal.	3 M Gal.	5 M Gal.	10 M Gal	20 M Gal.
1. General Conditions	$2,000	$2,000	$2,000	$2,000	$2,000
2. Bond and Insurance	4,500	6,000	7,800	14,000	23,000
3. Earthwork*	25,000	30,000	34,000	40,000	52,000
4. Sheet Piling and Dewatering*	33,000	36,000	40,000	48,000	58,000
5. Lawn and Planting	800	1,000	1,200	1,500	2,000
6. Timber Piles on Shore*	20,000	24,000	30,000	35,000	45,000
7. Off-Shore Work*	78,000	92,000	115,000	150,000	190,000
8. Concrete	35,000	40,000	48,000	60,000	80,000
9. Miscellaneous Metal	6,000	6,000	8,000	8,000	10,000
10. Structural Steel	20,000	28,000	44,000	80,000	140,000
11. General Construction	10,000	10,000	12,000	12,000	15,000
12. Painting	3,000	3,000	5,000	5,000	8,000
13. Mechanical Equipment	60,000	68,000	80,000	96,000	125,000
14. Pipe and Fittings	20,000	24,000	30,000	40,000	55,000
15. Electrical	7,000	8,000	10,000	13,000	18,000
16. Underwater Storage Tanks and Piping	240,000	350,000	525,000	1,050,000	2,100,000
TOTALS	$564,300	$758,000	$987,000	$1,754,500	$2,923,000
Cost per Gallon - Total Installation	28.2¢	25.3¢	19.7¢	17.5	14.6¢
Cost per Gallon - Storage Only	16.9¢	15.7¢	13.7¢	12.8¢	12.2¢

*Price depends on design and ground conditions. Will vary from project to project.
Cost per gallon for storage only is obtained from the sum of Items 7, 10 and 16 of the Estimate.

COST PER GALLON

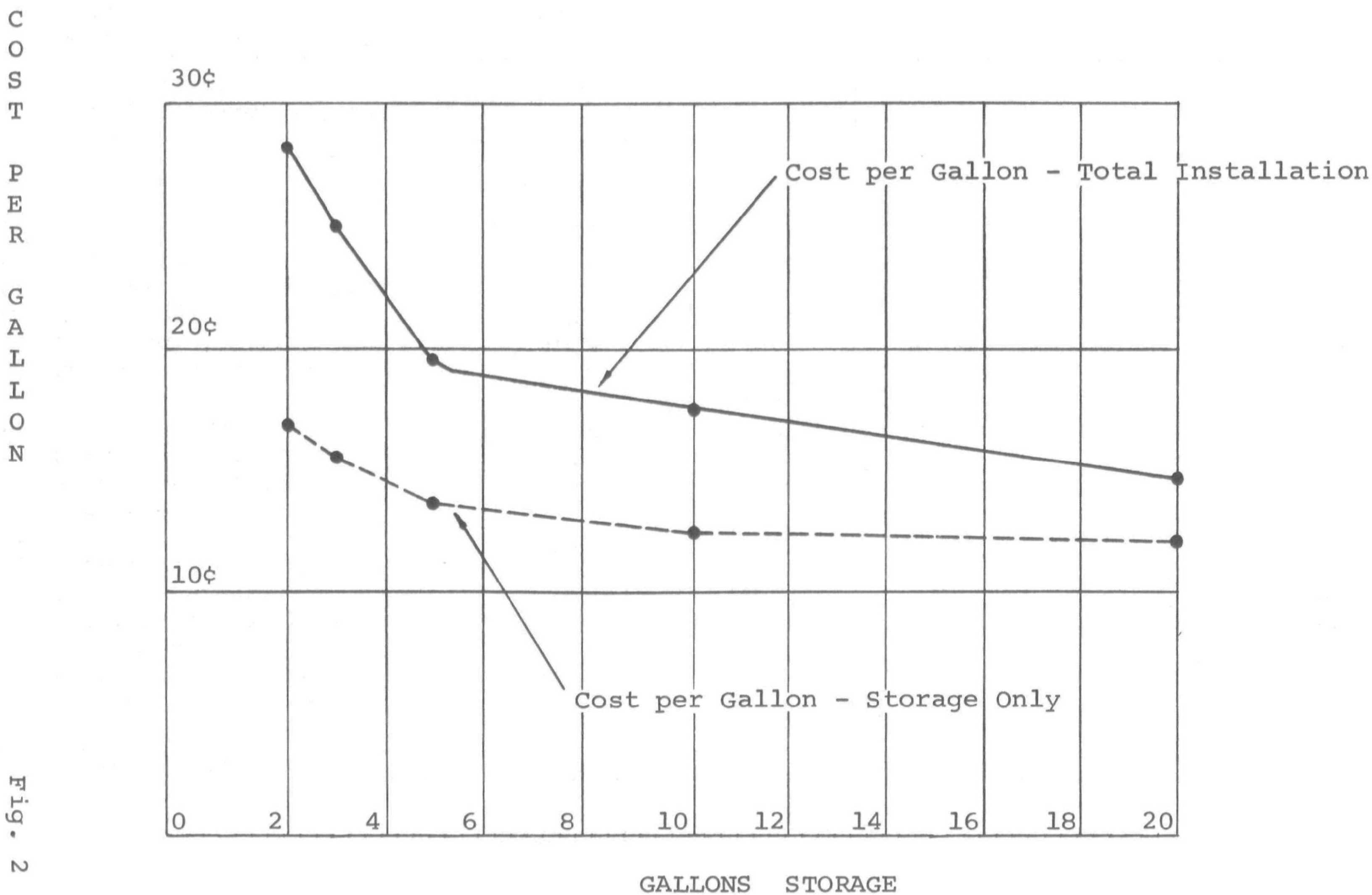


Fig. 2

Annual operation and maintenance costs for the underwater storage project have been estimated based on experience gleaned in the demonstration facility. The items of labor and supervision include the necessary disposal of screenings and pump station operation. Actually, labor need only be provided for operation during overflow conditions. Materials and supplies for operation are minimal. Power includes that required for pump and comminutor operation. Maintenance costs allow for general upkeep of plant equipment and periodic inspection of tanks and underwater piping. Miscellaneous costs include telephone, charts, etc.

TABLE II

ESTIMATED ANNUAL
OPERATION AND MAINTENANCE COSTS

Storage Capacity	2 M Gal.	3 M Gal.	5 M Gal.	10 M Gal.	20 M Gal.
Labor and Supervision	$7,500	$8,500	$10,000	$12,000	$12,000
Materials and Supplies	500	500	500	500	500
Power	2,000	2,800	4,500	8,500	16,000
Maintenance	5,000	6,000	8,000	10,000	14,000
Miscellaneous	600	600	600	600	600
Total Annual Operating and Maintenance Cost	$15,600	$18,400	$23,600	$31,600	$43,100

Total annual cost is the total of annual operating and maintenance cost and cost of amortizing the capital expenditure. The useful life of the project is estimated to be in excess of twenty years. However, for the purpose of this report, twenty years is selected with interest rate of 6.0%. On this basis, the capital recovery factor is established at 8.4%.

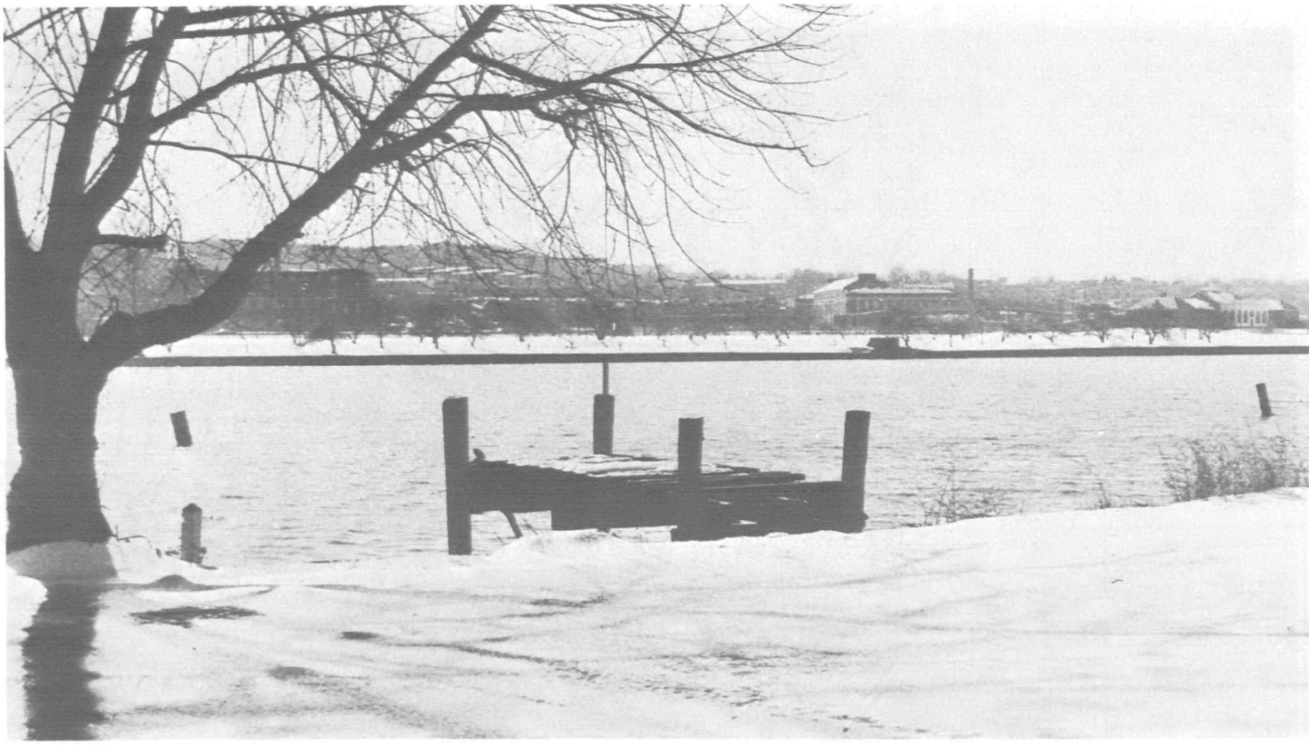
TABLE III

ESTIMATED TOTAL ANNUAL COST

Storage Capacity	2 M Gal.	3 M Gal.	5 M Gal.	10 M Gal.	20 M Gal.
Capital Project Cost	$564,300	$758,000	$987,000	$1,754,500	$2,923,000
Amortization Cost	47,400	63,700	82,900	147,400	245,500
Operating and Maintenance Costs	15,600	18,400	23,600	31,600	43,100
Total Annual Cost	$ 63,000	$ 82,100	$106,500	$ 179,000	$ 288,600

Side View of Pump House Fig. 3

Rear View of Pump House Fig. 4



Marker Buoys Locating Underwater Storage Tanks Fig. 5

It is noted by observation and by analyses at this site that a major portion of the polluting material is contained during the initial flush of the overflow. During this period of several hours duration, it is important to capture and store all of the overflow. The balance of the overflow has less polluting effect on the receiving waters; therefore, complete storage may not be absolutely required. This conclusion may be an important factor where limited funds necessitate partial storage at the outset of a program.

In order to properly establish the capacity of holding facility, it is necessary to estimate the assimilated pollution capacity of a particular receiving body of water as related to rainfall and to provide storage capacity for that rainfall in excess of the receiving water requirement, but no less than the initial flush of the overflow.

Under the above conditions, the capacity of a holding facility for storm sewer overflow could be planned to control anywhere from 25% to 50% of actual overflow discharge during heavy rains, and 100% during light rains.

These findings will also offer additional flexibility in the possible design of on site sewage treatment facilities. By storing the initial flush of storm water overflow for subsequent pumping, it may be possible to treat the diluted low content pollutant flow with a lesser treatment facility than would be required for a sewage treatment plant to serve the entire flow.

The pilot facility has demonstrated that it is possible to divert combined sewer overflow to underwater rubber storage containers and to return the stored waste water to the interceptor sewer at a time when the flow in the combined sewer is such that the stored waste water can be accepted. This may occur shortly after the storm ceases.

The demonstration has proven conclusively that combined sewer overflows need not be discharged into rivers, lakes and waterways, but can be received, treated, stored and pumped to final disposal, and this major cause of water pollution can be controlled.

SECTION 2

RECOMMENDATIONS

It is recommended that existing overflow from combined sanitary and storm sewer systems be controlled wherever feasible. Each municipality or sewage authority should analyze the costs and problems of various methods presently available or proposed to achieve this purpose. In analyzing various proposed alternatives, it is necessary to view the entire picture, not just direct costs, but side effects and costs such as tearing up of streets, interference with traffic, operational problems, system reliability and equipment durability, effectiveness, system flexibility and esthetics. It will be found in many cases that underwater storage will be the answer to this critical problem.

It is recommended for the future that consideration be given to additional treatment in connection with underwater storage overflow installations. Inasmuch as storage provides the key factors of hydraulics and time that is a prerequisite for treatment plant operation, the amount of treatment for one or more overflow systems need not be of the full capacity of the overflow, but may be a fraction of the flow. The relationship of treatment capacity to storage capacity should be a matter of economics. If more storage is provided, then lesser treatment capacity would be necessary. If, in future studies for additional treatment, it is determined that full pumping back to the interceptor sewer is not required, then as many overflows as possible, within economic reason, should be served by a single installation.

Where many overflows occur in a municipal system, it is recommended that installations be automated to operate from a central computerized control. This not only would reduce operating cost, but would eliminate potential human error.

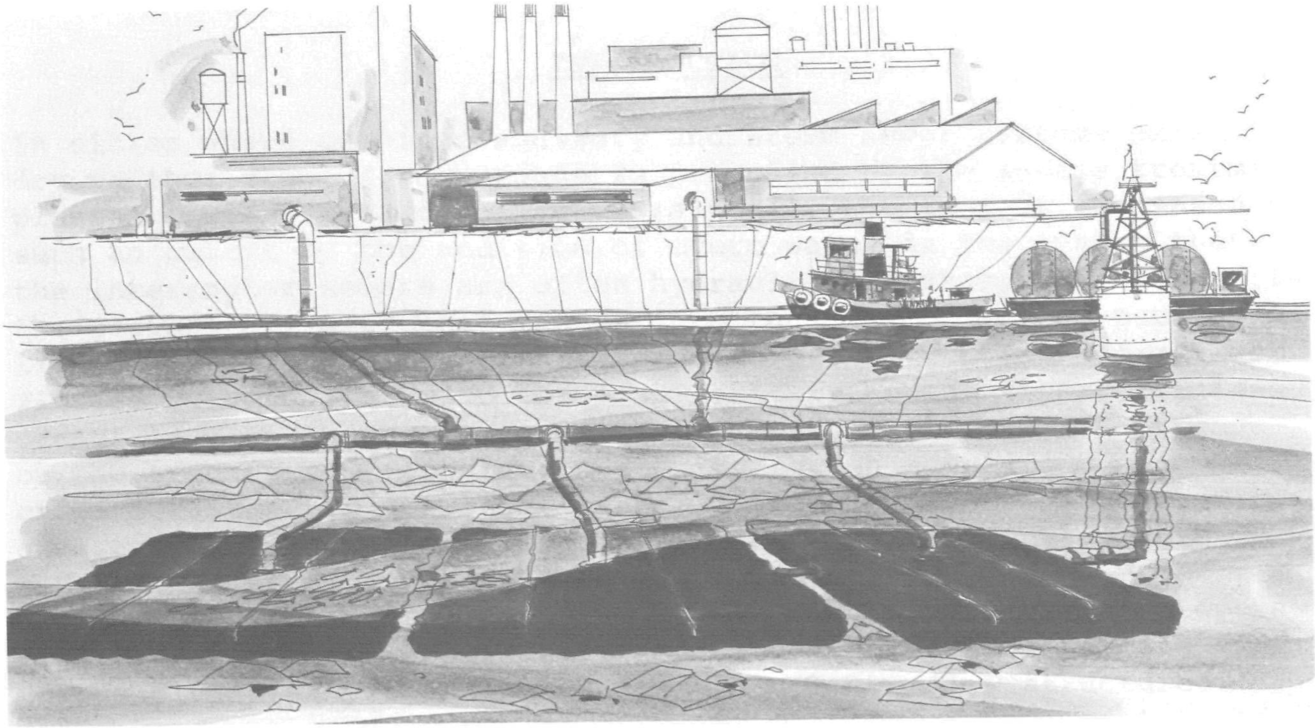
It is recommended that underwater storage tanks be provided with built in flotation facilities for raising and lowering the tanks.

It is recommended that quick-connect connections be used on all piping below water, not just on air and vent lines. In the demonstration project, flanged connectors were used on the main lines; as a result, costs for piping connections were excessive because of labor costs of divers. These high costs were also reflected in disconnecting lines when raising tanks.

Another area in which it was found that installation of disconnecting costs can be reduced was in the use of plate clamps or shackles, rather than bolted connections, for securing underwater storage tank supports to the anchoring system.

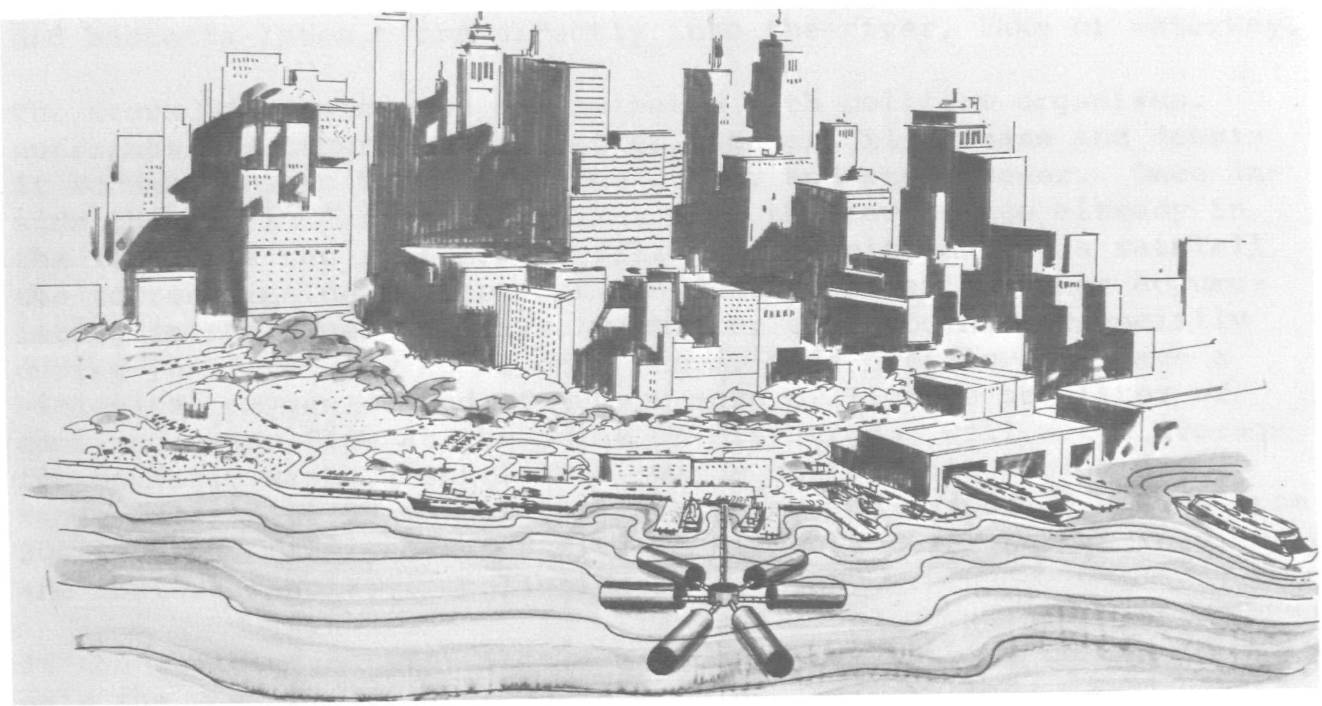
It is recommended that underwater storage tanks be provided with marine nets for hold-down purposes and to enable uniform pressure to be exerted on the tanks as well as controlling tank configuration. In addition the use of underwater diaphragm purge valves for venting air and gas from the underwater storage tanks have proven extremely effective and should be utilized on all underwater projects. These vents also control internal tank pressures.

Multiple tank installations can be installed in a cluster as shown in rendering Figure 6, or in a line from a single header as shown in rendering Figure 7. The in-line arrangement can also be installed with a single branch runout to each tank as was accomplished in the pilot project.



Rendering of Multiple Tank Installation
In a Cluster

Fig. 6



Rendering of Multiple Tank Installation
In a Line

Fig. 7

SECTION 3

INTRODUCTION

In cities where combined sanitary and storm sewer systems exist, dry weather flow of raw sewage is processed by the sewage treatment plants. During periods of heavy rainfall, the flow is increased to such an extent by the addition of storm water in the system that the interceptor sewers are often hydraulically incapable of handling the cascade of water that floods into the lines. During periods of light rainfall, the problem is found to exist when the storm sewer flow is coincidental with the peak sanitary sewage flow. The increase in flow in the interceptors because of storm water depends on the area served, population density, green space, and the characteristics of the sewer system. Under these conditions of sanitary and storm water flow, present sewage treatment plants do not have sufficient capacity to treat the increased flow. To relieve the system and to protect the sewage treatment plant operation, overflows are customarily employed to direct all excess flow to the rivers, lakes and waterways through overflow structures. This expedient reduces pressure on the main sewer interceptor and aids in avoiding backup into the streets. However, as a result, much of the sanitary waste (as much as 90%) never reaches the sewage treatment facility. The overflow combined sewer, unprocessed and bacteria-laden,pours directly into the river, lake or waterway.

The storm water alone is contaminated with coliform organisms, nutrients and other foreign matter such as oil, grease and debris it accumulates on its way to the gutter and storm sewer. Once inside the combined sewer, it mixes with the raw sewage already in the line. As the interceptor fills to capacity during a rainfall, the torrent inside flushes out the sediment that has been accumulating in the lines for days, weeks, or even months. Especially during the first few hours of a storm, this overflow may have a biological oxygen demand of 400 or more milligrams per liter of combined sewer flow at the point of discharge, well above average for domestic waste. Settleable solids during this period may range from 10 to 60 milliliters per liter and suspended solids from 200 to 1,000 milligrams per liter. All this leads to the inevitable and obvious result.....pollution.

If the sewage treatment plants were of adequate size to accommodate the peak flows existing in combined sewers, they would have to be tremendously enlarged at prohibitive land and construction costs.

One solution that has been advanced and widely used to resolve the problem of combined sewer overflow is to separate the systems; i.e., to retain the existing combined sewer for storm water flow and to provide a new and independent system for sanitary sewer flow. The problems inherent with this separation concept are many and complex. Besides being extremely costly, the installation of separate sanitary sewer lines entails tearing up of streets, interference with other underground utilities, tying up of traffic and the complete revamping of individual building connections. Even with the separation of the systems taking place, there is still the problem of storm water pollution; which, though not as extensive, is still a major factor in the overall problem.

Other solutions to the combined sewer overflow situation take into account some form of holding facility to retain the overflow for separate treatment or for treatment by the municipal sewage plant during periods of low flow. With either situation it is necessary to provide adequate storage at the holding facility to enable the sewage plant to effectively treat the stored overflow over a period of several hours to several days, depending upon plant capacity, during off peak hours.

Some of the holding systems which have been proposed with treatment at the facility have been retention and sedimentation basins, above ground holding tank and deep tunnels. With off-shore retention basins there is the problem of blocking off large water areas to receive the flow. The above ground holding facility requires large land structures, results in potential odor problems, and necessitates the need for costly and time consuming acquisition of shore line real estate. The deep tunnel project is costly; present estimates range from \$.40 to \$.45 per gallon of storage, based only on preliminary studies.

It is desirable to find an economical system for holding of combined sewer overflow that is not costly, that removes the necessity of tearing up streets, that is esthetically acceptable and out of sight, that permits unhindered development of shore line property for industrial, residential or public use, that is flexible for expansion to keep pace with municipal growth, that provides some means for primary and possible secondary treatment, and that is equally or more effective in reducing pollution than those methods presently in use or being considered. This report deals with the feasibility of one such possible alternative.

SECTION 4

SITE SELECTION

A preliminary investigation was made by studying the "Sewerage System" map of the District of Columbia (See Figure 8 on Page 23) in conjunction with the Sewer Separation Program - 1966, report by the District Sanitary Sewer Department. There are approximately fifty outfalls of combined sewers along the Potomac and Anacostia Rivers, all of which had potential as sites for the pilot plant. Investigation revealed most of them as unusable for compliance with contract requirements as to size of drainage area, availability of work space during testing period, accessibility and interferences, all as hereinafter discussed.

The outfalls into the Washington Channel and Anacostia River between the Twelfth Street and South Capitol Street Bridges were immediately eliminated because the sewer system in this area was in the process of being converted from a combined system to a separate system.

Elimination of the above group of sewers left two groups to be investigated. The sewers of Group 1 outfall into the Potomac River between Three Sisters Islands and the Arlington Memorial Bridge. The sewers of Group 2 outfall into the Anacostia River between the South Capitol Street Bridge and the John Philip Sousa Bridge.

Within Group 1 were further eliminated the outfalls between Rock Creek and the Arlington Memorial Bridge because they drained areas that were either too large or too small for the study, and they outfell on existing improved Park Service property. Again, to the West of Rock Creek, the elimination of four outfalls was concluded because of the size of drainage area served. The remaining outfalls that showed possibility in Group 1 then, were as follows: 21-inch outfall at 30th Street, 24-inch outfall at 31st Street, 24-inch outfall at Wisconsin Avenue, and a 4-foot by 4-foot outfall West of Key Bridge near 36th Street.

The outfalls at 30th and 31st Streets respectively did not serve large enough areas unless they were connected in some manner for the project. More important, however, interference from barges serving industry at these locations was anticipated, and there was indication that there would be a lack of work space available during certain phases of the project. These locations, therefore, were taken out of further consideration.

The two remaining sites of Group 1 are discussed herein in conjunction with any possible sites of Group 2 sewers after reviewing that group.

Within Group 2 along the Anacostia River, use of the outfalls within the U. S. Navy Yard area would be far less than ideal since another Government agency would be involved, not to mention ship interference, site accessibility problems, etc., so these locations eliminated themselves.

On the East side of the Anacostia River there were three combined sewer outfalls, but two of these were located within the turning basin of the U. S. Navy Yard and, therefore, would have been involved with interference from many more large ships and boats than upstream. In addition, all of the sites on the East side were on newly reclaimed Park Service property, and construction here for the pilot plant would not be desirable unless absolutely necessary.

The remaining four sewer outfalls of Group 2 were located on the West Bank of the Anacostia River between the Anacostia and Sousa Bridges. Of these, two of the outfalls served very large drainage areas and another served a very small drainage area; therefore, these were eliminated. The fourth sewer, outfalling closest to the Sousa Bridge,was found to be the best possibility of Group 2 sewers and will be discussed along with the two sites available from Group 1.

Three sites remained for final consideration and are referred to hereinafter as the "Key Bridge" site, the "Wisconsin Avenue" site, and the "Sousa Bridge" site; the first two being located on the Potomac River and the latter on the Anacostia River.

<u>Key Bridge Site (Group 1)</u>

a. The area drained by the sewer network is about seventeen acres at this site.

b. The depth of water in the Potomac River near the outfall of this sewer is about ten-feet.

c. The only use of the waterway in this area is that of the small boat and pleasurecraft.

d. Ample work space is available at this site as well as good accessibility.

e. It is assumed that property and right-of-way rights, as well as all required permits, will be readily available to the Contractor at this location.

f. There should be no problem in making the installation compatible to the surroundings at this site.

g. Highway: Future extensions of the Whitehurst Freeway as a depressed roadway or tunnel will remove any underwater installation from the River. This highway work could take place in about two or more years.

Wisconsin Avenue Site (Group 1)

a. The drainage area served by the sewer at this site is about eight acres.

b. The depth of water of the Potomac River at this sewer outfall is approximately 36-feet.

c. The waterway at this site is available to large boats and barges as well as small boats and pleasurecraft.

d. The work space available at this site is more than enough for the project needs and access is excellent.

e. There does not appear to be any problem in securing property and right-of-way rights or construction permits at this site.

f. An installation at this site could easily be made to fit into the surrounding area even considering proposed beautification of these surroundings.

g. Highway: The future extension of the Whitehurst Freeway will not interfere with the installation. It could, however, substantially alter the shape and area of the drainage basin served. This highway work could take place in about two years.

Sousa Bridge Site (Group 2)

a. The area of land drained by the sewers of this site is about 28-acres.

b. The indicated water depth at this site is thirteen-feet, however, increased depths upstream and downstream were noted.

c. No large boats are known to travel as far upstream as this site, however, the area is used extensively by small boats and pleasurecraft.

d. This site has ample work space available and access to the site poses no problem.

e. It was not anticipated that there would be any problem in securing the necessary right-of-way or property rights or permits for construction at this site.

f. There were no problems anticipated in making an installation at this site fit the surrounding conditions as they are now or may be in the future.

g. Highway: Design of the extension of I-295 is in the final stage, but the new road work would not interfere with the proposed installation. It is not likely that the shape and area of the drainage basin served would be substantially altered by this road work.

After a study of all the factors involved, it was recommended that the "Sousa Bridge" site on the Anacostia River be chosen as the location for the proposed water pollution control project. This site came closest to meeting the requirements for the required size of drainage area, at about 28-acres. Also, it was anticipated that because of the smaller size of the Anacostia River, the problems of river and tidal currents and their effects on the installation would be less than at the other sites. With all other criteria for the site being more or less equal at all locations, the other factors which influenced the final site selection were (1) anticipated lower installation costs; (2) a more evenly and well-defined drainage pattern; and (3) less interference from new highway construction.

Because the "Sousa Bridge" site was selected for the pilot plant does not preclude the other sites, either separately or by grouping, as not being adaptable to the underwater storage of combined sewer overflow.

PAGE NOT

AVAILABLE

DIGITALLY

SECTION 5

GENERAL

The pilot plant on the Anacostia River in Washington, D.C. was built under Contract No. 14-12-139 with the Federal Water Pollution Control Administration to determine the feasibility of storing overflow of combined sewers in inflatable tanks anchored to the bottom of the river bed. The entire project was installed under Permit No. 6:830:106 obtained from the National Capitol Park Service on a piece of property 50-feet by 105-feet adjoining the river at the Sousa Bridge. Application was made and approved by the U. S. Army Corps of Engineers for dredging and by the U. S. Coast Guard for installation of navigational aids. The construction of the pilot plant began in May, 1968, was completed in November, 1968 and was in continuous operation until October, 1969.

The overflow sewer selected served approximately thirty-acres of mixed residential and commercial area. The combined sanitary and storm sewer normally discharges into a 72-inch interceptor sanitary sewer, with overflow through a 33-inch line into the Anacostia River during heavy sewer flow. The site proved suitable conditions for dredging and installation of underwater storage tanks and anchorage system to stabilize the tanks.

Patents 3,114,384, 3,114,468, 3,155,280 and 3,187,793 issued to Harold G. Quase, assigned to Underwater Storage, Incorporated, were used for the storage system,tank cradle supports and anchorage.

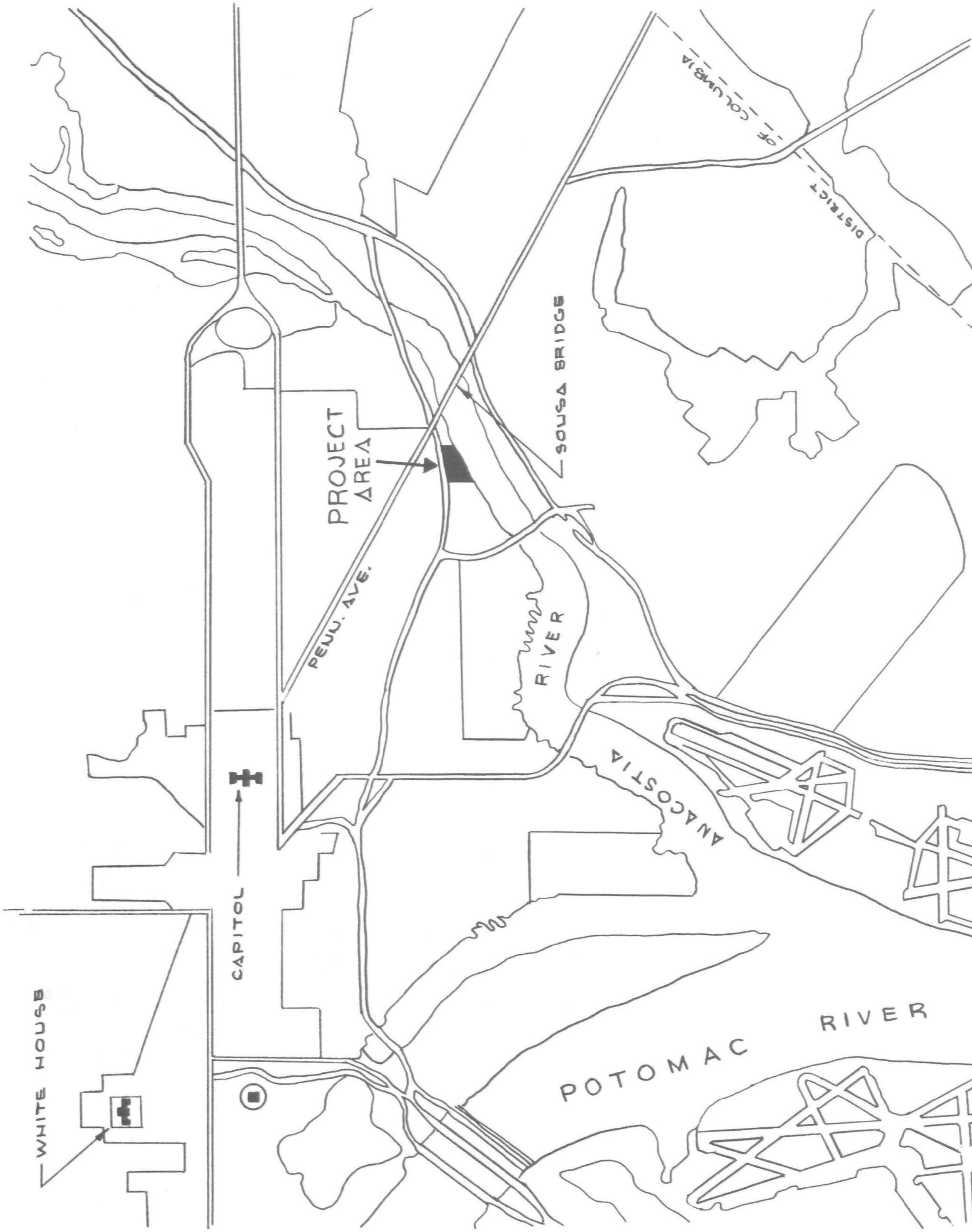
No interferences resulted to waterway users, ample work space was available and the site was readily accessible.

Two 100,000-gallon synthetic rubber-coated nylon fabric tanks were installed in the bed of the river through a patented system of anchors. The existing 33-inch diameter combined sewer overflow from a 30-acre site was diverted through a Parshall flume for purposes of measuring total flow. Flow to the underwater storage tanks passed through a concrete grit chamber where oil and grease floated to the top and grit, sand, etc., fell to the bottom. In addition, a motor-operated comminutor was provided in the flow channel to shred all solids down to 3/8-inch. Flow to the underwater tanks was by gravity. In the main header pipe from the comminutor to the underwater tank, a meter was provided to determine the actual flow in and out of each tank. When tank or tanks were filled, the remaining overflow was allowed to take its normal course of flow into the river. If 100% storage could have been provided, no flow would have been allowed to discharge into the river.

Between the hours of midnight and 8:00 A.M., after rainfall, when the sewage treatment plant operation was at a minimum, the underwater storage tanks were pumped out by the main sewage pump through a force main to discharge the contents into the interceptor sewer.

A good portion of the solids flowing into the underwater storage tanks remained in suspension and could be readily pumped out. The balance of solids was agitated by compressed air to assist in the pumping operation. The tanks were vented to the atmosphere and were protected from river debris by an underwater fence enclosure. A water pump was used for pumping river water into the tanks for test purposes during dry periods, and also for washing down water channels when desired. A sludge pump was provided to dispose of pumpable solids from the grit chamber.

To test the pilot plant during the operational period, meters and recorders were provided to measure total flow runoff of the overflow sewer and the amount of flow into and out of each tank. Facilities were provided for sampling of all flow for laboratory analysis for determination of BOD, pH, bacteria, TOC, settleable solids, suspended solids, coliform and industrial waste. Facilities were set up for automatic sampling in sterilized bottles which were refrigerated prior to testing.



Project Location Plan

Fig. 9

SCALE: 1"= 200'

Project Area Plan

Fig. 10

SECTION 6

DESIGN

After site selection was completed, a survey was made to determine conditions of the ground on both sides of the seawall. Soundings were also taken of the river bed.

From the boring log, it was determined that all structures would have to be erected on pilings, because of the ground and water conditions.

Design was predicated on anticipated and past records of rainfall and water flow in the 33-inch diameter District of Columbia combined sewer overflow line.

A Parshall flume 4-foot in size was selected to handle a minimum of 570 GPM and a maximum of 30,000 GPM at free flow. The existing 33-inch sewer would be diverted into the Parshall flume where meters would record flow in a meter house built above the flume. Leaving the flume, the system design would incorporate flexibility to divert the storm overflow to storage tanks or back to the outfall in the river.

A structure to house grit chambers, comminutor, supplemental chambers, sewage pump, sludge pump, river water pump, air compressor, and operators' console was designed of reinforced concrete, because of ground and water conditions. A United States Park Service directive would allow only five-feet of any structure above grade; design of this building was carefully plotted to keep within this directive.

Storage of 200,000-gallons of water in the river bed was mutually decided upon with provisions for future expansion to 300,000-gallons.

A complete piping system was then designed to take storm overflow from the Parshall flume to the grit chambers, and then from the comminutor discharge to the storage tanks through metering facilities. Water being pumped out of the storage tanks is piped back through the metering device into the sewage pump where it is then pumped uphill to a 6-foot interceptor sewer which flows to the sewage treatment plant. Grit chambers can be cleaned by pumping out through the sludge pump. River water can be pumped into the system for test purposes. An air compressor and a piping system to the tanks is used for agitation to prevent build-up of solids at tank base. All valving, pumping and metering are accomplished within one room.

A metering station was designed adjacent to the overflow manhole at Barney Circle (See Site Plan, Appendix A), which would record the height of all water running through the manhole. The first 8-inches of this water goes into the interceptor sewer, and all above 8-inches flows into the 33-inch storm overflow line.

Principal items of design were:

a. Storage Tanks: Flexible synthetic rubber-coated nylon fabric with a tensile strength of 1500 psi.

b. Pumps: Horizontal double volute self-priming centrifugal type. Pumps to have replaceable check valve, wearplate, impeller, and mechanical shaft seats. Motors to be non-overloading type.

c. Comminutor: Readily installable in a straight rectangular sewage channel. Replaceable steel cutters. Design for minimum flow of 4 MGD, average flow of 11 MGD, and a maximum of 19 MGD.

d. Air Compressor: Heavy duty, adjustable, complete with compressor, motor, controls, starter and tank. All A.S.M.E. approved.

e. Piping:

    1. Pump House: Schedule 40 black steel.
    2. Underground: Extra-heavy cast iron.
    3. Force Main: Mechanical joint.
    4. In River: Rubberized hose.

f. Slide Gates: Aluminum, rising-stem type.

g. Controls: Metering and recording equipment, Foxboro and Hersey-Sparling.

A steel cradle was designed to hold the tank and enable lowering tanks into river to be fastened to steel "H" piles driven to anchoring depth.

Basic to the design of a storage system to handle storm sewage overflow is the quantity of storage required and the necessary rate of acceptance. Statistical analysis of records for many years of rainfall of a single storm or rainfall per day, or rainfall per hour, will give the percent of total sewage overflow a given sized system will store and return for processing. This percentage could be considered the system efficiency if we believed that drainage and sewage mixed uniformly during a storm. This form of calculation would be of great value in the design of a final system and in the evaluation of the performance of a pilot plant; however, other considerations such as increased operating experience overshadow optimum design for pilot plants. With flow over short distances, the flow rates are determined by precipitation rates over rather short time periods such as ten minutes, particularly in an area such as Washington, D.C. that is subject to an average of some thirty odd thunder storms per year. The storm rain rates for the specific area considered were unavailable; therefore, the largest hourly rate of recent record was used to calculate the required flow rate. The resulting rate of just over 2 $feet^3$ per second per acre compared favorably with handbook values (p. 44, WPCF Design and Construction of Sanitary and Storm Sewers).

Design Parameters

The size of the system is determined by: the area served, drainage characteristics, rain rate and storm duration. Most of the system components, however, must be determined from rates of flow rather than total quantity. Discharge rates can be calculated for the defined service area of thirty-acres. Investigation of official weather records for the last twenty-years showed the greatest hourly rate to be 2.9-inches per hour; however, for computational purposes 3.0 will be used. Based on visual inspection of the area served, with regard to the amount of paved area and the imperviousness of the soil, the runoff factor was assumed to be 0.75. The average design flow rate (Q) into the storage system is calculated as:

$$Q = (\text{runoff factor})(\text{rain rate})(\text{sq.ft./acre})(\text{number of acres})$$

$$\begin{aligned} Q &= (0.75)(3/12)(43{,}560)(30) \\ &= 255{,}025 \text{ ft.}^3/\text{hr.} \\ &= 68.1 \text{ ft.}^3/\text{sec.} \end{aligned}$$

Return Pump Rate

The District of Columbia, Department of Sanitary Engineering established an operating procedure whereby the only permitted time for pumping out of storage tanks would be between the hours of 12:00 Midnight and 8:00 A.M. on the day following the subsiding of a storm. To compute the pumping rate to meet this requirement, it is assumed pumping to start at 12:00 Midnight, and end at 4:00 A.M. the following morning.

Total Volume	200,000 gallons (two tanks)
Pumping Time	Four hours, or 240-minutes

The required pump rate is:

$$\text{Pump Rate} = \frac{\text{Volume}}{\text{time}} = \frac{200,000}{240} = 830 \text{ gallons per minute}$$

A 1,000 gallon per minute pump was selected for the project. If a third storage tank were used, the additional pumping time would be:

$$\text{Time} = \frac{\text{Volume}}{\text{Pump Rate}} = \frac{100,000}{1,000} = 100 \text{ minutes}$$

Therefore, the time interval to pump three tanks would be from 12:00 Midnight to 5:40 A.M., well within the time allowable for pumping.

Return Pump

From observation of the proposed site, the following pump requirements were estimated: The suction would require an eight (8) foot lift when returning directly from the underwater storage tanks, allowing for a low tide. The discharge would require a net head of 41-feet to reach the interceptor sewer. Pipe losses of one foot of head were assumed on suction, and 20-feet of head on discharge, resulting in a total suction lift of 9-feet and a discharge head of 54-feet. The velocity head in the pipe was neglected. The return pump specifications are:

Capacity	1,000 gallons per minute
Suction Lift	9-feet
Discharge Head	61-feet

Sludge Pump

The sludge pump is used to pump effluent going through the floor drains of the comminutor room, out to the intercept sewer.

Capacity	200 gallons per minute
Suction Lift	5 -feet
Discharge Head	65-feet

Drains From Grit Chamber

These pipes are arbitrarily set at 4-inch diameter because of no flow requirements.

Water Pump

The water pump is used to pump water from the river to the storage tanks and the grit chamber for testing purposes. It is also used for washing purposes for the grit chamber and the storage tanks.

Capacity	200 gallons per minute
Suction Lift	5 -feet
Discharge Head	25-feet

Air Compressor

The air compressor is used to supply compressed air to the storage tanks through three parallel 3-inch pipes installed at the base of each tank with fifty 1/32-inch holes per 100-foot run on top of pipes. The purpose of the compressed air is to agitate the sludge in the tank to prevent settling of the sludge material.

Capacity	175 CPM (Actual)
	20 PSI, 25 HP
	213 CFM Piston Displacement

Sump Pump

The submersible sump pump is used to pump water from the floor of the grit chamber to the intercept main. This will prevent seepage water from accumulating.

Capacity	10 gallons per minute
Suction Lift	3 -feet
Discharge Head	67-feet

Return Line to Intercept Sewer

A length of 400-feet, change in elevation of 34-feet and a head loss of 20-feet, requires the installation of a 6-inch diameter return line. This 6-inch return line is installed inside the 33-inch overflow sewer up to the manhole near Barney Circle. From there it is buried underground and connected to the intercept sewer.

Anchor Forces

From the dimension of each storage tank, which is 124-feet by 24-feet by 5.5-feet, and a maximum river velocity of three knots (5-feet per second) (based on information obtained from the Corps of Engineers) the uplift force on each tank is as follows:

Hydrodynamic Lift:

$F_L = C_L A p\, V^2/2$ from Fluid Mechanics by Richard Pao

Where:

C_L = Coefficient of Lift
A = Area of Bottom Surface of Tank in Square Feet
p = Density in Slugs per foot^3 = 1.94 for fresh water
V = Maximum Current Velocity

$C_L = 2\pi \sin a$ from lift characteristics of a typical Joukowsky profile as found in Mechanics of Fluids by Hunter Rouse

a = The Angle of Attack of the Current on the Storage Tank

$$F = (2\pi \sin 15°)(124 \times 24)(1.94)(5)^2/2$$

$$= 117{,}500 \text{ pounds}$$

Buoyancy Lift (Based on 10% Buoyance of Tank):

F_B = 10% x V x D

Where:

V = Volume of Tank in Cubic Feet
D = Specific Weight of Water in Pounds per Cubic Foot

F_B = 10% (124 x 24 x 5.5) (62.4)
= 102,000 pounds

Total Uplift = Hydrodynamic Lift plus Buoyancy Lift
= 117,500 pounds plus 102,000 pounds
= 219,500 pounds per tank
= 109,750 pounds per side
= 885 pounds per foot

Three rows of seven (7) piles were driven. The two outboard rows each served one side of a tank and the center row served the load from both tanks. 8BP36 steel piles were used throughout.

Area of pile = 10.6 square inches

Maximum pile capacity at 20 kips per square inch
= 212 kips (tension)

Length of pile 40-feet; 37-feet was driven into river bed.

Based on report dated March 6, 1968 obtained from Schnabel Engineering Associates, Foundation and Soil Mechanics Consultants, Washington, D.C., the cohesion for the initial 24-feet of pile would be expected to be 300 pounds per square foot and for the lower 13-feet of pile would be expected to be 1,500 pounds per square foot.

Pile Surface	=	4 by 8-inches		
	=	32-inches		
	=	2.67-feet		
Initial 24-feet	=	2.67 x 0.3 x 24-feet	=	19.2 kips
Final 13-feet	=	2.67 x 1.5 x 13-feet	=	52.0 kips
		Total	=	71.2 kips

Seven (7) piles per side:

Force $= \frac{124\text{-feet}}{6\text{-spaces}} \times 0.885$ k (uplift)

$=$ 18.3 kips per pile

Safety Factor (outer row of tank)

$= \frac{71.2}{18.3}$

$= 3.9$

Safety Factor (inner row of tank)

$= 1.95$

Drag:

$F_D = C_D A \rho V^2/2$ from Fluid Mechanics by Richard Pao

Where:

C_D = Coefficient of Drag

A = Frontal Area of Tank in Square Feet

P = Density in Slugs per Foot^3

V = Maximum Current Velocity

C_D = 1.20 based on Value of L/D = 124/24 = 5.2 from Mechanics of Fluids by Hunter Rouse

$F_D = 1.20\ (24 \times 5.5)(1.94)(5)^2/2$

$=$ 3,850 pounds per side

F_D (Two Tanks) = 7,700 pounds

With 7 Piles = 1,100 pounds per pile

Based on cantilever of 6-feet on pile:

Momentum = 1,100 x 6
= 6,600 pounds per foot
= 79,200 pounds per inch

Stress = 79,200 pounds per inch/29.9 (section Modulus of pile)
= 2,640 psi

$$\frac{2{,}640 \text{ psi (actual stress)}}{20{,}000 \text{ psi (allowable stress)}} = 13.2\%$$

This is satisfactory, since actual stress is less than 15% of allowable stress.

Stresses on Piles:

Bending Stress = 2,640 psi

Tension = $\dfrac{18.3 \text{ kips/pile} \times 2 \text{ (tanks)}}{10.6 \text{ square inches (area of pile)}}$

= 3,450 psi

Parshall Flume

A Parshall flume is a specially shaped open channel flow section which is installed to measure the rate of flow of water. Since it was estimated that a maximum flow of 68-feet^3 per second would exist at the overflow, a 4-foot Parshall flume was chosen to be the most economical for the project.

There are two water levels recorded in the Parshall flume; one upstream and the other downstream. For a free flow, the upstream head (H_a) will be the determining factor, while the downstream head (Hb) will have a zero reading. However, whenever there is backed-up water from downstream, submergence occurs. This occurs when the water surface downstream from the flume is far enough above the elevation of the flume crest to reduce the discharge. Parshall flumes tolerate 50% to 80% submergency before the free-flow rate is measurably reduced. Submergency ratio is the ratio of downstream reading Hb to upstream reading Ha, or Hb/Ha.

During free flow, the discharge depends solely upon the width of the throat, W, and the depth of water, Ha, at the gauging point. Calibration tests show that the discharge is not reduced until the submergency ratio Hb/Ha, expressed in percent, exceeds 70% for a 4-foot flume. When submergence occurs, a correction factor must be subtracted from the free flow discharge rate. For all practical purposes, the flow rate may be determined from a table (U. S. Department of the Interior, Bureau of Reclamation, Water Measurement Manual , Department of the Interior, Denver, Colorado, second edition, 1967, pp. 274-277.) specifying the head reading Ha and a throat width of 4-feet.

The range of a 4-foot Parshall flume is 1.26 to 62.93-feet^3 per second. This rate corresponds to head readings of 0.20-feet and 2.50-feet respectively. Readings outside this range will not be accurate, and also are beyond the range of the table. Therefore, for a 4-foot flume, the minimum Ha reading should be at least 0.20-feet.

Unfortunately, during much of the time, the readings in the Parshall flume were less than 0.20-feet. Since extrapolation would be erroneous, these lower readings had to be disregarded, since only readings 0.20-feet and above were considered. There was no other way to determine the rate, unless, of course, a Parshall flume smaller than one-foot were constructed. However, the upper range of this smaller Parshall flume would then be drastically reduced. It is interesting to note here that for 1-foot to 4-foot flumes, only readings 0.20-feet and above are considered.

One factor that confuses readings in the Parshall flume is the presence of backed-up water in the float pipes for the Parshall flume recorders. This back-up may be caused by stagnant water, ground seepage, or other factors such as tide water and waves from boats outside on the river lashing back into the seawall and flowing back into the sewer pipes. It is, therefore, difficult to differentiate between these factors as to which is causing readings on the Parshall flume. One characteristic of waves from boats outside is that this often occurs during Saturday afternoons and Sundays.

One way to measure the amount of water going into the comminutor, and hence to the underwater tanks, is to determine from the reports exactly when the 24-inch butterfly valve was opened to let water into the tank. This time could then be compared against the Parshall flume chart to determine the total amount of water that is let into the tanks.

Since the Parshall flume readings contain data from factors other than storm overflow, one way to avoid backed-up water and to make sure that the reading is only from storm overflow, is to put a one-way gate where the sewer pipe meets the seawall. This would prevent waves or tide water from flowing back into the sewer pipe.

Table IV indicates the flow through the Parshall flume in cubic feet per second and in gallons per minute for corresponding head reading at the upstream water level and no submergence. For manual operation, to assist the operator, time is indicated for filling each 100,000-gallon underwater storage tank for the various flow rates.

TABLE IV

TIME IN MINUTES
NEEDED TO FILL 100,000-GALLON TANK

4-Foot Parshall Flume Reading (Feet)	$Ft.^3/Sec.$	GPM	Time Needed (Minutes)
0.20	1.26	566	177
0.25	1.80	809	124
0.30	2.39	1,075	93
0.35	3.06	1,375	73
0.40	3.77	1,695	59
0.45	4.54	2,040	49
0.50	5.36	2,410	42
0.55	6.23	2,800	36
0.60	7.15	3,220	31
0.65	8.11	3,640	27
0.70	9.11	4,090	24
0.75	10.16	4,570	22
0.80	11.25	5,050	20
0.85	12.38	5,560	18
0.90	13.55	6,080	16
0.95	14.76	6,640	15
1.00	16.00	7,190	14
1.05	17.28	7,760	13
1.10	18.60	8,350	12
1.15	19.94	8,960	11
1.20	21.33	9,540	10.5

NOTE: This Table is applicable only when the Parshall flume is not either fully or partially submerged.

SECTION 7

PLANT EQUIPMENT

Parshall Flume: Leopold "Leo Lite" for channel flow measurement.

Gates: Aluminum gates were provided for flow control at grit chamber and at bar screen. Inlet to the plant was controlled by a Lunkenheimer butterfly valve.

Comminutor: Worthington Corporation type 36-C unit to operate continuously and automatically to screen, cut or shred coarse solids directly in the flowing raw sewage without requiring removal of the screenings from the channel. The comminutor was provided with a two horsepower vertical, squirrel cage induction motor wound for 208 volts, three phase, 60 cycle and is of size for flow range of 5 MGD to 19 MGD. The comminutor consists of stationary, vertical semi-circular screen with horizontal slots and an oscillating cutter that passes through a stationary cutter or cutters located along the screen for the purpose of comminuting sewage screenings. The screenings were reduced to a size that would permit them to pass through the screen slots. The rate of comminution was such that no undue accumulation of screenings would result on the screen and cause excessive upstream head conditions. The comminutor was provided with permanent oil lubrication. The screen consisted of replaceable steel sections and the comminutor was so designed that the cutters could be removed for sharpening or replacement. The stationary cutters are adjustable so that proper rubbing contact may always be maintained. See Fig. 14.

Bar Screen: Manual screen comprised of 2-inch by 1/4-inch bars, 1-inch on center was provided for use in the event comminutor was out of service.

Grit Chamber: A concrete structure to slow down the velocity of flow and to remove sand and heavy particles by settling. Oils and floating materials could be skimmed off the surface. For the purpose of this demonstration this was accomplished by hand. In a full scale project this could be mechanized.

Sewage Pumps: Gormann Rupp self-priming centrifugal pumps of horizontal double volute construction, fitted with replaceable suction check valves, replaceable wear plates, open type impeller and double face grease lubricated mechanical shaft seal. The sewage pump has a capacity of 1000 gpm at 70-foot total head and 9-foot lift. It is equipped with a 40-horsepower, 1750 rpm, three phase, 208 volt motor. Pumps are shown in Figures 11 and 12

The water pump has a capacity of 200 gpm at 30-foot head and 5-foot lift. It is equipped with a three horsepower, 1750 rpm, three phase, 208 volt motor.

The main pump was used for pumping out the storage tanks to the interceptor sewer. The sludge pump, designed for solids handling was used for pumping out grit chambers to the interceptor sewer. The water pump was used to provide river water for test purposes, and for wash down. This pump could also be utilized for flushing of the underwater storage tanks.

Sump Pump: Piqua submersible pump has a capacity of 10 gpm at 70-foot head. It is equipped with a single horsepower, three phase, 208 volt, 60 cycle motor, and a level sensor to automatically control operation. This pump was installed in a pit and was designed for pumping out any leakage on pump house floor.

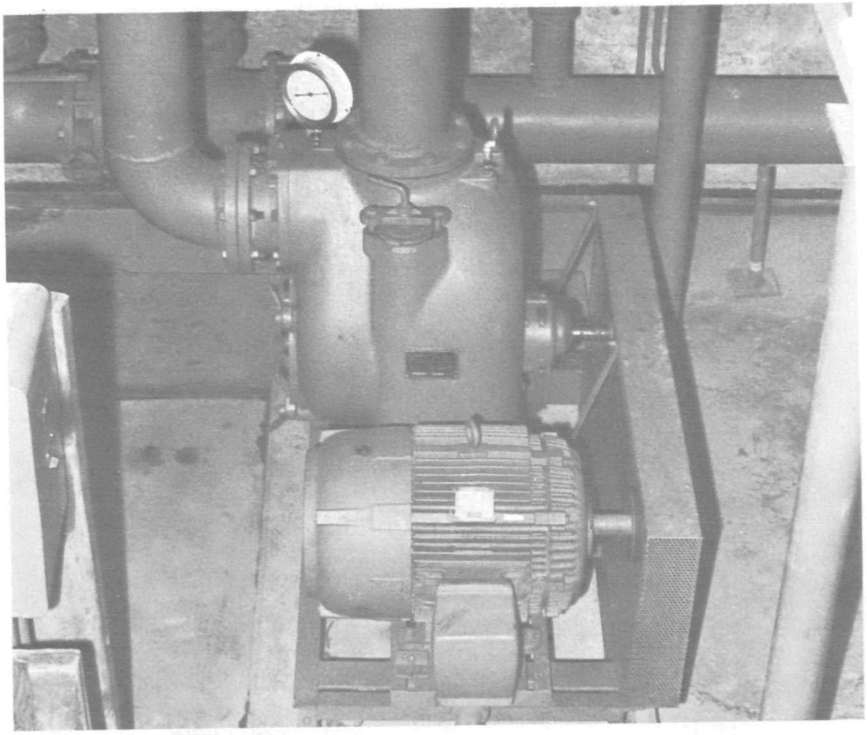
Air Compressor: Worthington Corporation package radial, air cooled vertical, single stage, belt driven compressor has a delivery of 175 CFM at 20 psi and is equipped with a 25-horsepower, 1800 rpm, three phase, 208 volt motor. The compressor provided plant air for instrumentation and for agitation of underwater storage tank contents. Figure 13 shows air compressor.

Exhaust Fans and Louvers: Manufactured by Power Line Fan Company were provided for forced ventilation in pump house and in meter house.

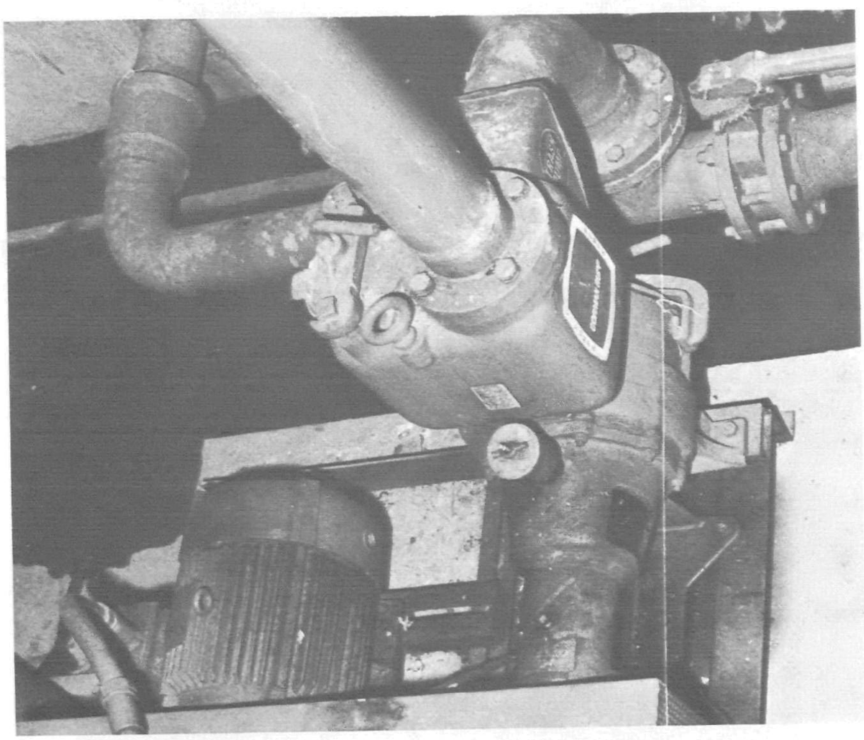
Unit Heaters: Berko electric unit heaters were provided in pump house. Control was provided by remote thermostats.

Electric Service Equipment: Equipment provided consisted of Square D panelboards, Crouse Hinds heavy duty switches, plugs and lights.

Instrumentation: Hersey-Sparling flow meters, transmitters and recorders, and Foxboro measuring station. Belfort Instrument Company continuous recording twelve-hour rain gauge. Figure 17 shows forward and reverse meter installation in main pipe header line in pump house. Figure 18 shows meter recording station.

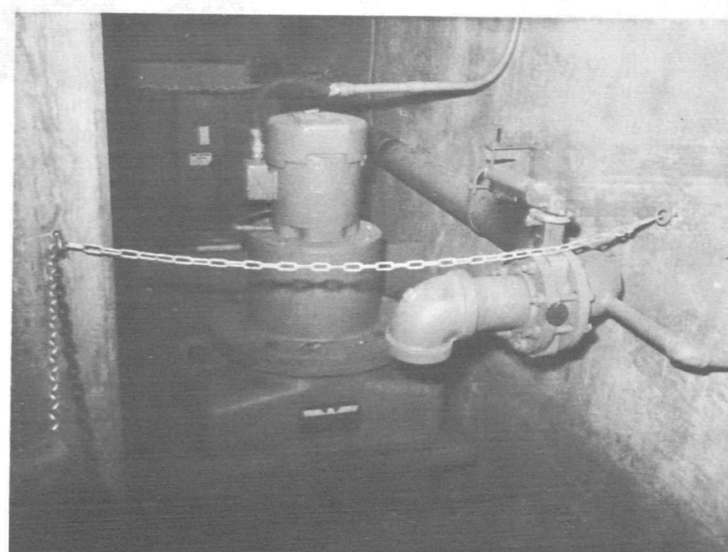


Main Sewage Pump Fig. 11

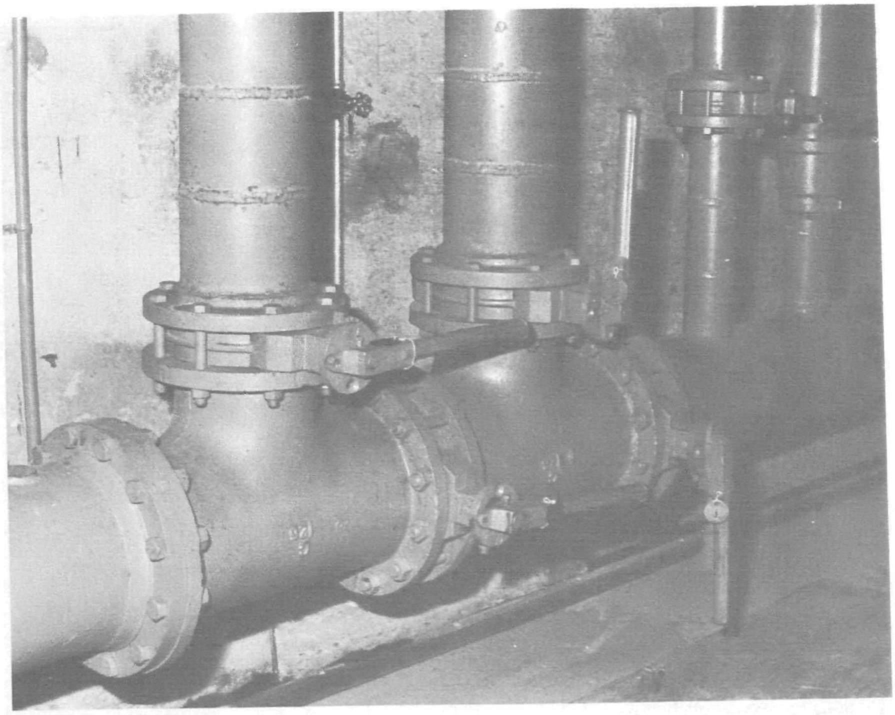


Sludge Pump Fig. 12

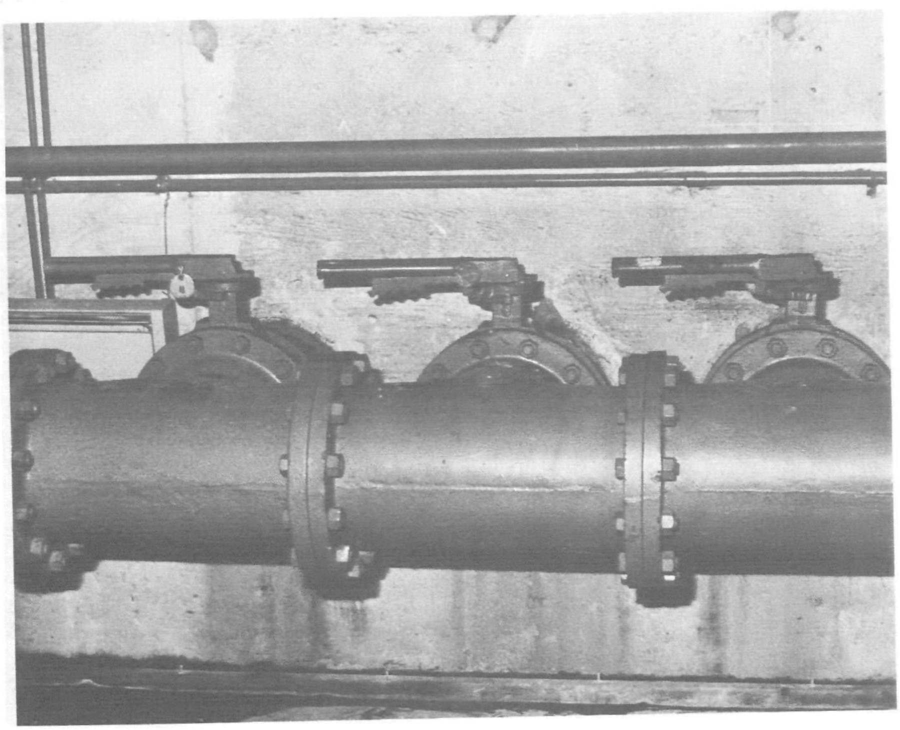
Air Compressor Fig. 13



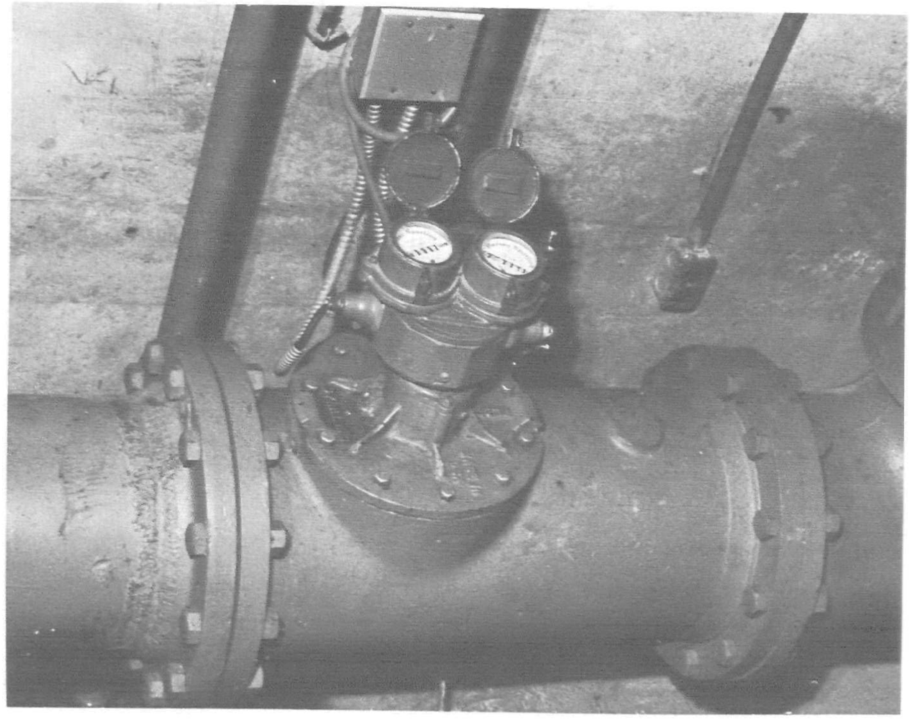
Comminutor Fig. 14



Main Header Line from Comminutor Chamber
Showing Pump Bypass Line Fig. 15

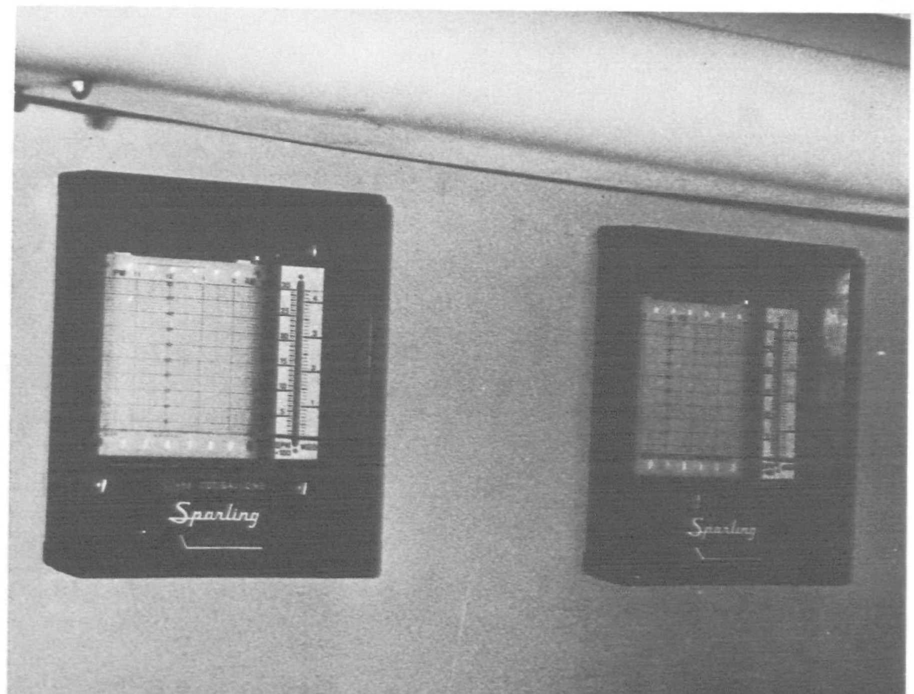


Main Header Line Showing Branch Runouts
to Underwater Storage Tanks Fig. 16



Forward and Reverse Meter Installation
in Main Header Line

Fig. 17



Forward and Reverse Meter Recording
Station

Fig. 18

Hose: Goodyear Rubber Company "Diversipipe" with flanged ends was used in water for main sewage flow. Small diameter hose for vent and air lines was Goodyear "Ortac" with quick acting couplings matching connections at the underwater storage tanks.

Piping: Piping in pump house is Schedule 40 black steel, ASTM 120. Underground piping is extra heavy cast iron. Compressed air piping installed in the underwater storage tanks is PVC, Schedule 40 perforated with ends plugged. See Figures 15 and 16 for pump house.

Underwater Storage Tanks: In order to obtain shortest delivery period to meet stringent construction time schedule, standard pillow shaped tanks were utilized. The synthetic rubber-coated nylon fabric tanks were furnished by the Goodyear Tire and Rubber Company. These tanks are an adaptation of the tanks that have been used by the United States Air Force for 25-years for oil storage.

The general specification of the pillow tank falls within MIL-T-12260B, Amendment No. 1, with specific requirements being as follows:

A. Physical Characteristics of the Coated Fabric

1.	Weight, ounces per square yard	60 + or - 5.0 oz.
2.	Breaking strength, pounds per inch minimum both warp and fill	600 by 600 pounds
3.	Adhesion (cloth to compound) pounds per inch minimum	20 pounds
4.	Tear strength, warp and fill minimum	20 by 20 pounds
5.	Coated fabric thickness	.065 + or - .005 inches

B. Physical Characteristics of Coating Compound

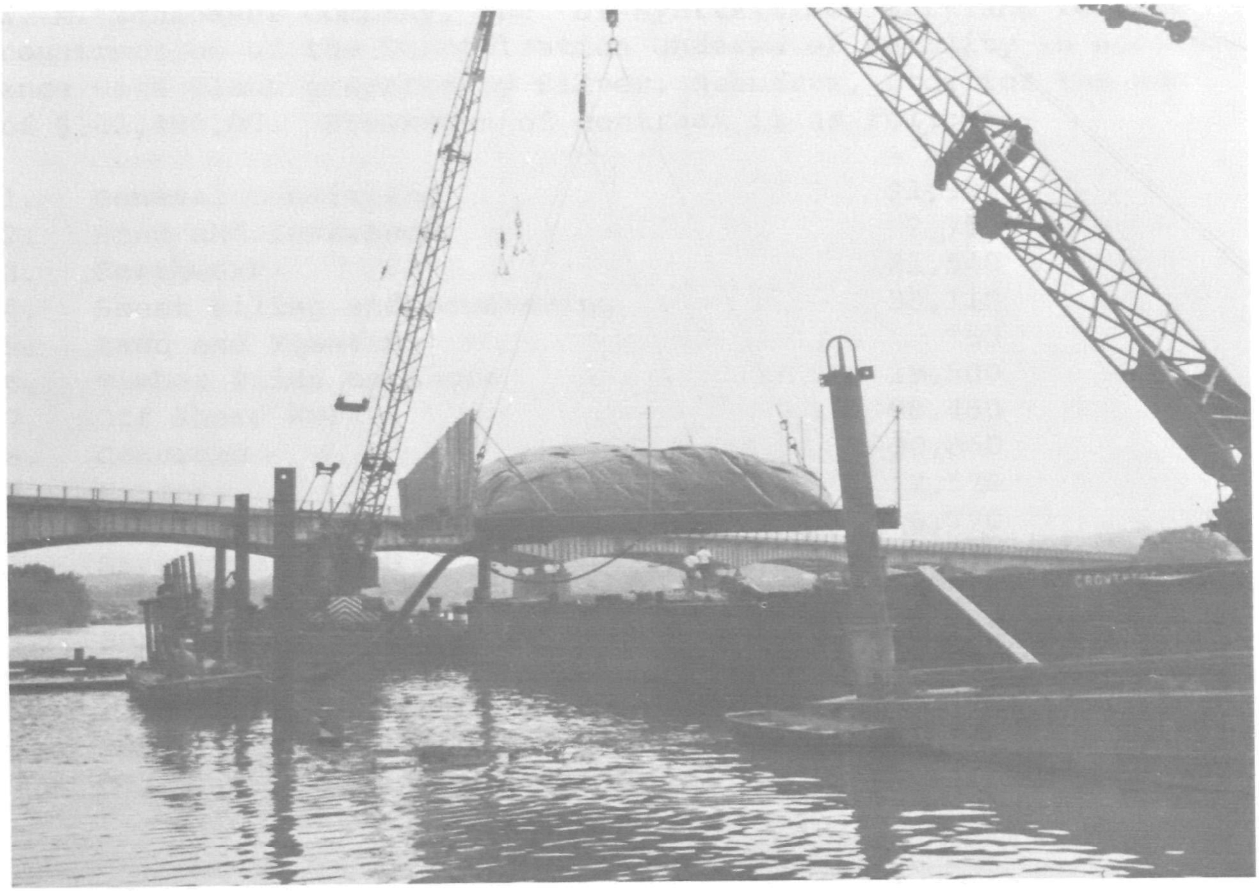
1.	Tensile Strength a. Original psi minimum	1500 psi
2.	Ultimate elongation	300%
3.	All seams have a tensile strength equal to the tensile strength of the coated nylon fabric.	

Each 100,000-gallon pillow tank was equipped with a 10-inch flanged stainless steel inlet connection which served as both the inlet and outlet fitting. This fitting was located on the bottom of the tank. Also located on the bottom of the pillow tank was a 1-1/2-inch fitting for connection to the 3-inch PVC aeration tubes. Air was pumped through these tubes to keep any solids from settling to the bottom of the pillow tank. On the top surface of each was located a 1-1/2-inch stainless steel vent fitting. This was later changed to six (6) 1-inch vents as described hereinafter under Section 9, Operational Description. A 20-inch stainless steel manhole located on top of the tank allowed easy access into the tank. All connections on the tank were connected to the on-shore fittings with rubber hose.

The pillow tanks were strapped to a fabricated steel cradle for permanent installation. The cradles were attached to steel piles driven to bed rock on the bottom of the river by a patented system of anchors to a depth of 37-feet below the river bed.

Installation of the pillow tank was easily accomplished. Nearly 1,100 yards of rubber fabric was used in the manufacture of the tanks. The shipping crate measured 140-inches by 54-inches by 34-inches and weighed approximately 3,000 pounds. The pillow tank was located in the center of the cradle and unrolled. Connections were made, the hold-down straps connected and the tank was ready for lowering and attachment of the cradle to the piers. Each tank was 124-feet long by 24-feet wide and 5-1/2-feet high when inflated. Tanks used for this project cost $14,400.00 each. Cost included all fittings on the tank, including manhole, to meet the requirements of the pilot plant.

Figure 19 shows one of the underwater storage tanks held in place by cranes ready for submergence.



Underwater Storage Tank Ready for Submergence

Fig. 19

SECTION 8

CONSTRUCTION COSTS

A contract was entered into on May 10, 1968 with the low bidder, W. M. Schlosser Company, Inc. of Hyattsville, Maryland for the construction of the Demonstration Underwater Facility in accordance with plans prepared by Silver, Schwartz, Ltd. for the sum of $341,486.00. Breakdown of contract is as follows:

1.	General Conditions	$1,900
2.	Bond and Insurance	2,750
3.	Earthwork	21,540
4.	Sheet Piling and Dewatering	33,110
5.	Lawn and Planting	750
6.	Timber Piles on Shore	19,500
7.	Off Shore Work	68,450
8.	Concrete	30,860
9.	Masonry	2,576
10.	Miscellaneous Metal	6,770
11.	Structural Steel	11.098
12.	Miscellaneous General Construction	9,865
13.	Painting	4,825
14.	Mechanical Equipment	47,942
15.	Pipe and Fittings	16,600
16.	Electrical	5,500
17.	Off Shore Tanks, Cradles, Hoses and Belts	57,450
	Total	$341,486

1. General Conditions: Includes construction sign, temporary toilet, telephone and field office.

2. Bonds and Insurance: Includes performance and payment bonds as required to meet contract requirements.

3. Earthwork: Includes all shore excavation and backfill.

4. Sheet Piling: It was necessary to sheet pile the entire shore construction area because of latent conditions observed at the site.

5. Lawns and Planting: A requirement of the National Park Service was to maintain green area around the project.

6. Timber Piles on Shore: Required for the entire construction because of soil conditions.

7. Off Shore Work: Includes dredging, installation of steel piles, setting of tank and cradle assemblies.

8. Concrete: Includes all work necessary for construction of flume, channels, grit chambers, pump house and meter house.

9. Masonry: Includes structure over meter house.

10. Miscellaneous Metal: Includes circular stair to pump house, gratings and louvers.

11. Structural Steel: Includes cost of steel piles in river, and tank anchorage system.

12. Miscellaneous General Conditions: Includes waterproofing of structures, caulking as required, hollow metal and hardware.

13. Painting: Includes painting of exterior of pump house to fulfill National Park Service requirements, painting of pumps, piping and equipment.

14. Mechanical Equipment: Includes comminutor, bar screen, pumps, meters, compressor, instrumentation, channel gates, flume liner, automatic sampler.

15. Pipe and Fittings: Includes all lines and valves in pump house for sewage flow, force main, compressed air, water.

16. Electrical: Includes all lighting, wiring, conduit and outlets.

17. Off Shore Tanks and Belts: Includes underwater storage tanks, cradles, piping and hoses in river and web belting around tanks.

Construction Change Orders - Charges

1.	Addition of Manhole	602.51	
2.	Supports and Modifications to Tanks	15,472.00	
3.	Added Dredging	2,730.00	
4.	Cable Instrumentation	465.46	
5.	Added Grading	3,378.00	
6.	Spar Buoys	950.00	
7.	Dock Repair	450.00	
8.	Sewer Changes	3,000.00	
9.	Chains and Closures	185.00	
10.	Signs	117.42	
11.	Sodding	405.00	
12.	Modifications to Pillow Tanks	3,550.00	
13.	Raising and Lowering of Pillow Tanks	21,896.00	
	Total Charges		$53,201.39

Construction Change Orders - Credits

1.	Deleted Manhole	1,731.50	
2.	Deleted Concrete Wall Section	45.00	
3.	Butterfly Valves and Aluminum Gates	4,778.00	
4.	Deleted Parging	144.00	
5.	Deleted Explosion-Proof Motor	40.00	
	Total Credits		$ 6,738.50
	Net Construction Changes		$46,462.89

Explanation of Charges

1. Added Manhole: A manhole was added over the baffle section of the Parshall flume in order to provide a cleanout for anticipated build-up of sewage and rubbish.

2. Tank Supports and Modifications: Includes various items of concern during installation:

a.	Added a 24-gauge galvanized decking in tank cradles to support storage tanks	$5,600
b.	Added sprayed urethane on metal decking of pillow tanks and around perimeter of steel cradle to protect against pillow tanks rubbing against decking and sides during inflation and deflation	6,005
c.	Provided a six-foot high chain link fence and posts around cradle to protect against river debris from damaging pillow tanks	3,027
d.	Field conditions necessitated installation of brackets to support ten-inch pipe connections at pillow tanks to prevent sagging through bottom of galvanized decking and installation of suspension cables for tank manholes to support weight of manholes when tanks are in a deflated condition	840
	Total	$15,472

3. Added Dredging: The original contract was based on 700-cubic yards of dredging. Actual dredged material amounted to 1,225-cubic yards. The charge is based on the additional dredging of 525-cubic yards at $5.20 per cubic yard.

4. Instrumentation: Revisions were made to instrumentation for transmitting flow readings to the pump house.

5. Added Grading: Additional backfill and grading was required when it was decided to extend the grading line around the pump house structure beyond property limits.

6. Spar Buoys: Two spar buoys were installed as per Coast Guard requirements.

7. Dock Repair: It was necessary to repair the existing dock and to provide safety rails for the safety of visitors to the project.

8. Sewer Changes: Revision was made to the force main from the pump house to the interceptor sewer manhole.

9. Chains and Closures: It was necessary to add posts and chains around the comminutor well to provide safety for visitors to the installation and to provide closures at the bar screen and comminutor to prevent solids from entering underwater storage tanks.

10. Signs: Three "No Trespassing, Government Property" signs were provided at the site.

11. Sodding: An additional 450-square yards of sodding was provided beyond limits as required by the National Capitol Park Service.

12. Modifications to Pillow Tanks: A 100-foot long wire mesh screen in the form of cylinders approximately 12-inches in diameter was installed down the center of each underwater storage tank for the purpose of preventing the collapse of the tank over the suction inlet during a pump out period. In addition, various air vents were installed along the top of the tanks to improve the air relief system.

13. Raising and Lowering of Pillow Tanks: During construction, it was noted the inboard pillow tank was physically wider than the outboard tank. Time did not permit obtaining a replacement unit. When this tank was lowered into place in the river bed, it was noted that the top protruded out of the river during periods of low water condition. It was also noted that the belts had apparently slipped and twisted, creating a potential pressure problem on the surface of the tank. In a meeting with representatives of Federal Water Pollution Control Administration, it was decided to raise the inboard tank to visually inspect the tank and determine if any damage resulted.

Following this inspection, it was found that corrective measures were necessary to both tanks as outlined hereinbefore in Change Order 12. In addition, all belts were realigned and adjusted. Breakdown of costs for raising and lowering tanks is as follows:

a.	Crane and Barge Rental $1,744 per day for nine days	$15,696
b.	Cost to Unload Barge of Steel Piles Prior to Bringing the Barge to the Site	500
c.	Overtime for Crane Personnel	300
d.	Tug Rental, Two days at $450	900
e.	Divers Cost, Nine days at $500	4,500
	Total	$21,896

Explanation of Credits

1. Deleted Manhole: Manhole shown on plans adjacent to railroad property was deleted since it interfered with railroad communications system.

2. Deleted Concrete Wall Section: It was found that concrete wall in grit chamber was not required for the operation of the plant.

3. Butterfly Valves and Aluminum Gates: Substituted butterfly valves and aluminum gates for the Armco gates originally specified because of delivery problems.

4. Deleted Parging: Parging on concrete walls below grade at comminutor and pump house was not required since pitch was provided.

5. Deleted Explosion-Proof Motor: Substituted explosion-proof comminutor motor with a fan cooled weatherproof motor because of problems in delivery schedule.

TABLE V

PILOT PROJECT COST

1.	Design and Construction		
a.	Detailed Plans and Specifications	$20,364	
b.	Field Engineering and Inspection	30,750	
c.	Design Consultants	6,652	
d.	Construction	341,486	
e.	Construction Change	46,463	
f.	Plant Modifications	24,630	
g.	Project Management	71,260	
	Sub-Total		$545,475
2.	Plant Operation and Maintenance		
a.	Plant Labor	25,800	
b.	Plant Supervision	5,526	
c.	Electric Power	1,271	
d.	Plant Materials	7,538	
	Sub-Total		40,135
3.	Test and Evaluation		
a.	Program Management	21,050	
b.	Program Planning	3,250	
c.	Testing and Data Collection	4,740	
d.	Laboratory Analyses	5,982	
e.	Evaluation	3,820	
f.	Reports	13,376	
	Sub-Total		52,218
4.	Fee		30,000
	Total Program Cost		$667,828

NOTE: These figures are actual costs expended for this pilot installation which includes those items not normally reflected in an actual operating installation.

SECTION 9

OPERATIONAL DESCRIPTION

During a severe rainstorm, the water level in the combined sewer manhole rose above 8-inches. In this case, sewage automatically overflowed into a 33-inch line and thence into the Anacostia River. A recorder measured the height of the water at the manhole.

Overflow was diverted through a 4-foot Parshall flume for flow measurement. Automatic recordings of the flow rate were made continuously, 24-hours a day.

On the downstream end of the Parshall flume, an inlet line was installed to allow a portion of the overflow to enter the grit chamber of the pilot plant. When the meter in the line to the underwater storage indicated full capacity in the tanks, the 24-inch butterfly valve at the inlet to the grit chamber was closed, and the influent allowed to flow freely into the Anacostia River. During dry weather, the 24-inch butterfly valve was placed in the open position, ready to accept the next wet weather flow.

Inside the grit chamber, particles were reduced by the comminutor to a maximum of 3/8-inch. However, when the comminutor was not in use, particles 2-inches and larger were removed by a bar screen. Drains allowed settled materials in the grit chamber to flow to a sludge pump, discharging into a new 6-inch force main inserted inside the 33-inch main to the interceptor sewer.

Liquid from the grit chamber flowed by gravity into the underwater storage tanks. Liquid entering or leaving the storage tanks was metered and recorded. After the storm subsided and during non-peak hours, liquid was pumped from the tanks by the sewage pump into the interceptor sewer.

Storm water in excess of the pilot plant capacity of storm water overflow, when the system was shut down, took its normal course of flow through the existing tide gate into the Anacostia River.

In order to prevent undue settlement of solids in the storage tanks, compressed air was forced into the tanks for the purpose of agitating any settled sludge and to enable possible pumping out of all the contents of the storage tanks to the interceptor main.

The facility was manually operated to fill and empty the underwater tanks. Samples of effluent from the sewer overflow and from the tank discharge to the interceptor were taken at frequent intervals during the operational period.

The samples were analyzed for the following characteristics:

a. pH
b. Suspended Solids
c. Suspended Solids - Volatile
d. Settleable Solids
e. Biochemical Oxygen Demand
f. Coliform Bacteria
g. E Coli Bacteria
h. Petroleum Products

Volumetric data was taken, recorded and analyzed at the following points:

a. Storm water flow passing through the 33-inch overflow line.
b. Water volume passing through Parshall flume.
c. Water volume passing into or out of pump house.

The quantitative and qualitative data obtained by metering and analysis at the facility by the rain gauge installed in the drainage area, and the instrumentation installed in the sewers and facility was assembled and analyzed to record the effects of rainfall on the 33-inch outfall and the ability of this facility to minimize discharge of storm water overflow to the Anacostia River.

Testing without rainfall was also accomplished by introducing river water into the system by means of a water pump. These tests approximated tests during rainfall.

General Operating Procedure

1. Rain charts were changed every morning if it had rained the day before. If no rain was recorded, the charts were changed every third day.

2. Dates were placed on all charts every day at 12:00 Midnight.

3. When sewage pump was used, electric meter reading was recorded.

4. When overflow sewage reached the Parshall flume, the operator was instructed to assure that the 24-inch valve was in open position and to allow gravity flow to the underwater storage until Parshall flume recorders read "zero" if tanks did not fill up, or to 100,000-gallons per tank, whichever came sooner.

5. Grit chambers were cleaned after each rain. Caution was taken to make sure comminutor was turned off while cleaning.

6. Samples were taken according to the Parshall flume chart. Two bottles were taken on each maximum and minimum point on recorder.

Plant Operating Procedure

Operating procedure outlined herein was furnished to plant personnel. It can be followed on Figure 20.

1. Check and read 10-inch meter for forward reading and start recorder. Note on recording charts date and time of starts.

2. Open valves 1, 3 and 6. Make sure valve 7 is closed.

3. Open gates 16, 18 and 21 in comminutor room.

4. Make sure that 24-inch chain operated butterfly valve is in open position to allow storm water to enter building.

5. Start comminutor.

6. Gates 20 and 22 should be closed when comminutor is in operation.

7. When meter and recorder reads 100,000 gallons more than original reading, close valve 6.

8. Check and read 10-inch meter and recorder for forward reading.

9. Open valve 7 for second tank.

10. When meter and recorder reads 100,000 gallons more than second reading, close valves 7 and 1.

11. Close 24-inch chain operated valve and turn off comminutor.

12. During the filling procedure, either first or second grit chamber may be used. When comminutor is out of service, third chamber may be used for manual screening of particles in the storm overflow.

13. After filling procedure is completed, grit chambers shall be cleaned. For cleaning water, 4-inch river water pump shall be used.

14. Before starting 4-inch pump, open valves 12 and 13.

15. Start river water pump.

16. Any one hose bibb in the grit chamber area may be used for cleaning.

17. In order to get rid of dirt, gravel, etc., in grit chambers, sludge pump shall be used.

18. To clean first grit chamber, open valves 9, 11 and start sludge pump.

19. To clean second grit chamber, open valves 10, 11 and start sludge pump.

20. When grit chambers are clean, turn off sludge pump and close valve 9 or 10. Valve 11 may be left open at all times.

21. Close all gates and valves previously opened.

22. Before discharging the liquid from tanks, start compressor and open air valves 24 and 25 for agitation.

CAUTION: Discharging of liquid line into District of Columbia sewers shall only be accomplished from 12:00 Midnight to 6:00 A.M.

23. Open valves 6, 3, 2 and 5.

24. Check and read 10-inch meter for reverse reading and start recorder.

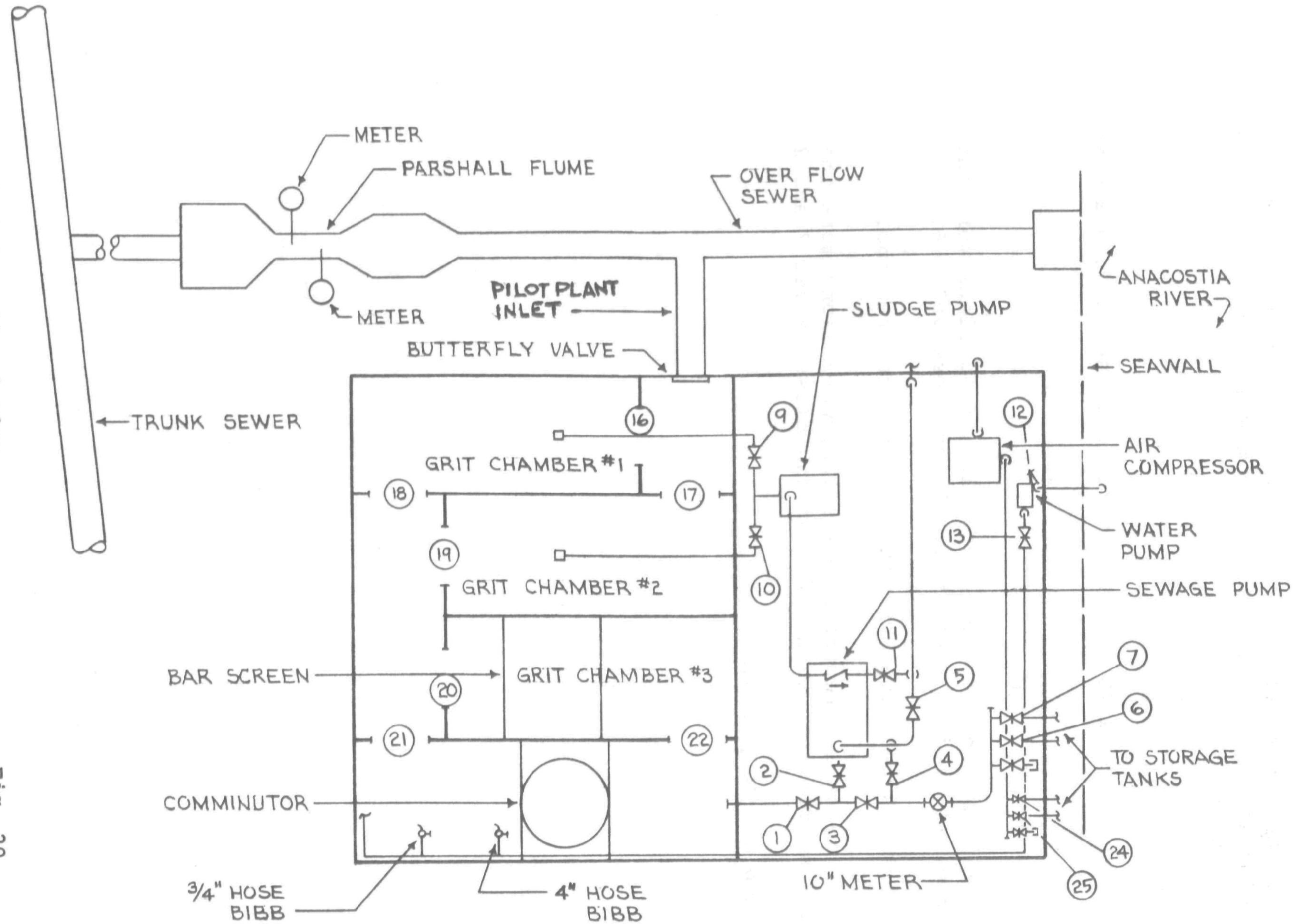
25. Start sewage pump.

26. When meter and recorder reads 100,000-gallons more than original reading, open valve 7 and close valve 6.

27. Check and read 10-inch meter for reverse reading.

28. When meter and recorder reads 100,000-gallons more than second reading, turn off pump and close valves 7 and 2.

29. Stop air compressor.



STORM WATER OVERFLOW DEMONSTRATION FACILITIES

NOT TO SCALE

Fig. 20

Plant Operational Plan

Metering System

Using two meters in Parshall flume at different depths enables measurement of the total flow from the combined overflow sewer. The meters transmit the water level information to the recorders in the pump room. With the use of a diagram furnished by the meter manufacturer, it was possible to compute the submerged flow.

A separate metering system in the pump room measured the amount of flow into and out of the underwater storage tanks.

Sampler Operating Instructions

1. Be certain all bottles and tubing are clean.

2. Set rubber stoppers securely in the 24-bottles.

3. Place vacuum head over sampling head and start the vacuum pump. Hold vacuum head in place until all bottles are vacuum sealed. Vacuum is assured in each bottle if each of the rubber tubes is collapsed (flattened).

4. Seal off each bottle by setting the switches on the switch plate.

5. Release the vacuum head and check to see if all bottles are evacuated and holding vacuum. Check the rubber tube to see if each remains collapsed.

6. Set tripping arm to desired position.

7. Drop sampling head into manhole. Make sure head is submerged in liquid.

8. Start sampler.

9. Record the bottle number of the first sample.

10. After sampling period, remove bottles and cover with screw on caps. Place bottles in refrigerator.

11. For cleaning: Back flush each line with water from the rubber stopper end and allow to dry for future use. Make sure all solids are flushed or blown with compressed air from sampling head and plastic pipes.

TABLE VI

HISTORY OF OPERATION

Date	Total Rainfall (inches)	Flow to Tanks (gallons)	Pumped from Tanks (gallons)	Storage in Tanks Inboard (gallons)	Storage in Tanks Outboard (gallons)	Number of Samples Taken
1968						
12/10	Began operation of Parshall flume (samples only).					
12/16	Trace					4
12/23	0.6					7
12/30	0.1					2
1969						
1/15	Underwater storage system in place.					
1/18	0.3	78,000			78,000	15
1/20	0.2	5,000		5,000		5
1/21	0.5					7
1/22	0.1					4
1/24	Trace					2
1/25	0.0		68,000		10,000	
2/3	Trace					7
2/8	0.3					7
2/9	Samples taken from inside both tanks through manhole.					
2/15	0.0	50,000 RW			60,000	
2/16	0.0		60,000			
2/17	0.0	30,000 RW		35,000		
2/18	0.0		128,000	-93,000		
	Realized a tear existed in the inboard tank. This is discussed in detail in this section under operational problems. Inboard tank out of service through April 29, 1969.					
2/22	Trace					6
2/23	0.2					4
2/26	Trace					2
3/6	Trace					4
3/7	0.1					5
3/9	Trace					7
3/15	0.0	54,000 RW			54,000	
3/16	0.0		54,000			
3/22	0.0	53,000 RW			53,000	
3/23	0.0		53,000			
3/24	0.2					2
3/25	0.2					2
3/29	Trace					2
3/30	Trace	20,000 RW			20,000	

RW indicates river water pumped into tanks.

TABLE VI - Continued

HISTORY OF OPERATION

Date	Total Rainfall (inches)	Flow to Tanks (gallons)	Pumped from Tanks (gallons)	Storage in Tanks		Number of Samples Taken
				Inboard (gallons)	Outboard (gallons)	
4/5	0.3	6,000			26,000	1
4/7	0.0	22,000 RW			48,000	
4/8	0.0	24,000 RW			72,000	
4/9	0.0		52,000		20,000	
4/12	0.0	100,000 RW			120,000	
4/13	0.0		120,000			
4/16	0.2	18,000			18,000	6
4/18	Trace					4
4/19	0.3					2
4/20	0.0		12,000		6,000	3
4/22	0.3	15,000			21,000	7
4/26	0.0	86,000 RW			107,000	
4/27	0.0		88,000		19,000	1
	Developed a problem in the main sewage pump. This is discussed in this section under operational problems. Pump out of service through May 21, 1969.					
4/28	0.2	5,000			24,000	1
4/29	0.1	5,000			29,000	
5/9	0.6	37,000		37,000		6
5/10	0.0	63,000 RW		100,000		
5/12	0.0		48,000	52,000		
5/19	0.4	1,000		53,000		11
5/20	0.2					6
5/28	0.0		4,000	49,000		
6/1	0.1	6,000		55,000		4
6/2	1.2	9,000	29,000	64,000		10
6/3	0.1	11,000	46,000	29,000		4
6/8	0.5	7,000		36,000		4
6/15	0.6	35,000		71,000		7
6/18	0.4	2,000		73,000		10
6/28	0.0	78,000 RW		113,000	38,000	
6/29	0.0		151,000			

TABLE VI - Continued

HISTORY OF OPERATION

Date	Total Rainfall (inches)	Flow to Tanks (gallons)	Pumped from Tanks (gallons)	Storage in Tanks Inboard (gallons)	Storage in Tanks Outboard (gallons)	Number of Samples Taken
7/5	0.2	15,000		15,000		
7/6	0.2	11,000		26,000		
7/9	0.0		26,000			
7/12	0.2	7,000			7,000	3
7/15	0.0		7,000			
7/19	0.4	17,000			17,000	4
7/20	1.8	54,000	16,000		55,000	7
7/22	2.0	141,000		101,000	95,000	10
7/23	0.0	196,000				
7/26	0.0	16,000 RW		16,000		
7/27	0.8	68,000		84,000		6
7/28	2.3	96,000			96,000	26
7/29	0.0		84,000			
7/30	0.0		96,000			
8/1	0.3	49,000			49,000	6
8/2	2.4	30,000			89,000	7
8/3	0.6	38,000		38,000		8
8/4	0.0		127,000			
8/5	0.3	53,000			53,000	4
8/9	1.0	50,000			103,000	6
8/10	0.8	63,000		63,000		9
8/11	0.0		60,000		43,000	
8/14	0.0		43,000			
8/19	0.2	12,000		75,000		4
8/20	0.2	19,000		94,000		7
8/21	1.8	115,000			115,000	3
8/22	0.0		94,000			
8/23	0.0		115,000			
9/2	0.2	17,000		17,000		2
9/3	0.2	12,000		29,000		2
9/4	1.7	75,000		104,000		6
9/8	2.6	114,000			114,000	8
9/9	0.0		218,000			
9/17	0.4	24,000		24,000		4
9/30			24,000			

Operational Problems

In order to expedite construction and operation of the demonstration facility, it was decided to use a standard type underwater pillow tank as manufactured by Goodyear Tire and Rubber Company. Since the tanks had no flotation devices or hold-down facilities, it was necessary to fabricate steel cradles to support the tanks as well as to provide the means for connection to the patented system of anchors in the river bed. The tank cradles were not designed for raising and lowering as was necessary during demonstration and during tank repair. Not only is this an expensive proposition, but may result in damage because of improper crane handling or to uneven stresses exerted on the frame.

The use of the cradle for support of the underwater storage tanks necessitated installation of nylon straps laced diagonally across the top of tank and fastened to the cradle. Underwater storage tanks were installed and ready for operation in January, 1969. It was found after a few weeks of operation that only one-half of the design straps were installed and that those which were in place had pulled apart. Interlacing between straps, at each cross-over point, where the straps could have been sewn would have prevented the condition from occurring. When the straps pulled apart because of the strap problem and because of air vent problems within the tank, several air pockets developed and top of tank sections appeared above the water surface. This was remedied at the time by the addition of air vents at the air pockets. Originally, this problem would not have existed if a cargo type net were used for hold-down in lieu of the web straps.

In December, 1969 a 2-inch, 9,000-pound test nylon net, double-stitched at each crossover point was installed over each tank. The nets were each provided along the periphery with 74 3/4-inch diameter triangular rings (31 per side and 6 per end). The rings were connected underwater to the tank cradle by 5/8-inch shackles to the existing "D" rings welded to the frame. This installation proved successful and indicated that in future installations there would be no need for strapping. Cost of installation of nets was $7,500.00.

After several weeks of operation it was found that the top of the underwater storage tanks were not level and that high points appeared because of the strapping problem, because of the compressed air used in the tank for aeration and agitation, and because of inadequate air venting on the top of the tanks. As a result of this situation it was necessary to add six (6) air vent fittings along the top of each tank and to extend air vent piping to Urethane floats on top of the water. This was not effective since floats broke away because of floating debris. As a result of this problem, it was apparent that underwater air vents would be required to be installed within the tank to relieve air directly into the surrounding waterway.

Six (6) equally spaced underwater diaphragm purge valves were installed in December, 1969 on the top of tanks. These valves are designed to operate against the pressure head of the water above the tanks. The valves open on internal tank pressure to expel air and gases within the tank, and further, prevents water flow from the river into the tanks.

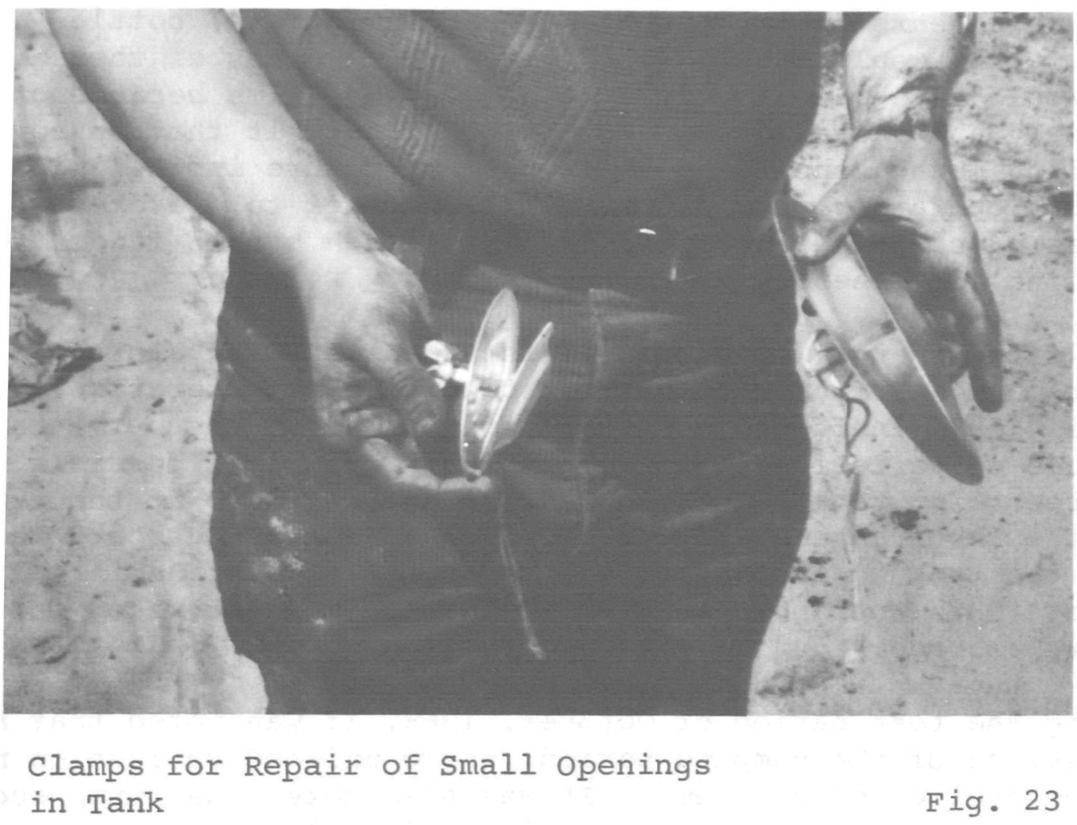
It was found during a pump out process on February 18, 1969, one month after tanks were in operation, that an extensive tear existed in the inboard storage tank. The size of the tear was too large for an underwater repair job. Divers attempted to install clamps, which would have been feasible if the tear had been noted earlier. A second attempt to use large lever type scissor clamps proved unsuccessful because of the fact that the rubber at the tear had folded back under the tank and it was impossible to exert enough pressure to pull the flap back into position. It was then decided to raise the tank and to make all repairs above water. When the tank was raised, it was noted that the tear occurred near the manhole location. It was presumed that the material at this point had been weakened because of a previous force exerted by a crane cable connected to the top of the manhole. In addition, it was found that air binding could have occurred at the tear location and this could have been causing additional pressure to the weak spot, thereby causing the rupture. The installation of the new air vents and the net over the tanks, as well as protection of the manhole from cable or crane attachment will prevent a future occurrence of this situation.

The repair to the tear was performed above the water between April 20, and April 29, 1969 with sheet rubber material applied with a base material and cured with heat, much like a vulcanizing process. Because of the size of the tear, and because of the limited number of electric iron assemblies available, it was necessary to set up nine times to patch the entire area. Each patch was cured in approximately three-hours. The cost of raising the tank from shore with two ninety-ton cranes and repairing the tear was $9,600.00. Figures 21 and 22 show the repairs being made to the tank tear. Figure 23 shows the clamps used in repairing small openings in tanks.

Fig. 21

Fig. 22

Repair of Large Tear in Tank



Clamps for Repair of Small Openings in Tank

Fig. 23

A standard type non-clogging self-priming sewage pump was used to pump from the underwater storage tanks to the interceptor sewer. It was noted during a pump out on April 27, 1969 that the pump was vibrating and clattering. Investigation found that the inlet check valve diaphragm had broken away and had clogged the pump impeller. This subsequently was repaired by the manufacturer on May 21, 1969 when it was found to be a defect aggravated by some floating solid that caught up in the check valve. This is not a normal type problem and should be a rarity for the type pump used.

The automatic sampler at the Parshall flume was found to be inadequate for the requirements of the testing laboratories. The equipment was purchased prior to construction and before any arrangements were made for testing. Actually, during the initial operation testing was performed in part by the F.W.P.C.A. Analytical Services Laboratory at District of Columbia Pilot Plant and in part by The C. W. England Laboratories. The sampling quantities required were four-times greater than that originally contemplated. As a result, samples were taken partly with the automatic sampler, and primarily by hand in accordance with schedule set forth for the project. The alternate would have been to change gears on the sampler to sample four (4) bottles at a time. This would have reduced the effectiveness of the sampler and would have resulted in extensive outage time because of delivery and installation of gears. It is felt that an advanced sampling method should be considered in future installations, such as pump operating on a programmed time clock to operate with a series of solenoid valves installed in each branch feeder to a sampling container.

An altitude gauge was installed in the underwater storage tank to record height of tank at various conditions of operation. This gauge proved ineffective because of unevenness of top of tank and because of variations in pressure in the tank. A possible method for recording tank height would be a sonar device at the sea wall for the inboard tank and one underwater for the outboard tank to scan the tank configuration and transmit to a screen in the pump house. This would be extremely costly and is not recommended for the value received.

During the test period of October, 1968, it was noted that at the conclusion of the pumping period, there existed cavitation noises at the suction of the pump. It was also noted that pump suction pressure had increased. This indicated a blockage in the suction line. The divers verified the conclusion that the tank had collapsed over the suction inlet of the tank. As a result, the tanks were raised on October 16, 1968 and 6-inch by 6-inch welded woven wire mesh formed in an 18-inch diameter cylinder were installed the length of each tank. This proved successful in prevention of tank collapsing during pumping.

Site Restoration and Renovation

Inasmuch as the 33-inch overflow sewer was to be abandoned by the District of Columbia Department of Sanitary Engineering because of the installation of a new highway in the vicinity of the overflow manhole, it was agreed that the pilot plant was of no further service as a holding facility for combined sewer overflow. It was decided that the facility, with minor modifications, could be used by the National Capitol Park Service as a holding facility and pumping station for various marinas and industrial facilities existing or contemplated in the vicinity of the pilot plant. As a result, various measuring equipment, instrumentation, Parshall flume, meter house and sundry piping external to the pilot plant were removed and the site was regraded for use by the National Capitol Park Service.

SECTION 10

SAMPLE COLLECTION AND ANALYSIS

During and after every rainstorm, samples were taken of combined sewer overflow flowing through the Parshall flume. The time of sampling had been developed as being spaced in accordance with the stage hydrograph. In general, one sample was taken every fifteen minutes during the first hour of rain; one sample every thirty minutes during the second hour; and one sample every hour thereafter until there was no overflow in the line.

Analyses carried out in the laboratory were: Coliform bacteria, Escherica Coli bacteria, BOD (biologican oxygen demand), TOC analysis, suspended solids, settleable solids, volatile suspended solids, pH value and petroleum products.

From December, 1968 through June, 1969 samples of the effluent were analyzed by the following laboratories:

1. Coliform and petroleum products tests by The C. W. England Laboratories, Inc., Washington, D.C.

2. BOD, TOC, pH, settleable solids, suspended and volatile suspended solids tests were made by the Analytical Services Laboratory at the Federal Water Pollution Control Administration Pilot Plant, Washington, D.C.

From July, 1969 through September, 1969 all testing was performed by The C. W. England Laboratories, Inc. In this latter period no TOC tests were performed since facilities were not available.

The results of the analyses have been plotted against time. Sample test results and graphs are shown in Table VII, and Figures 32 through 56 inclusive.

A complete listing of dates, rainfall and number of samples taken during the period of operation from December, 1968 to October, 1969 may be found in Section 9, under sub-heading "History of Operation," Table VI.

Tests were made by "Standard Methods for Examination of Water and Waste Water," 12th edition, 1965, except as otherwise noted herein.

1. Total Coliform - Standard Test A.
2. Coliform - E Coli - Standard Test A with the E Coli identified by selecting a representative number of colonies into E. C. Medium.
3. Biochemical oxygen demand - Section III with oxygen determined - Section III method A (Azide Modification).
4. pH - Electrometrically adjusted to 25° C.
5. Suspended Solids - Section III, part C, using a glass mat.
6. Settleable Solids - (Residue) - Section III, method F 1.1 by volume using Imhoff cone.
7. Volatile Settleable Solids - (Residue) - Section III, method D.
8. Petroleum Products - Section III, method B (grease).
9. Total oxygen consumed - chromatographic method.

During the initial stage of the project, samples of the overflow were taken at random, and not according to the required schedule. However, the latter samples conformed to the schedule and showed some basic trends. Two characteristic trends were observed. First, bacteria count, etc. increased abruptly during the initial onset of rain. They then decreased during the duration of the rain, and tapered off. This verifies the assumption that during a rainstorm, the bacteria concentration build up in the sewer is washed down into the overflow main during the initial phase. If this flow is diverted into a temporary storage tank during the initial stages of rainfall, river contamination would be greatly reduced.

Second, it was observed that bacterial count always increased during peak sanitary sewage usage hours. This occurs in the morning at approximately 7:30 A.M. and in the evening at approximately 7:00 P.M. During these peak hours, bacteria concentration is always at its highest, particularly during light rains. If storage during these peak periods of sanitary sewer overflow were accomplished, a major reduction of river pollution would result.

Biological oxygen demand was found to be greater than average for domestic waste at the outset of the overflow period. Samples of 400 to 600 milligrams per liter of combined sewer overflow were recorded. Following the initial phase of approximately 60-minutes of the overflow, BOD concentration dropped to well below domestic waste average. The general range during this period was 60 to 100 milligrams per liter. Sampling of flow from the underwater storage tank indicated an initial BOD of approximately 100 milligrams per liter falling to approximately 20 milligrams per liter during the latter stages of the pumping cycle. Higher BOD concentration in the initial draw from the tanks is attributable to the fact that pump suction was taken from bottom of the tank. High concentration of solids were also noted in the initial flow from the tank.

Settleable solids were found to fall in the range of 10 to 60 milliliters per liter at the outset of overflow and dropped rapidly to the range of 0.1 to 1.1 milliliters per liter, well below the range of domestic sewage flow because of dilution. Suspended solids followed the same pattern, 200 to 1,000 milligrams per liter at the start and 100 to 150 milligrams per liter following the initial flush.

TABLE VII

SUMMARY OF LABORATORY ANALYSES

Date and Time	Total Coli 1,000 per 100 ml	E. Coli 1,000 per 100 ml	BOD mg/l.	pH	Settleable Solids ml/l.	Suspended Solids mg/l.	VSS mg/l.	Petro Products mg/l.	TOC mg/l.
12/22 1968									
1730	14,000	6,200	106	7.20	4.0	248	NR	52.9	126
1850	12,000	5,800	80	7.15	1.8	148	NR	41.1	76
1915	3,500	1,200	53	7.35	0.4	80	NR	27.3	46
1940	5,100	1,400	43	7.26	0.8	62	NR	25.6	33
2015	5,700	1,600	37	7.25	0.2	80	NR	24.1	29
2045	1,600	200	49	7.28	0.1	151	NR	84.8	138
2115	1,400	500	39	7.33	0.1	40	NR	24.2	35
1/18 1969									
830	850	660	657	6.56	40.0	1,196	970	161.0	800
1020	240	210	433	7.12	60.0	1,068	850	-	460
1045	140	40	122	7.26	1.0	100	60	-	140
1115	40	30	119	7.45	1.0	100	64	-	125
1215	390	210	186	7.48	1.5	240	150	-	180
1300	80	9	119	7.12	5.0	176	108	35.0	126
1335	70	6	138	7.47	0.2	152	108	-	134
1700	60	11	72	6.65	1.0	108	60	-	126
1845	20	14	168	7.42	0.6	132	103	-	180
1900	81	32	202	7.43	0.3	120	100	-	220
2015	43	10	64	7.50	0.8	108	44	-	75
2100	11	2	63	7.50	0.1	68	48	-	70
2200	110	14	72	7.50	1.2	78	78	-	90
2230	64	32	99	7.50	2.5	112	112	78.0	95
2300	480	21	202	7.12	7.0	480	300	-	270

TABLE VII - Continued

SUMMARY OF LABORATORY ANALYSES

Date and Time	Total Coli 1,000 per 100 ml	E. Coli 1,000 per 100 ml	BOD mg/l.	pH	Settleable Solids ml/l.	Suspended Solids mg/l.	VSS mg/l.	Petro Products mg/l.	TOC mg/l.
2/1 1969									
0630	52	12	150	7.10	3.5	418	NR	-	135
0700	56	16	45	6.98	0.7	150	NR	20	42
0730	30	9.3	80	7.13	4.5	198	NR	-	66
2/2 1969									
2000	3	3	9	6.70	0.1	40	NR	-	15
2100	3	3	7	6.88	0	46	NR	11.0	12.6
2200	3	3	7	7.21	0	46	NR	-	11.4
2300	3	3	5	6.61	0	30	NR	-	12.6
2/8 1969									
2015	520	470	540	6.58	70.0	1,500	1,040	-	740
2035	130	120	360	6.90	35.0	844	530	-	300
2240	54	33	210	6.96	18.0	524	305	-	210
2300	21	11	105	7.02	3.5	656	249	.33	165
2315	19	11	26	7.02	0.8	162	NR	-	39
2345	11	8.2	27	7.01	0.4	124	72	-	35
2400	4.2	2.2	33	6.88	0.6	128	60	-	39
2/9 1969									
0200	3.8	2.6	23	6.93	NR	112	NR	-	27
0300	6.1	2.9	19	6.97	NR	86	NR	51	26
1000	5.7	2.2	4	7.47	0	90	NR	-	9
1400	4.3	2.3	24	7.45	0.8	108	NR	-	26

TABLE VII - Continued

SUMMARY OF LABORATORY ANALYSES

Date and Time	Total Coli 1,000 per 100 ml	E. Coli 1,000 per 100 ml	BOD mg/l.	pH	Settleable Solids ml/l.	Suspended Solids mg/l.	VSS mg/l.	Petro Products mg/l.	TOC mg/l.
2/9 1969									
Tank*	58.0	56.0	640	6.62	38.0	1,716	1,100	65	820
	76.0	62.0	118	6.83	0.5	192	115	438	102
4/18 1969									
2130	370	29	58	6.79	0.8	800	112	24	80
4/19 1969									
2040	440	110	136	7.00	1.0	164	108	61	106
4/20 1969									
0127**	1,400	570	111	7.00	5.0	560	190	-	128
0134**	610	180	16	7.06	0.3	112	28	-	22
0147**	230	28	18	7.05	0.1	92	10	21	18
4/22 1969									
0300	1,600	780	400	6.55	32	768	71	68.0	408
0320	340	110	47	6.73	2.5	248	53	-	63
0345	54	16	28	6.73	0.8	548	13	14.0	44
0415	56	27	23	6.65	0.2	196	31	-	29
1735	1,200	210	320	7.32	4.0	1,116	42	289.0	480
1745	450	56	350	7.23	10.0	2,580	32	-	520
1750	230	70	57	7.27	0.5	440	14	16.0	60

*Laboratory Analysis of Samples Taken from Inboard Underwater Storage Tank.

**Laboratory Analysis of Samples Taken from Outboard Underwater Storage Tank.

SUMMARY OF LABORATORY ANALYSES

Date and Time	Total Coli 1,000 per 100 ml	E. Coli 1,000 per 100 ml	BOD mg/l.	pH	Settleable Solids ml/l.	Suspended Solids mg/l.	VSS mg/l.	Petro Products mg/l.	TOC mg/l.
5/9 1969									
0915	840	290	350	6.64	25	1,308	49	-	470
0930	520	250	125	6.90	3.0	484	34	-	156
0945	170	80	85	7.23	5.5	1,468	22	51.0	190
1005	25	4	43	6.96	1.5	212	22	-	50
1150	65	43	82	7.32	4.5	312	33	-	72
1220	35	24	83	7.40	0.7	200	20	24.0	46
5/19 1969									
0815	59	12	90	7.43	1.2	152	45	-	80
1030	680	270	120	6.60	6.0	276	52	24.0	252
1045	540	180	115	6.43	3.0	228	46	-	114
1100	63	50	115	6.62	8.5	372	50	-	158
1130	89	62	58	6.73	0.75	148	38	29.0	71
1205	88	71	130	6.68	0.75	156	50	-	94
2130	870	440	200	6.98	2.0	236	66	-	172
2145	810	380	140	7.05	2.0	312	58	42.0	166
2200	550	220	100	7.16	7.0	252	57	-	112
2215	1,300	400	100	7.19	7.0	264	47	-	102
2230	1,100	320	100	7.12	NR	136	50	21.0	90
2430	470	80	34	6.90	NR	72	50	-	33
0145	870	600	380	6.58	30.0	2,528	41	-	456
0200	310	90	60	6.96	7.0	1,724	23	-	168
0215	540	70	40	6.94	2.5	476	24	-	69
0225	280	130	20	6.85	1.3	288	15	15.0	39
0330	460	110	18	7.00	0.8	164	17	-	40

TABLE VII - Continued

SUMMARY OF LABORATORY ANALYSES

Date and Time	Total Coli 1,000 per 100 ml	E. Coli 1,000 per 100 ml	BOD mg/l.	pH	Settleable Solids ml/l.	Suspended Solids mg/l.	VSS mg/l.	Petro Products mg/l.	TOC mg/l.
6/1 1969									
1930	-	-	1,040	6.15	13.0	1,560	640	-	840
1945	-	-	1,200	6.14	10.0	1,360	530	-	880
2000	-	-	800	6.46	1.2	300	100	-	630
2015	-	-	580	6.72	0.2	120	16	-	410
6/2 1969									
2015	3,000	600	330	5.50	3.0	328	161	-	290
2030	2,200	850	360	6.08	2.5	344	150	-	280
2045	-	-	390	6.90	3.0	316	136	-	270
2100	1,600	700	370	7.08	2.5	260	112	-	300
2115	1,800	400	380	7.45	2.0	224	108	-	250
2130	3,000	800	350	7.17	2.0	212	108	-	240
2220	2,000	850	170	6.53	5.0	716	243	-	130
2235	-	-	300	6.25	17.0	2,820	740	-	240
2250	1,500	250	200	6.35	8.0	2,540	762	-	230
2305	500	90	170	6.53	5.0	1,930	365	-	260
6/3 1969									
0050	100	35	12	7.11	0.3	92	20	-	18
0100	40	20	12	6.92	0.4	88	20	-	21
0115	300	80	12	7.20	0.1	44	12	-	16
0130	100	35	13	7.20	0.1	40	16	-	15

TABLE VII - Continued

SUMMARY OF LABORATORY ANALYSES

Date and Time	Total Coli 1,000 per 100 ml	E. Coli 1,000 per 100 ml	BOD mg/l.	pH	Settleable Solids ml/l.	Suspended Solids mg/l.	VSS mg/l.	Petro Products mg/l.	TOC mg/l.
6/15 1969									
1345	1,600	750	500	6.37	11.0	1,456	35	-	500
1400	740	320	480	6.33	16.0	1,332	36	62.0	575
1415	590	330	530	6.35	17.0	1,428	40	-	375
1430	190	74	290	7.16	1.2	264	33	-	204
1445	46	13	280	7.11	1.5	220	29	-	196
1500	73	21	80	6.95	1.8	508	20	11.0	140
1515	72	24	70	6.95	1.5	500	25	-	70
6/18 1969									
1305	3,800	940	340	6.88	20	544	76	130	408
1320	2,700	540	410	6.43	40	1,844	60	-	700
1350	240	100	50	6.75	1.5	228	39	12	51
1415	230	90	54	6.83	0.5	136	33	-	44
2215	910	180	110	6.64	6.5	296	51	-	118
2230	840	300	95	6.70	5.5	316	38	34	92
2245	570	180	100	6.70	4.5	536	32	-	108
2300	980	320	110	6.67	6.5	732	38	-	138
2315	480	110	90	6.67	3.0	812	27	23	120
2330	360	150	70	6.72	3.0	728	25	-	112

TABLE VII - Continued

SUMMARY OF LABORATORY ANALYSES

Date and Time	Total Coli 1,000 per 100 ml	E. Coli 1,000 per 100 ml	BOD mg/l.	pH	Settleable Solids ml/l.	Suspended Solids mg/l.	VSS mg/l.	Petro Products mg/l.	TOC mg/l.
7/1 1969									
1530	530	180	388	6.28	16.0	1,020	584	54	NR
1545	1,200	460	335	6.30	12.0	888	510	-	NR
1600	1,300	410	429	6.42	15.0	940	536	-	NR
1615	2,100	620	175	6.20	6.0	216	48	15	NR
7/20 1969									
1345	2,400	920	316	6.82	12.0	1,112	552	-	NR
1400	2,100	840	139	7.21	2.0	316	210	-	NR
1415	2,900	410	183	6.82	3.5	366	212	13	NR
1730	1,300	160	222	6.58	7.5	602	180	-	NR
1745	1,100	380	66	6.62	2.5	598	168	-	NR
1800	1,900	240	35	6.42	2.0	582	160	-	NR
1815	2,900	410	188	6.68	7.5	898	352	3	NR
7/22 1969									
1730	3,600	700	54	6.52	6.0	1,492	260	-	NR
1745	1,200	260	65	6.58	0.8	382	84	-	NR
1800	1,600	280	25	6.50	0.8	462	276	8	NR
1830	1,400	460	45	6.28	0.7	232	62	-	NR
1900	640	170	19	6.08	2.0	448	140	-	NR
1930	200	30	75	6.15	0.5	262	194	-	NR
2000	140	10	65	6.22	0.5	322	86	-	NR
2030	680	220	60	6.20	1.0	158	40	3	NR
2100	360	20	65	6.22	0.5	266	70	-	NR
2200	450	60	80	6.41	0.7	136	88	-	NR

TABLE VII - Continued

SUMMARY OF LABORATORY ANALYSES

Date and Time	Total Coli 1,000 per 100 ml	E. Coli 1,000 per 100 ml	BOD mg/l.	pH	Settleable Solids ml/l.	Suspended Solids mg/l.	VSS mg/l.	Petro Products mg/l.	TOC mg/l.
8/9 1969									
2115	970	170	180	6.50	1.3	712	218	-	NR
2130	400	160	50	6.57	0.4	218	98	8	NR
2200	800	120	60	6.49	1.1	266	126	-	NR
2300	920	70	80	6.41	1.2	364	182	-	NR
2330	920	50	40	6.38	0.7	586	182	5	NR
2400	840	120	80	6.42	1.1	386	160	-	NR
8/10 1969									
0100	6,100	810	120	6.70	0.1	90	50	-	NR
0130	5,300	760	140	6.71	1.1	102	64	-	NR
0230	6,200	810	90	6.68	0.1	44	24	7	NR
0330	6,500	510	60	6.70	0.1	48	28	-	NR
0430	6,500	790	90	6.71	1.0	112	80	-	NR
0500	5,300	540	110	6.75	2.5	74	54	-	NR
0530	6,600	840	70	6.70	0.1	60	32	4	NR
0600	6,500	710	70	6.72	0.1	52	24	-	NR
0700	7,100	460	60	6.72	0.1	54	30	-	NR
8/19 1969									
1900	620	360	640	5.98	29.0	11,062	3,774	-	NR
1915	930	410	530	6.02	24.0	8,212	1,230	69	NR
1930	360	160	110	6.12	1.4	1,030	172	-	NR
1945	300	160	120	6.09	0.9	632	184	-	NR

TABLE VII - Continued

SUMMARY OF LABORATORY ANALYSES

Date and Time	Total Coli 1,000 per 100 ml	E. Coli 1,000 per 100 ml	BOD mg/l.	pH	Settleable Solids ml/l.	Suspended Solids mg/l.	VSS mg/l.	Petro Products mg/l.	TOC mg/l.
8/20 1969									
0230	110	30	130	6.32	0.2	148	112	21	NR
0315	90	20	90	6.52	15.0	2,372	234	-	NR
0345	80	30	100	6.68	9.5	802	164	-	NR
0445	60	40	100	6.78	1.1	418	118	25	NR
0545	70	40	90	6.85	23.0	1,302	338	-	NR
0645	50	20	80	6.82	3.4	684	134	-	NR
0730	40	20	80	6.82	29.0	2,970	326	54	NR
8/21 1969									
0005	160	60	40	6.42	0.1	18	0	-	NR
0030	190	80	70	6.22	0.1	28	16	19	NR
0130	130	60	60	6.31	0.1	36	24	-	NR
9/9 1969									
0100	380	90	85	6.32	3.0	706	168	-	NR
0115	260	40	25	6.30	1.5	548	176	17	NR
0130	340	90	45	6.28	0.9	288	136	-	NR
0145	70	1	35	6.22	0.8	176	74	-	NR
0200	80	0.8	25	6.32	0.8	158	86	-	NR
0230	20	0.3	95	6.19	0.3	122	58	19	NR

SECTION 11

HYDROLOGY

Collection of rainfall data was taken by a recording rain gauge placed on the roof of the building housing the grit chambers and comminutor. The gauge was manufactured by the Belfort Instrument Company and was of the continuous recording type. Twelve hour strip charts were changed daily. These strip charts were used directly in obtaining rainfall intensity and duration for the construction of the hyetographs found in Figures 32 to 56.

It must be noted here that only in the last four months of the demonstration period was rainfall sufficient to run meaningful sampling and testing. Table VIII summarizes rainfall activity during the operation of the project. Original termination date for the project was June 30, 1969. This was extended to September 30, 1969 to take advantage of the summer rains. The rainfall summary chart indicates the decision was wise.

A liquid level recording station was located near Barney Circle beside a manhole to measure the height of water in the storm sewer. During dry weather flow, the water level was normally found to be below 8-inches. In this case, the water flowed into the existing 72-inch interceptor sewer and thence to the treatment plant. However, during peak usage hours or during a heavy rainstorm, the water level was found to rise well above 8-inches. In this case, the water overflowed into a 33-inch pipe, through the Parshall flume, and then either to the comminutor room or to the river.

A circular chart with a capacity of seven-days measured the height of the water on a continuous basis, 24-hours a day. The chart was changed at least once a week. From the chart, could be determined the time of day when the water level was above 8-inches.

The record of stage in the overflow manhole not only shows when overflows occur, but could be used to estimate the flow diverted from this manhole to the interceptor sewer.

The recording meter was run by nitrogen gas in a tank. Since there was frequent trouble with the nitrogen tank, such as leakage, etc., readings on the chart were sometimes erratic or not recorded. It could be gathered from the charts that during peak usage hours, the water level in the pipe always was at its peak.

TABLE VIII

RAINFALL SUMMARY

Month	Total Rainfall (inches)*	Record Mean (inches)	Rainfall (inches)**
December, 1968	2.22	2.89	0.8
January, 1969	1.69	2.70	1.40
February, 1969	2.08	2.58	2.85
March, 1969	1.60	3.45	1.60
April, 1969	1.71	2.88	1.8
May, 1969	1.20	3.88	1.9
June, 1969	3.46	3.30	3.6
July, 1969	9.44	3.85	5.95#
August, 1969	6.98	4.84	6.3
September, 1969	5.07	3.15	5.30

* From local climatological data, U. S. Department of Commerce

** Site data

\# Rain gauge out of order part of the month.

To determine overflow condition, a recording device measures the height of the water in the overflow manhole. The results of the data are then plotted against a consecutive time interval. This indicates how intensity of rainfall affects the amount of overflow effluent from the 33-inch pipe. An indication that a rain storm always causes overflow to the river will again justify diversion of the overflow to temporary storage in the underwater tanks. However, other factors may also cause an overflow. It seems that usage at peak hours (0700 to 0800, and 1800 to 1900) even during "dry" days will also affect overflow, and hence increase pollution to the river.

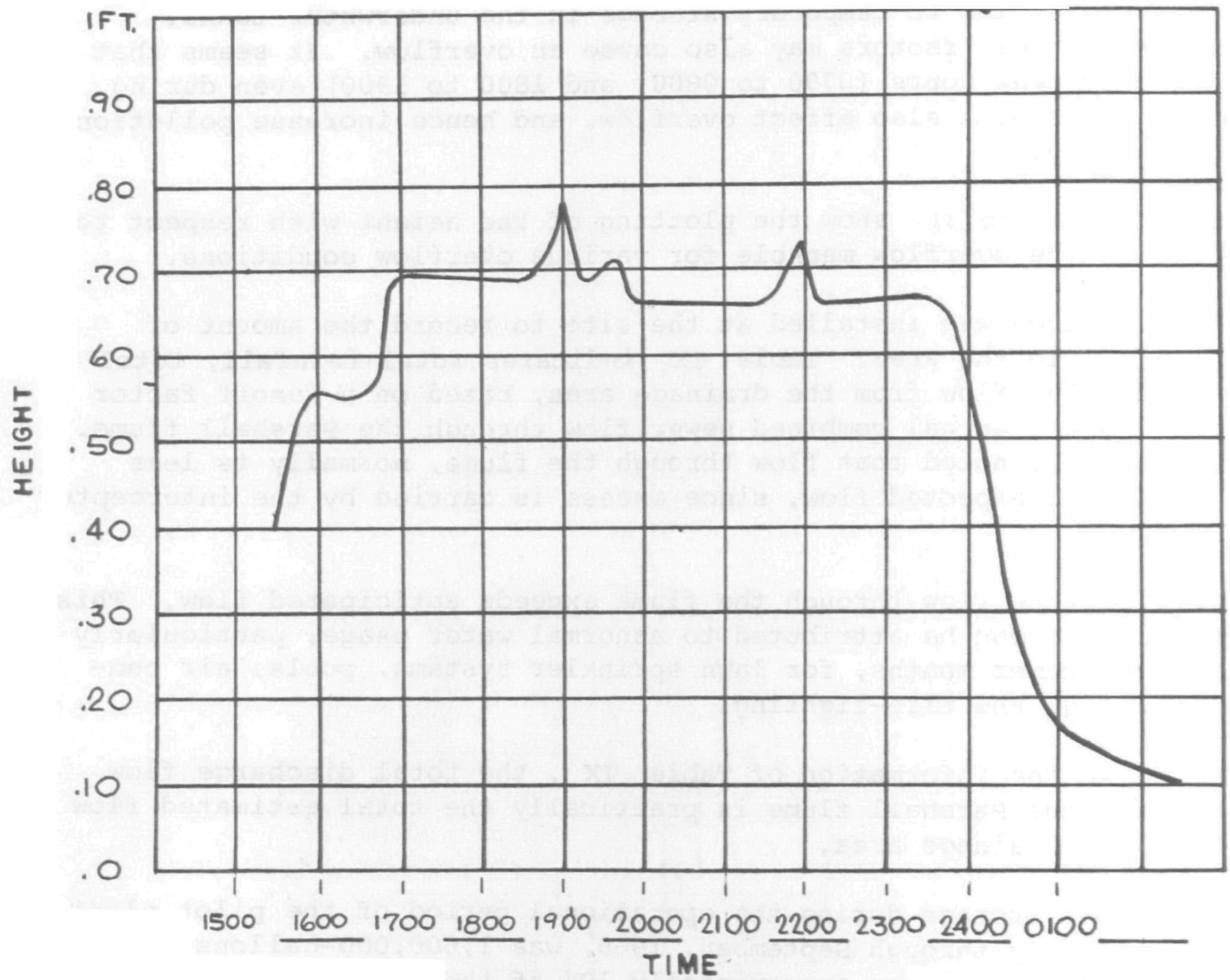
Figures 24 to 31 show the plotting of the height with respect to time in the overflow manhole for various overflow conditions.

A rain gauge was installed at the site to record the amount of rainfall in the area. Table IX indicates total rainfall, total anticipated flow from the drainage area, based on a runoff factor of 0.75 and actual combined sewer flow through the Parshall flume. It is to be noted that flow through the flume, normally is less than total expected flow, since excess is carried by the interceptor sewer.

At times the flow through the flume exceeds anticipated flow. This phenomenon can be attributed to abnormal water usage, particularly during summer months, for lawn sprinkler systems, pools, air conditioning, and fire-fighting.

Based on the information of Table IX , the total discharge flow through the Parshall flume is practically the total estimated flow from the drainage area.

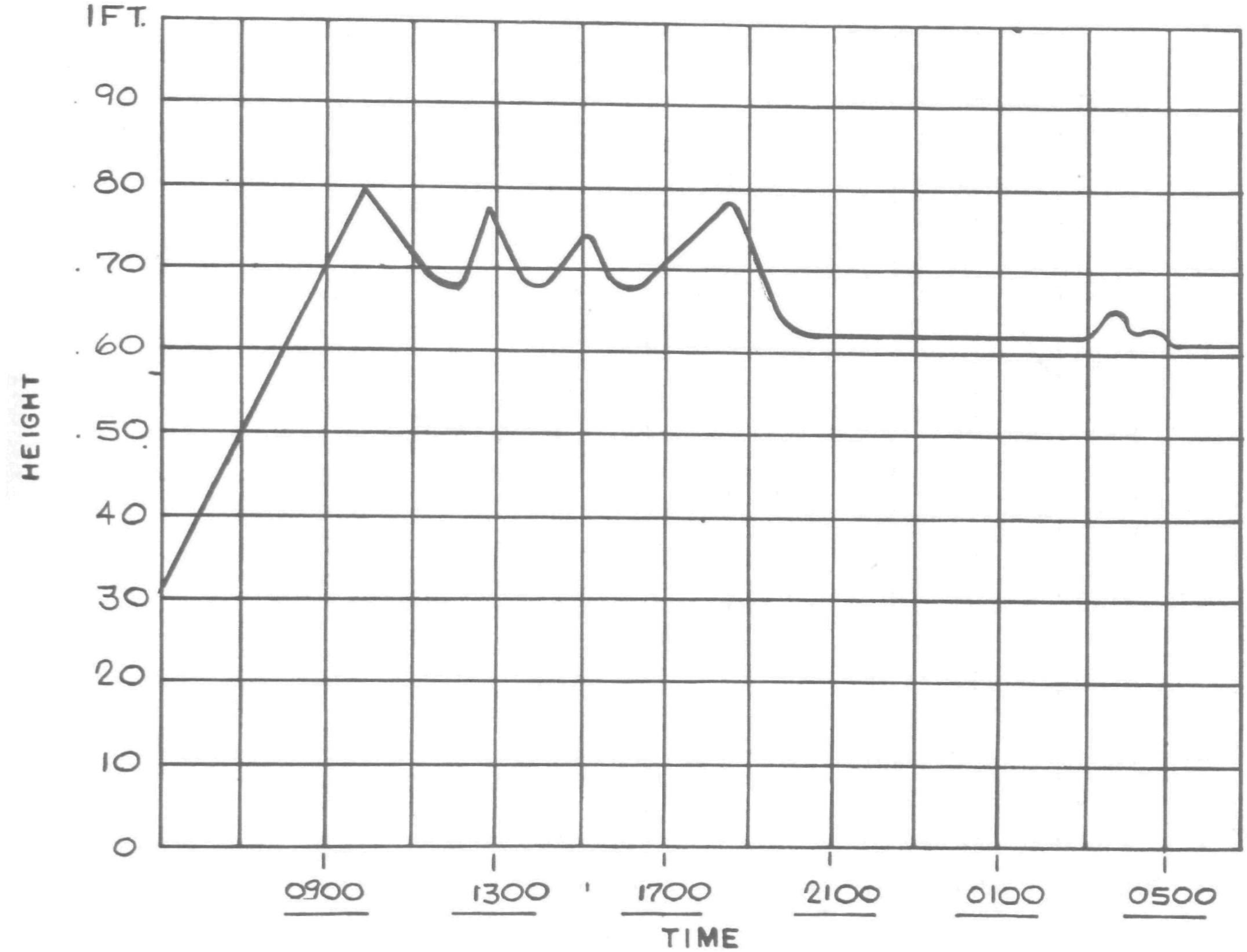
The total storage during the operational period of the pilot plant from January through September, 1969, was 1,600,000-gallons of diverted overflow or approximately 10% of the total flow. If 2,000,000-gallons of storage were provided instead of the 200,000-gallons that was used for this pilot project, no overflow would have reached the Anacostia River. Two million gallons of storage, which can be obtained with the installation of four 500,000-gallon tanks would be of sufficient size to accommodate the maximum condition of September 8, 1969 at which time 2.6-inches of rain fell with a flow of 1,850,000-gallons measured at the Parshall flume.



OVERFLOW MANHOLE

DATE 12-22-68

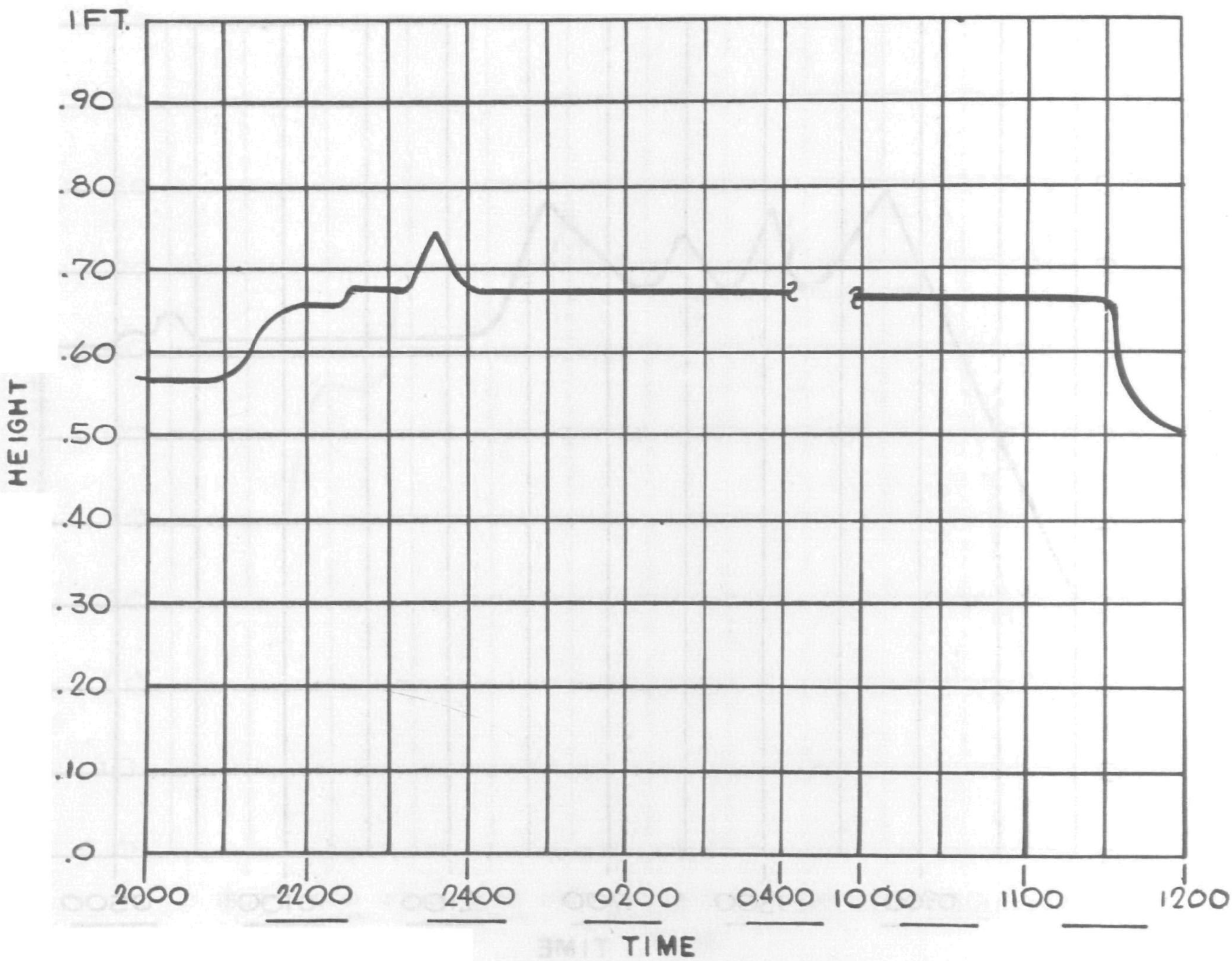
FIG. 24



OVERFLOW MANHOLE

DATE 1-18-69

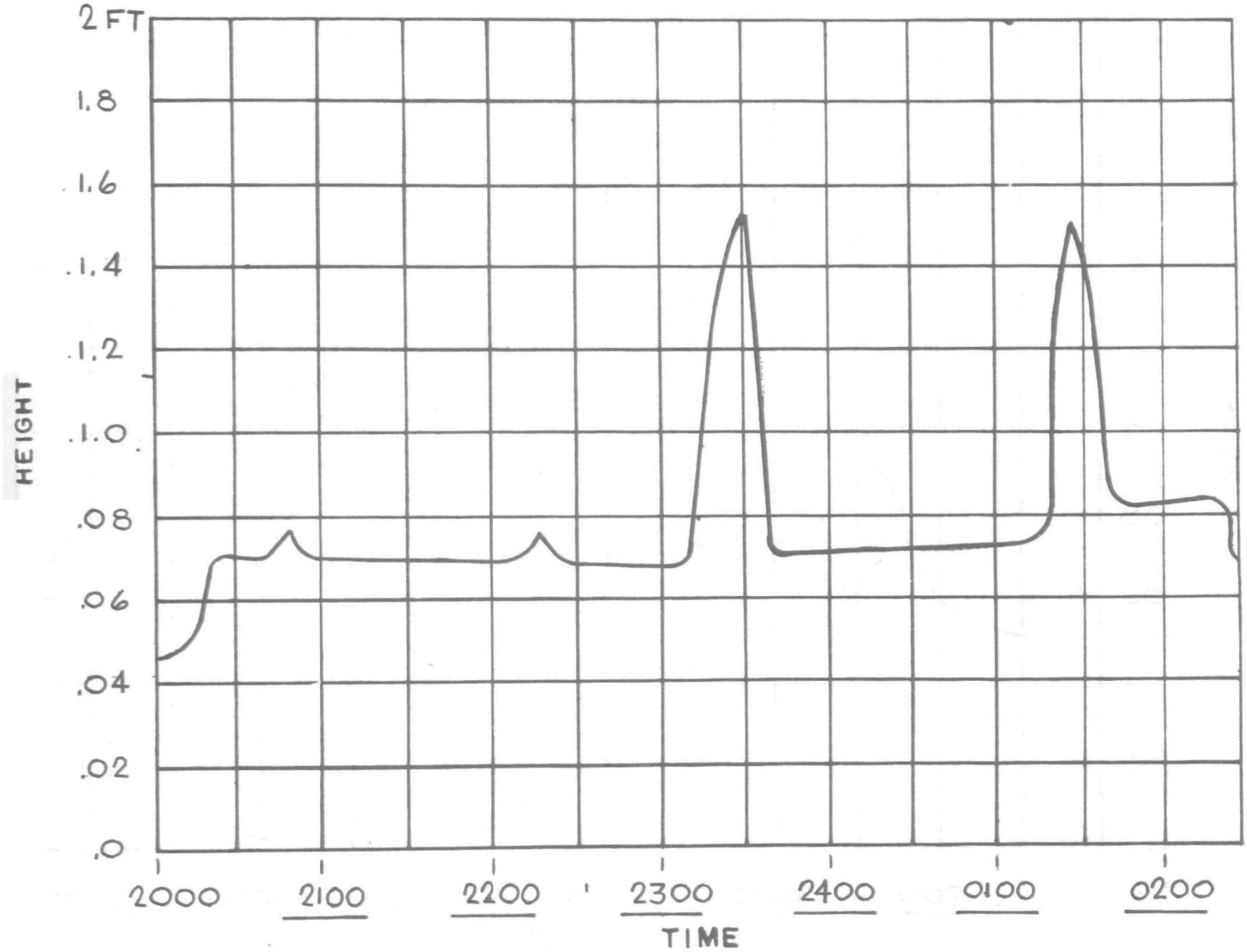
FIG. 25



OVERFLOW MANHOLE

DATE 2·8·69

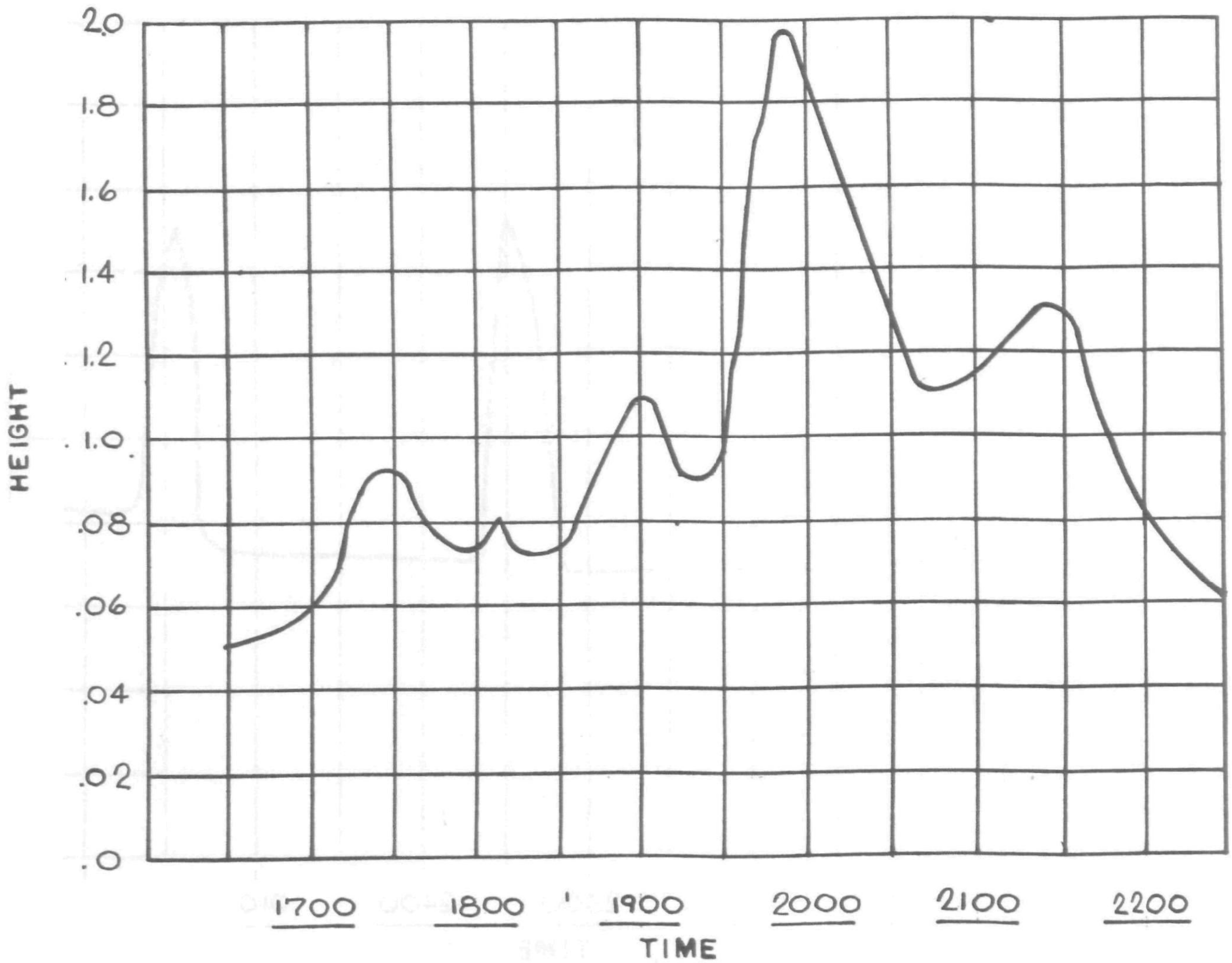
FIG. 26



OVERFLOW MANHOLE

DATE 6-2-69

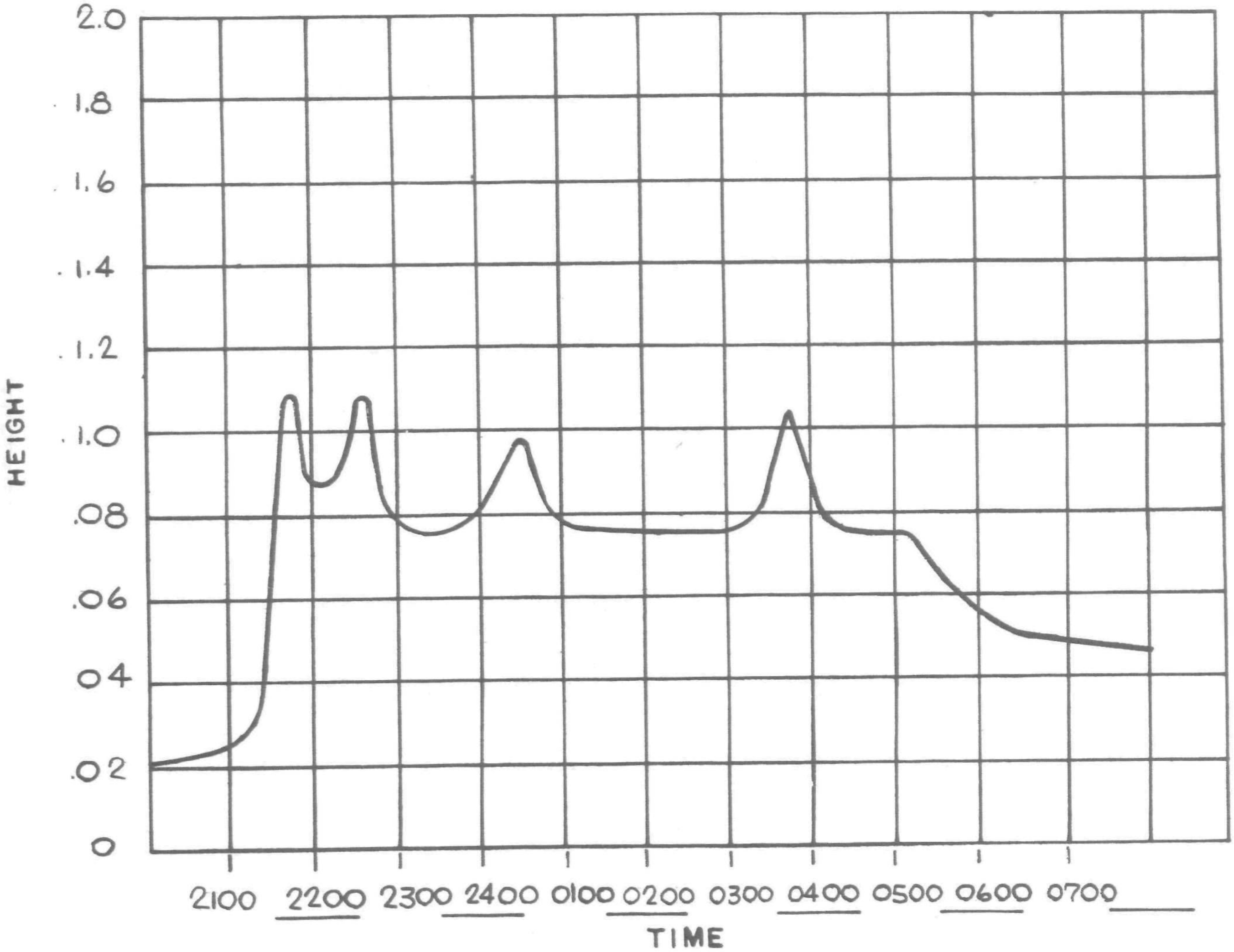
FIG. 27



OVERFLOW MANHOLE

DATE 7-22-69

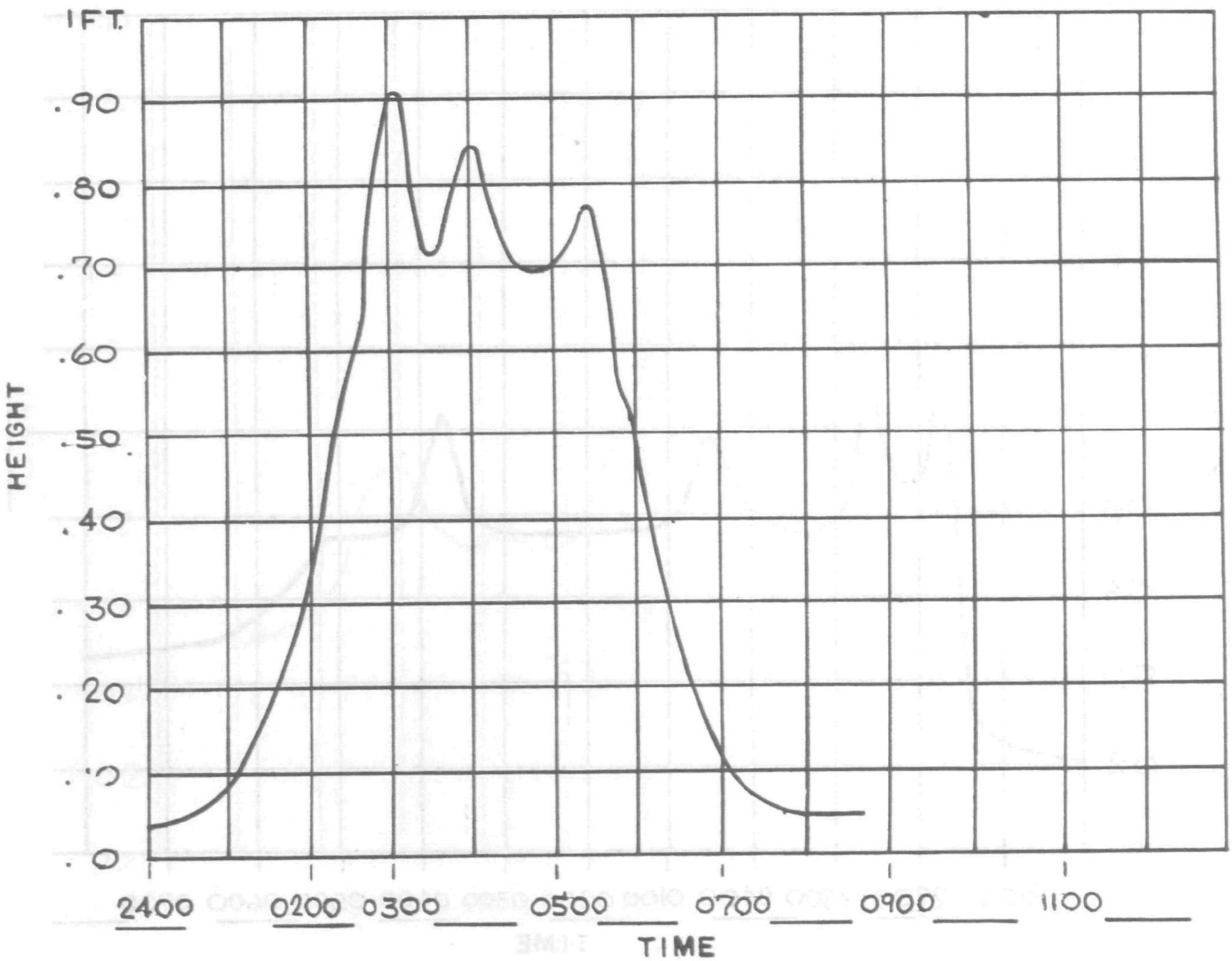
FIG. 28



OVERFLOW MANHOLE

DATE 8-9-69

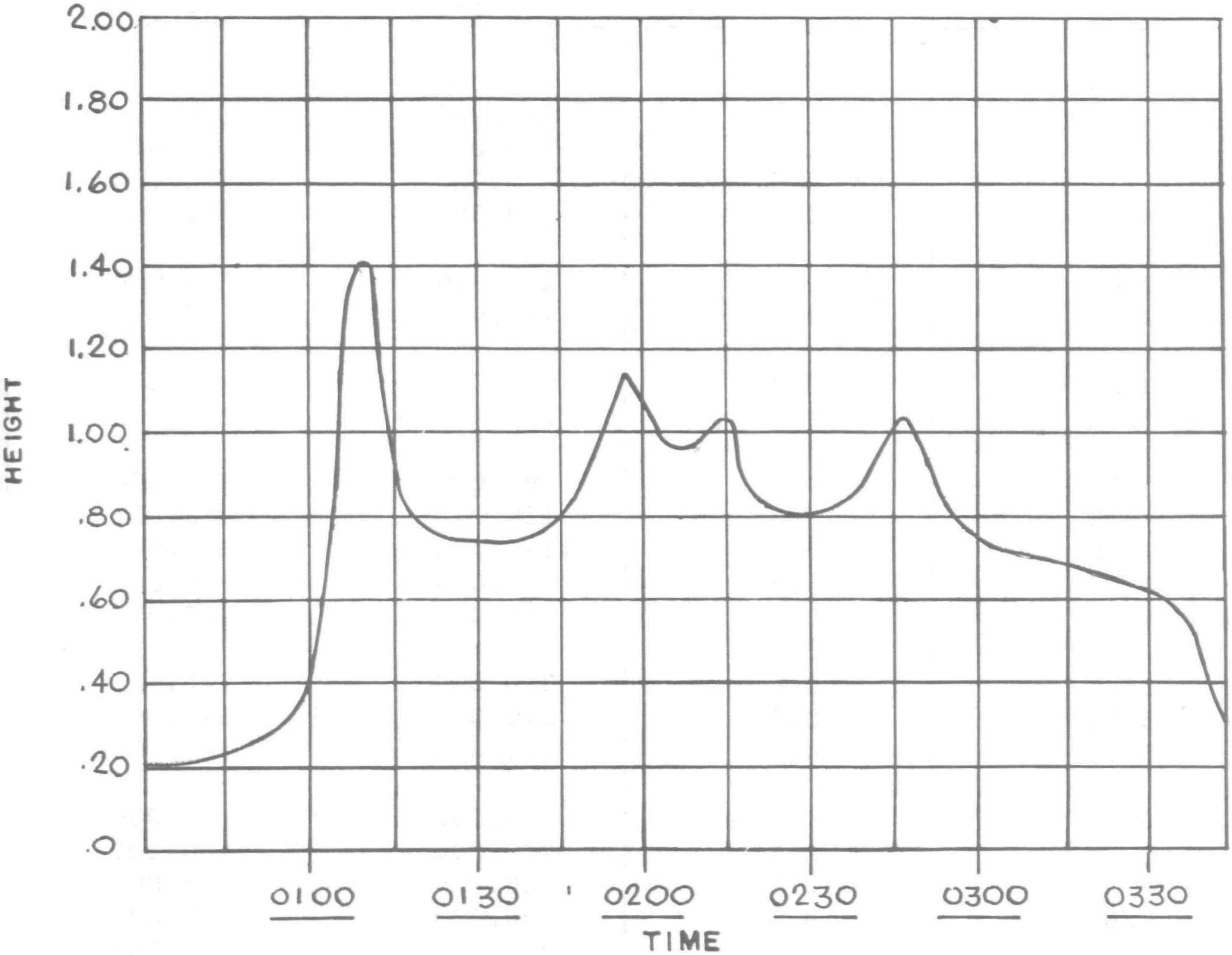
FIG. 29



OVERFLOW MANHOLE

DATE 8-20-69

FIG. 30



OVERFLOW MANHOLE

DATE 9-8-69

FIG. 31

TABLE IX

HISTORY OF RAINFALL AND FLOW THROUGH PARSHALL FLUME

Date	Total Rainfall (inches)	Total Area Drainage (gallons)	Flow Through Flume (gallons)
1969			
1/18	0.3	180,000	280,000
1/20	0.2	120,000	85,000
1/21	0.5	300,000	220,000
1/22	0.1	60,000	25,000
2/8	0.3	180,000	170,000
2/23	0.2	120,000	35,000
3/24	0.2	120,000	102,000
3/25	0.2	120,000	80,000
4/5	0.3	180,000	125,000
4/16	0.2	120,000	45,000
4/22	0.3	180,000	102,000
4/28	0.2	120,000	64,000
4/29	0.1	60,000	35,000
5/9	0.6	360,000	245,000
5/19	0.4	240,000	170,000
5/20	0.2	120,000	90,000
6/1	0.1	60,000	42,000
6/2	1.2	720,000	600,000
6/3	0.1	60,000	38,000
6/8	0.5	300,000	235,000
6/15	0.6	360,000	280,000
6/18	0.4	240,000	320,000

TABLE IX - Continued

HISTORY OF RAINFALL AND FLOW THROUGH PARSHALL FLUME

Date	Total Rainfall (inches	Total Area Drainage (gallons)	Flow Through Flume (gallons)
7/5	0.2	120,000	146,000
7/6	0.2	120,000	160,000
7/12	0.2	120,000	115,000
7/19	0.4	240,000	212,000
7/20	1.8	1,080,000	1,700,000
7/22	2.0	1,200,000	1,350,000
7/27	0.8	480,000	275,000
7/28	2.3	1,380,000	1,400,000
8/1	0.3	180,000	240,000
8/2	2.4	1,440,000	1,200,000
8/3	0.6	360,000	280,000
8/5	0.3	180,000	80,000
8/9	1.0	600,000	380,000
8/10	0.8	480,000	395,000
8/19	0.2	120,000	86,000
8/20	0.2	120,000	135,000
8/21	1.8	1,080,000	1,250,000
9/2	0.2	120,000	125,000
9/3	0.2	120,000	150,000
9/4	1.7	1,020,000	1,650,000
9/8	2.6	1,560,000	1,850,000
9/17	0.4	240,000	230,000
Totals	27.8	16,680,000	16,796,000

SECTION 12

DISCUSSION

The purpose of the project was to divert a part of the combined sewer overflow into the Anacostia River to temporary storage in two 100,000-gallon underwater storage tanks. After a storm subsides and at non-peak usage hours, waste water in the storage tanks is pumped back through a force main to the sewage treatment plant. Application of this kind of process could reduce and eventually eliminate contamination of the river.

A Parshall flume was installed along the path of the overflow main to measure the flow rate of the combined sewer. Analyses as to the contents of the effluent were also performed. These characteristics were analyzed: Coliform Bacteria, Escherica Coli Bacteria, suspended solids, settleable solids, percent of volatile suspended solids, BOD and TOC count, and pH value of the effluent.

One purpose of the analyses was to determine whether or not there was a significant increase in the pollution characteristics immediately after the beginning of a rainfall when the additional rain water helps overflow the pipe. If this increase were significant, then diverting the overflow sewer, at least during the initial part of a storm, to the underwater storage tanks instead of to the river would greatly reduce river contamination.

Graphs were prepared to show the various reading of bacteria count, etc., against the time the samples were taken during and after a rainfall. From the graph, fluctuations as to the various readings can be observed with respect to time. On the graphs were shown the discharge hydrograph and rainfall curve in order to study how the character of the overflow fluctuates according to the intensity and duration of rainfall. The effectiveness of the diversion of the overflow sewer to temporary storage in the underwater tanks will thus be demonstrated.

Measurements were also taken of the height of water in the overflow sewer manhole. During regular days, when the level is below 8-inches, combined sewer flows to an interceptor main and then on to the treatment plant. However, during peak usage hours, or during a severe rainfall, the water may rise above 8-inches, and overflow into the river. However, in this facility flow has been diverted through a Parshall flume, a comminutor room, a pump house, and then to temporary storage into two underwater tanks until the storm subsides.

Figures 32 to 56 show discharge hydrographs, hyetographs and corresponding fluctuations in Coliform bacteria, Escherica Coli bacteria, BOD, TOC, suspended solids, settleable solids, volatile suspended solids and pH value.

It is noted that except for pH values, fluctuations of water sample characteristics are fairly uniform. For purpose of discussion, only the BOD fluctuations are analyzed; the others are similar.

BOD Analysis

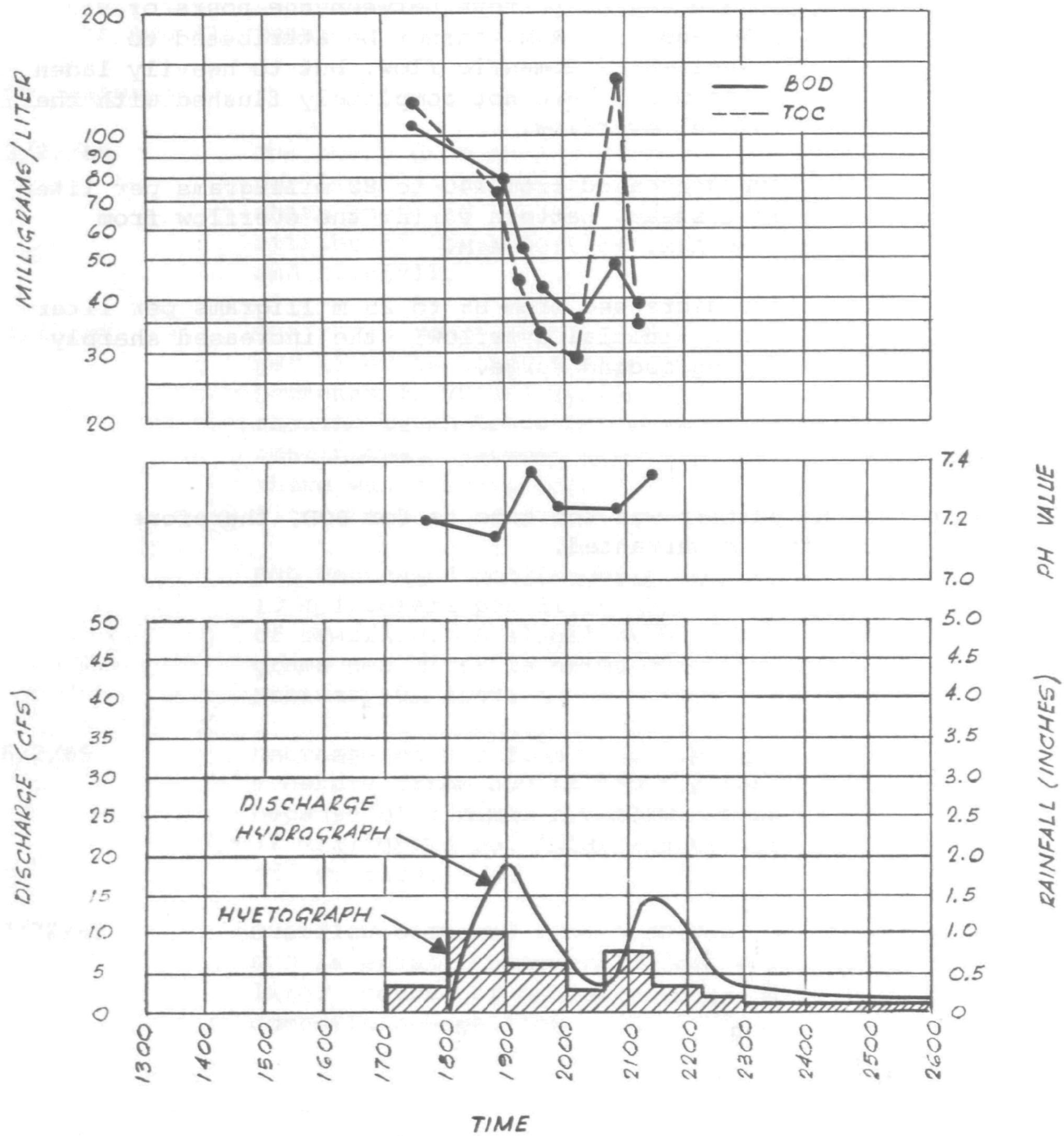
12/22/68	The sharp drop in BOD from 110 to 35 milligrams per liter is noted during initial overflow. The sharp increase that occurred at 9:00 P.M. can be attributed to the coincidence of sanitary flow and rainfall.
1/18/69	The sharp drop in BOD from 600 to 120 milligrams per liter is noted during the initial overflow. Decrease to 70 milligrams per liter over the ensuing seven hours of overflow is erratic, but continuous. Between 6:00 P.M. and 7:00 P.M. there was a sharp increase to 200 milligrams per liter because of domestic sewage flow.
2/8/69	BOD decreased continuously from 600 to less than 10 milligrams per liter over the entire period of overflow. A slight rise in BOD to 20 milligrams per liter is noted during morning hours of February 9, 1969.
6/2/69	Decrease of BOD from 8:30 P.M. to 11:00 P.M. was moderate (from 380 to 180 milligrams per liter) because of minimal overflow. Drop off to 12 milligrams per liter was severe during balance of overflow.
7/22/69	Overflow occurred from 5:30 P.M. to 10:00 P.M. BOD is erratic between 20 and 80 milligrams per liter because of concurrence of overflow with domestic sewage flow.

8/9/69	BOD decreased from 180 to 50 milligrams per liter during initial overflow. Increase to 80 milligrams per liter is noted at start of second overflow peak and further increases to 150 milligrams per liter at third overflow peak. Continuous drop off then occurs over balance of the overflow. This behavior pattern between the hours of 9:30 P.M. and 1:30 A.M. cannot be attributed to coincidental domestic flow, but to heavily laden lines which were not completely flushed with the initial overflow.
8/20/69	BOD decreased from 140 to 80 milligrams per liter in a steady pattern during the overflow from 2:30 A.M. to 7:00 A.M.
9/8/69	BOD decreased from 85 to 25 milligrams per liter during initial overflow; the increased sharply during second surge.

TOC Analysis

It was noted the pattern was the same as for BOD, therefore, no further discussion is warranted.

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 12-22-68

Fig. 32

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

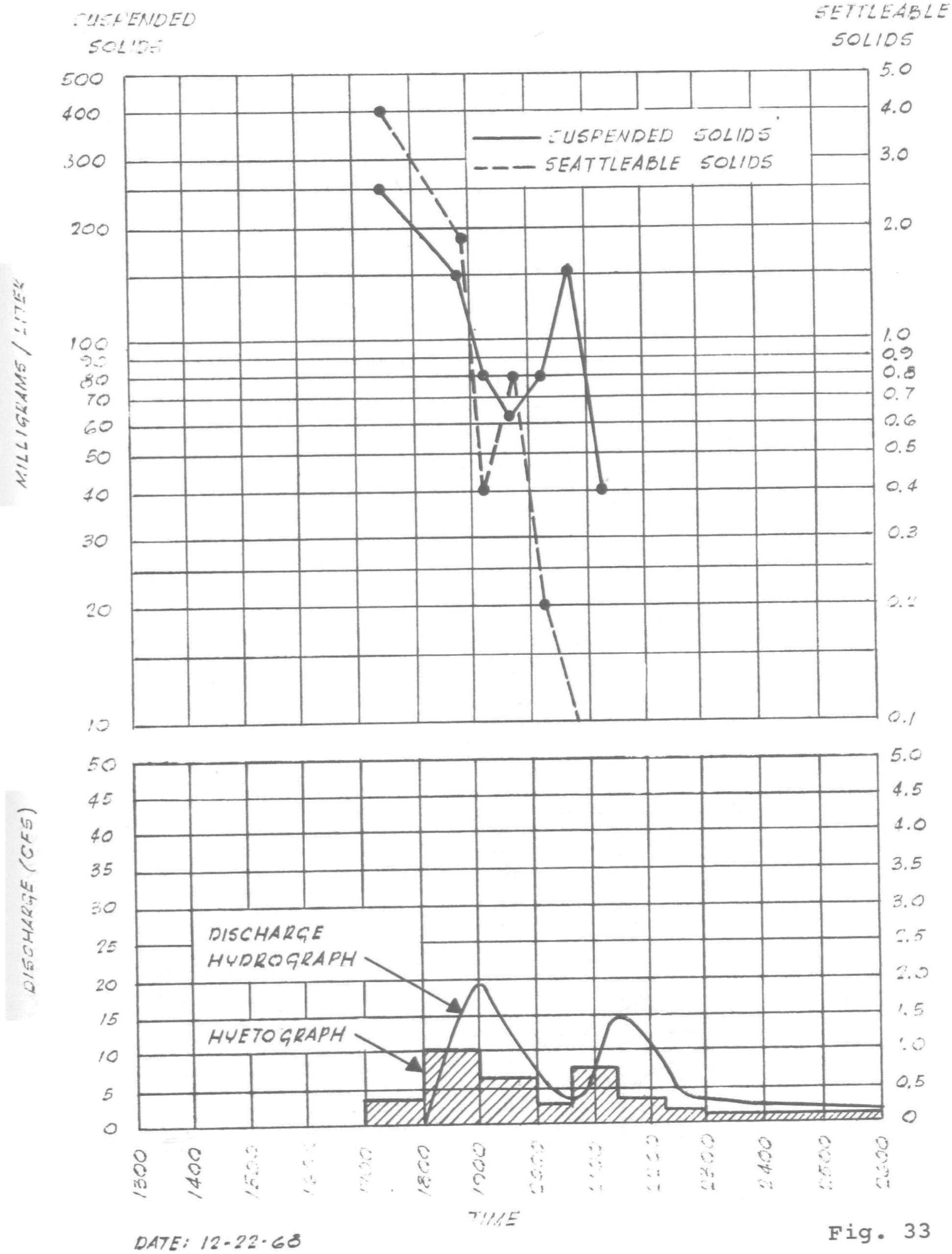
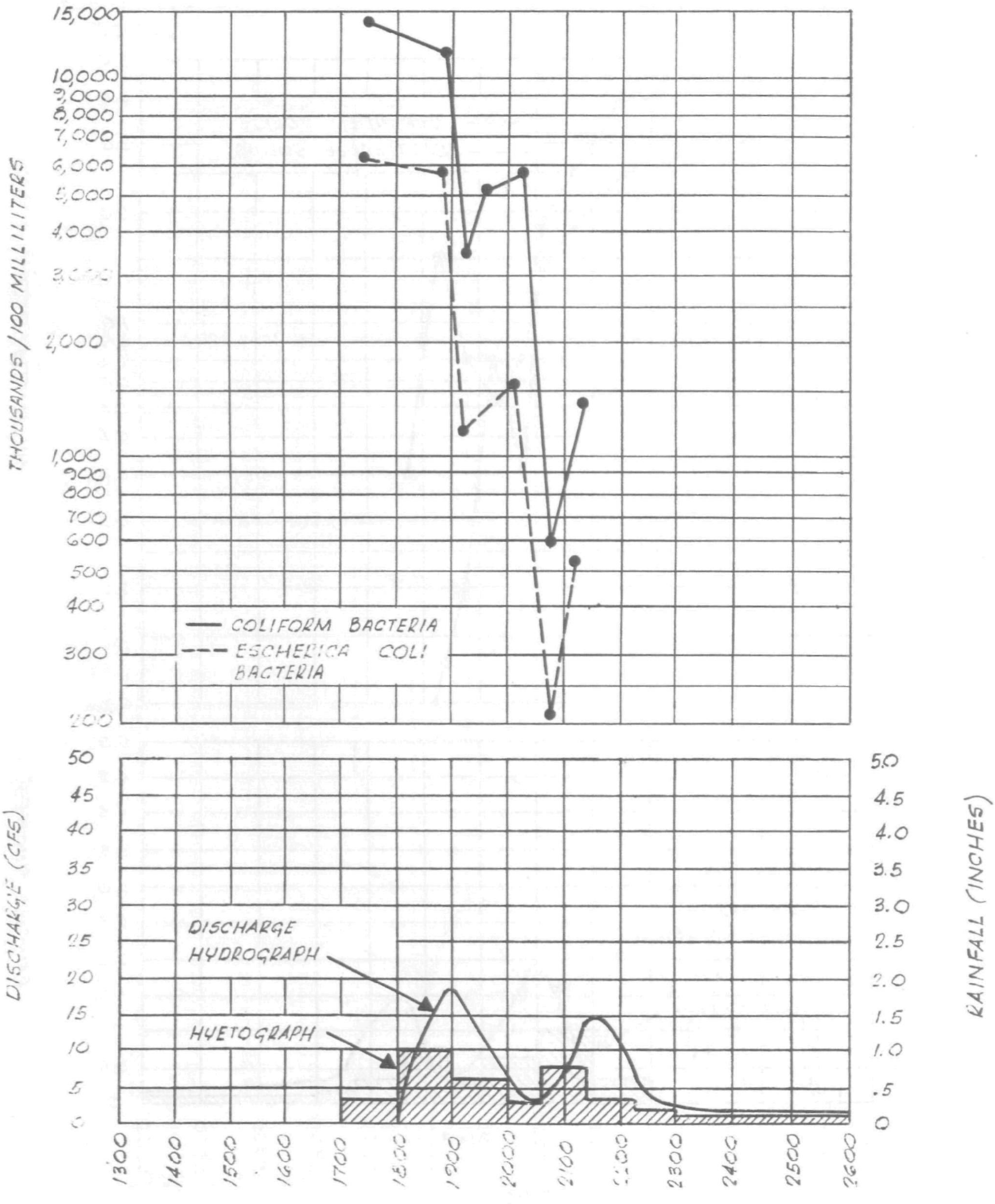


Fig. 33

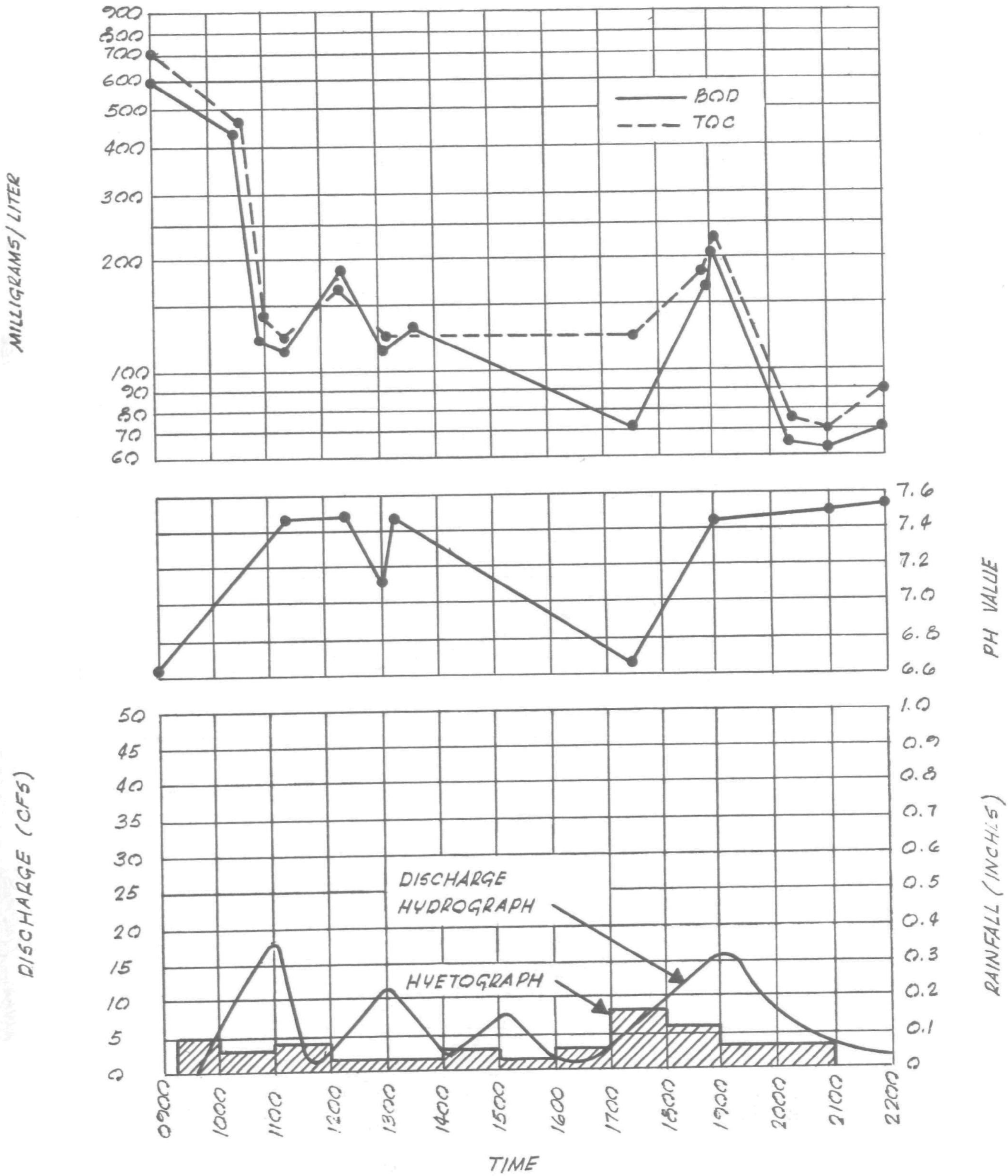
DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 12-22-68

Fig. 34

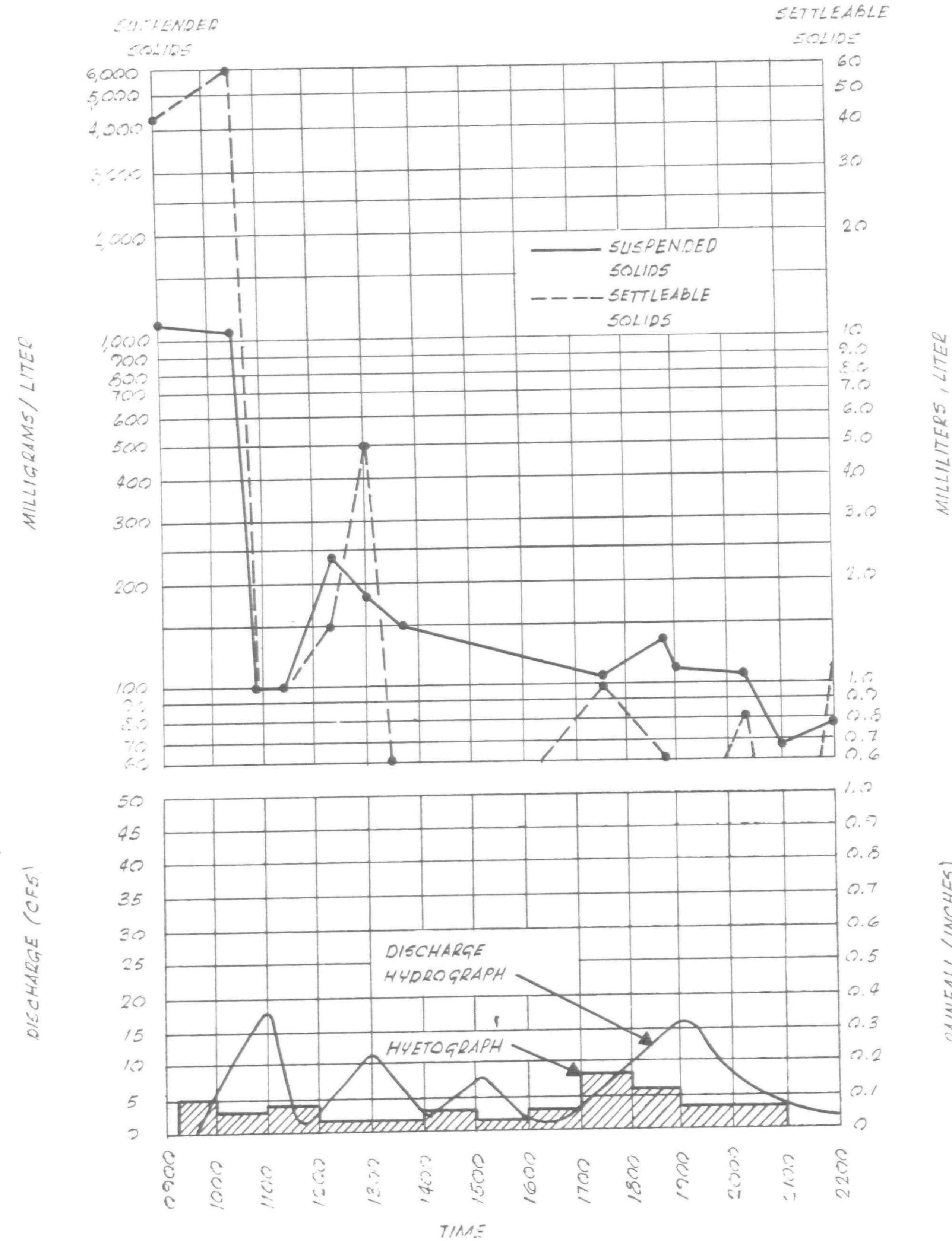
DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 1-18-69

Fig. 35

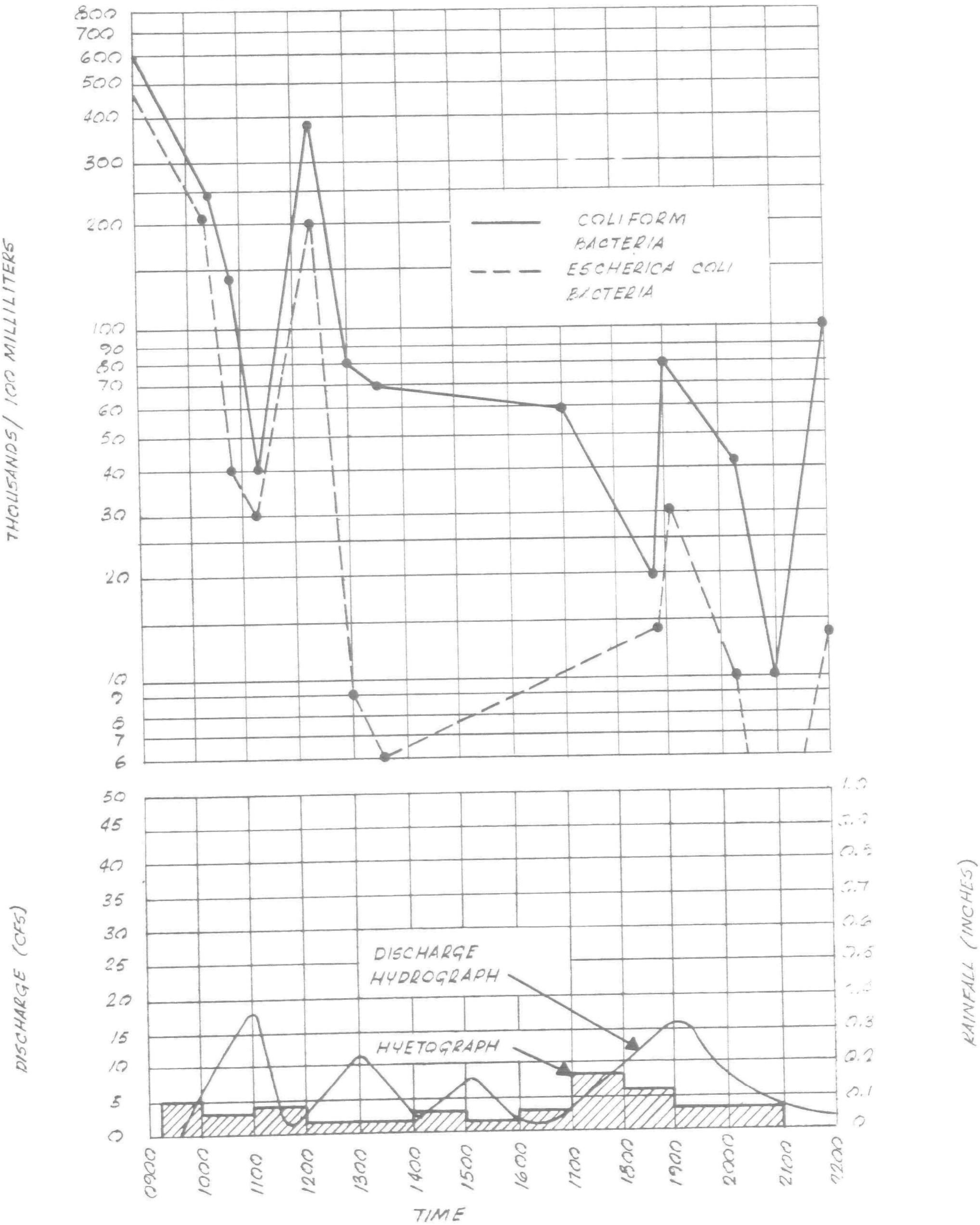
DISCHARGE HYDROGRAPH, HYETOGRAPH AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 1-18-69

Fig. 36

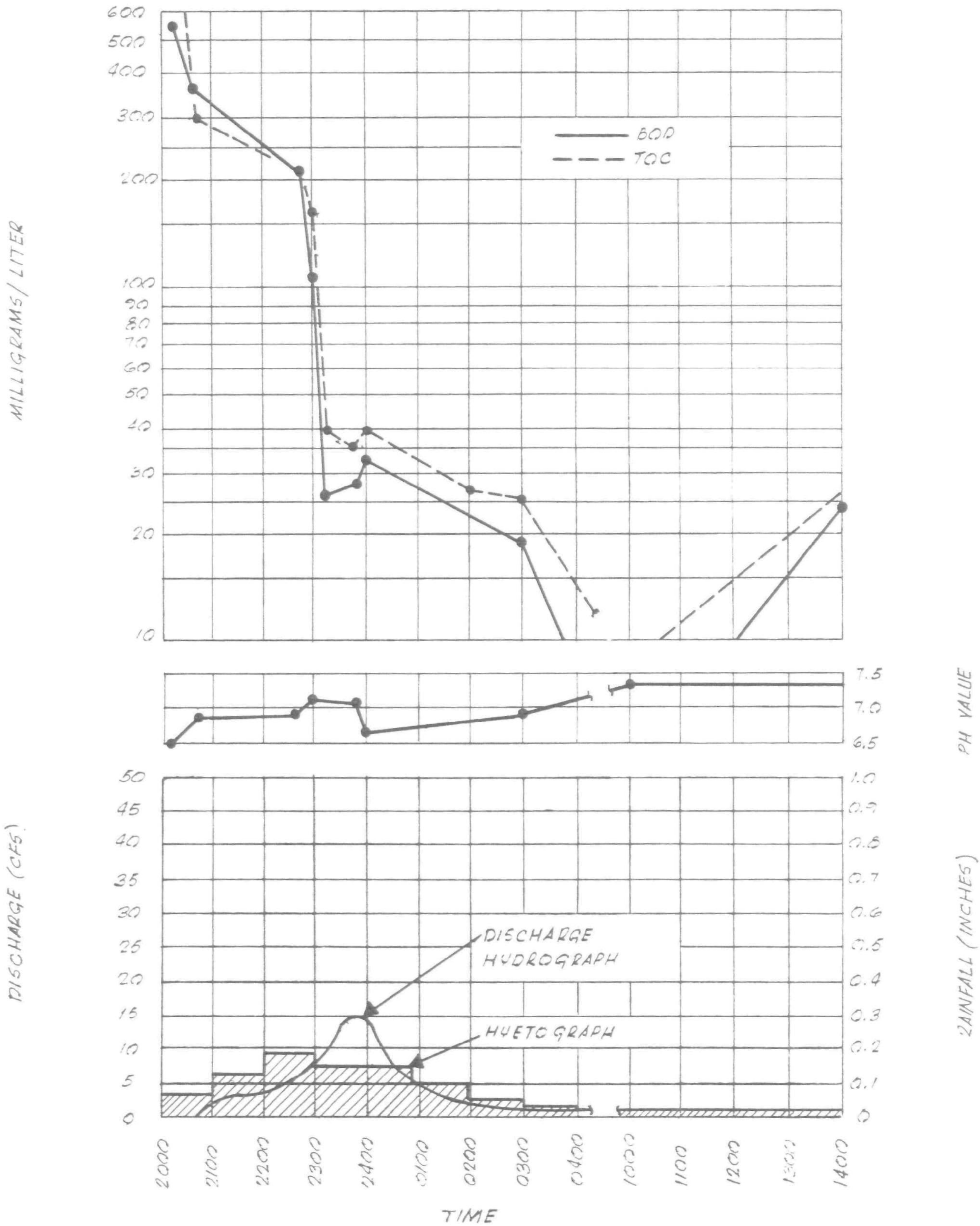
DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 1-18-69

Fig. 37

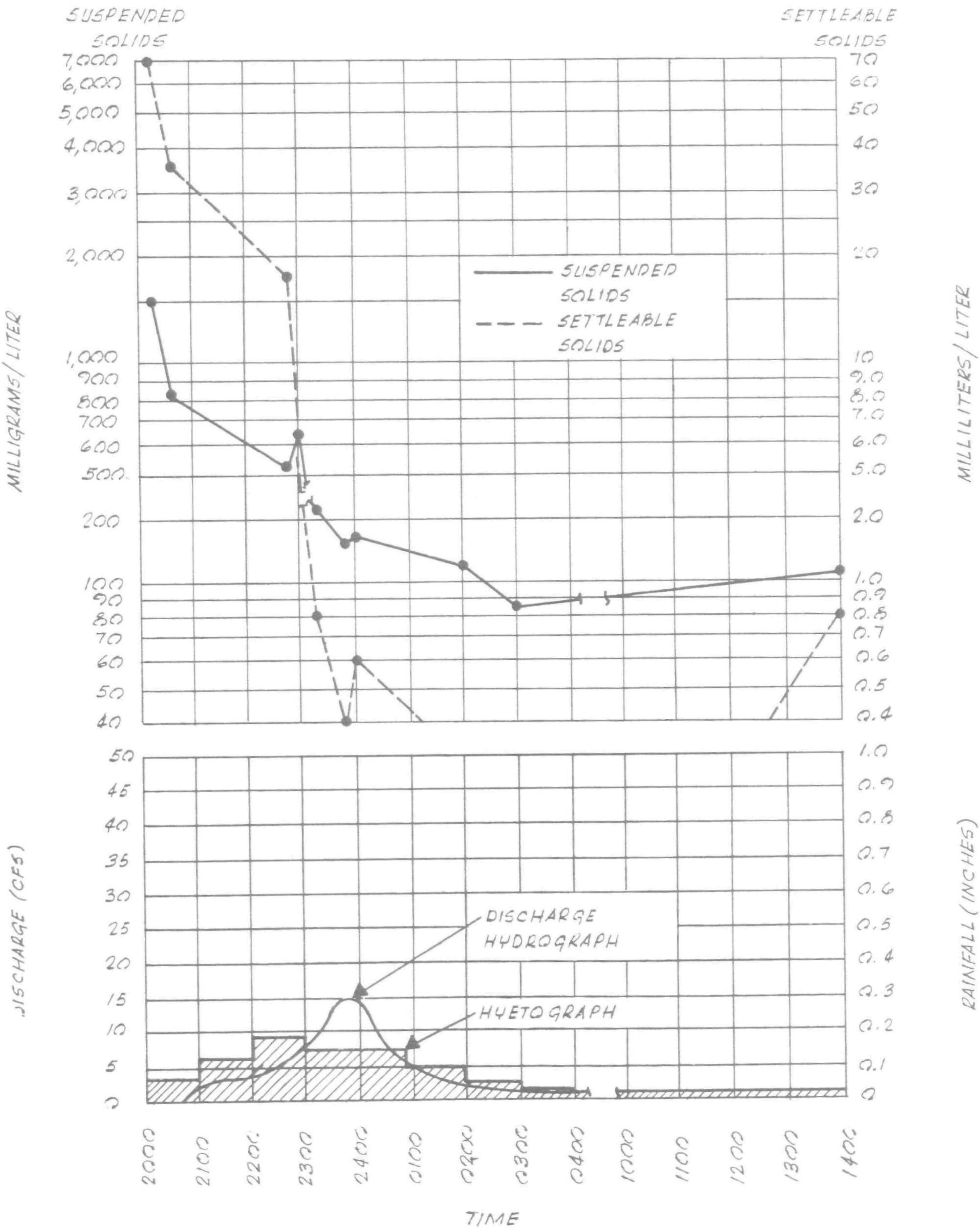
DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 2-8-69
2-9-69

Fig. 38

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 2-8-69
2-9-69

Fig. 39

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

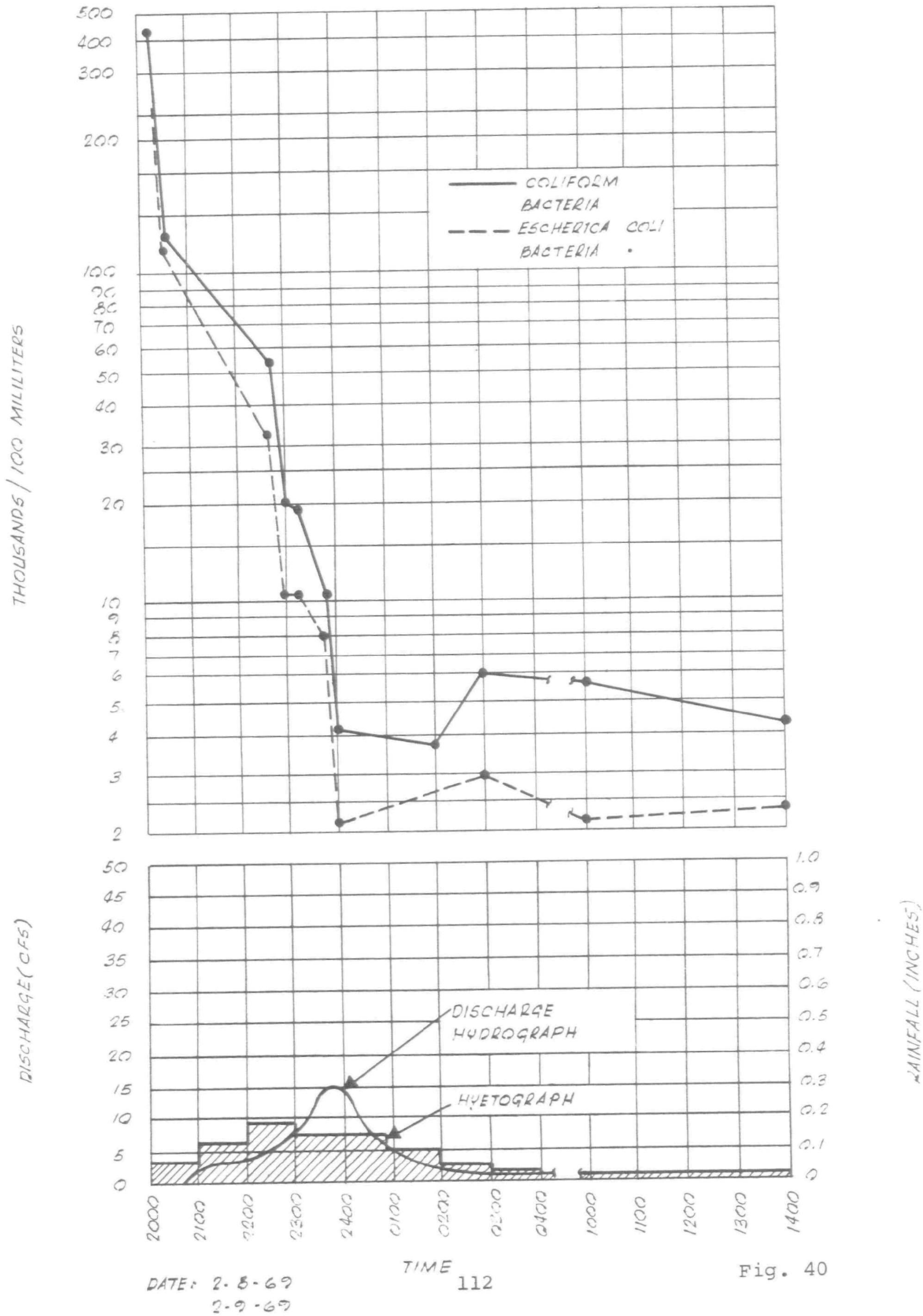


Fig. 40

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

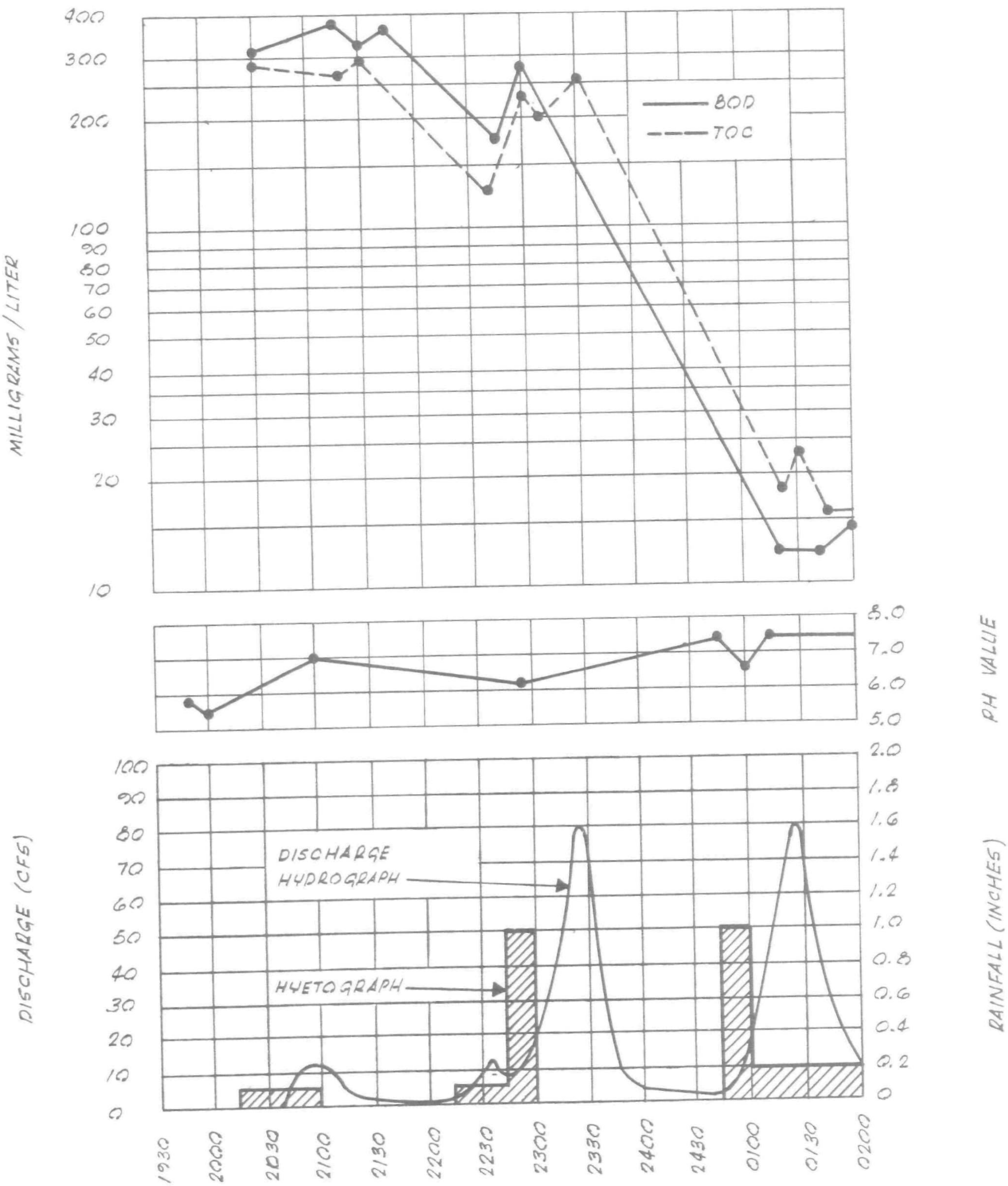


Fig. 41

DATE: 6-2-69
6-3-69

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

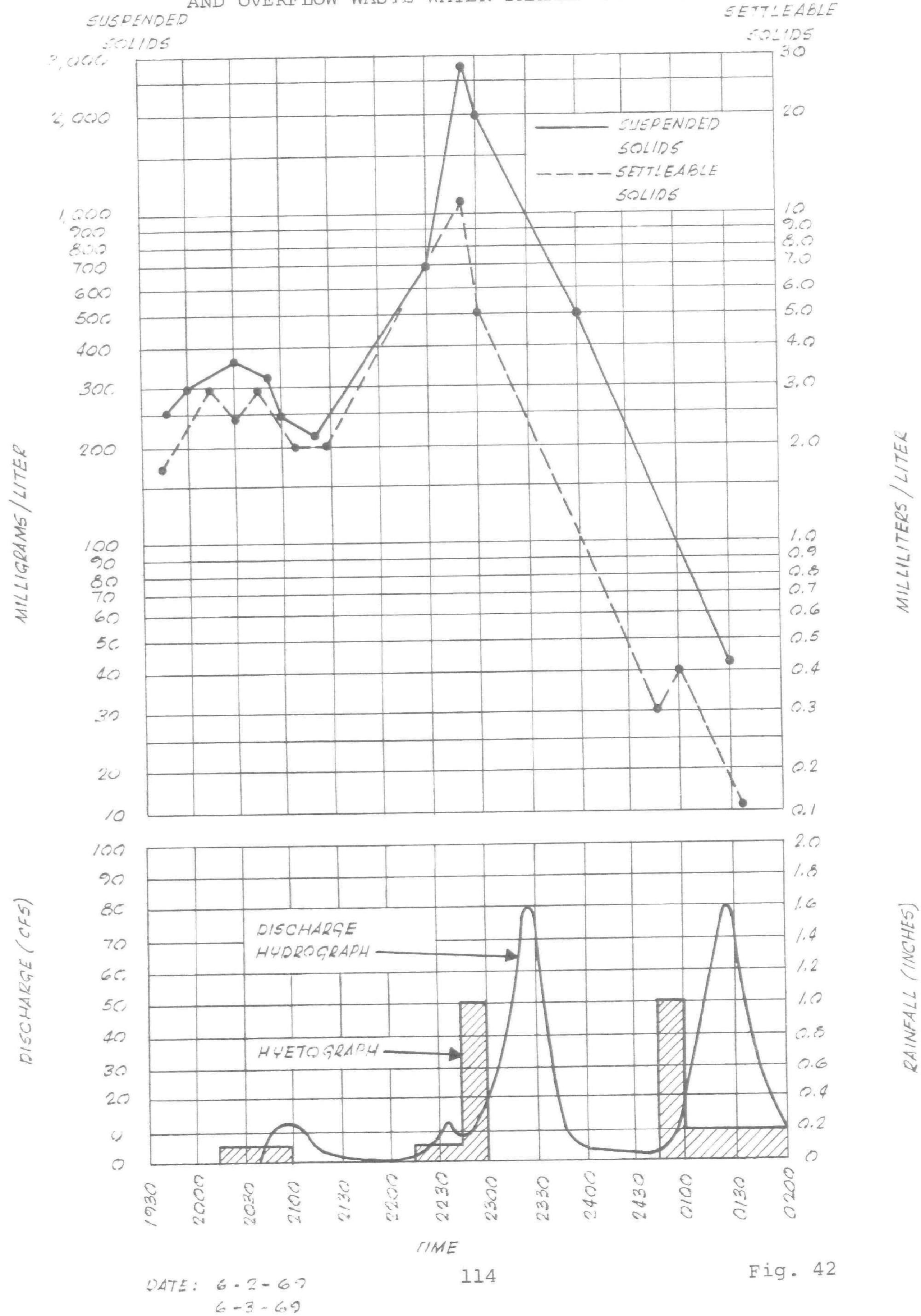
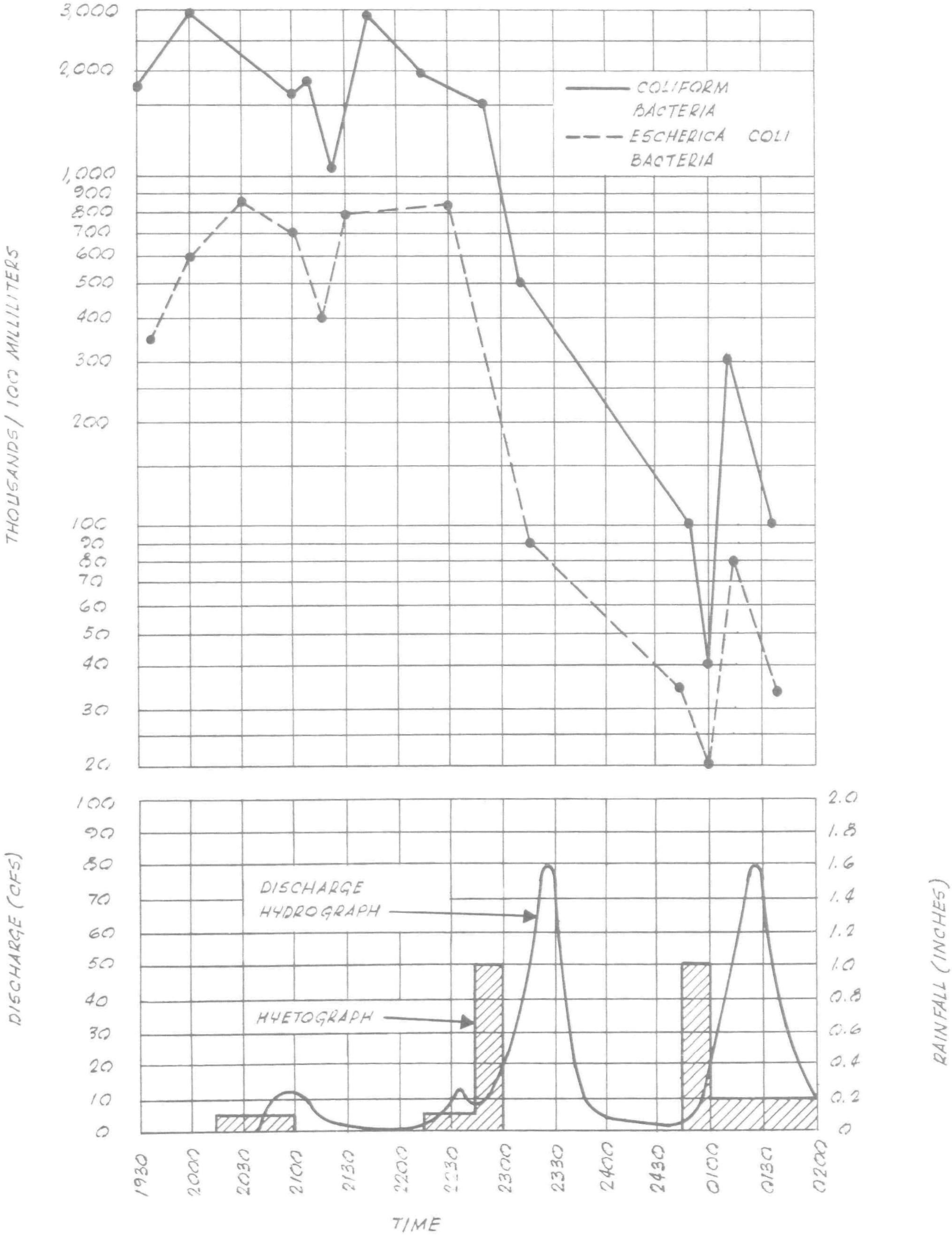


Fig. 42

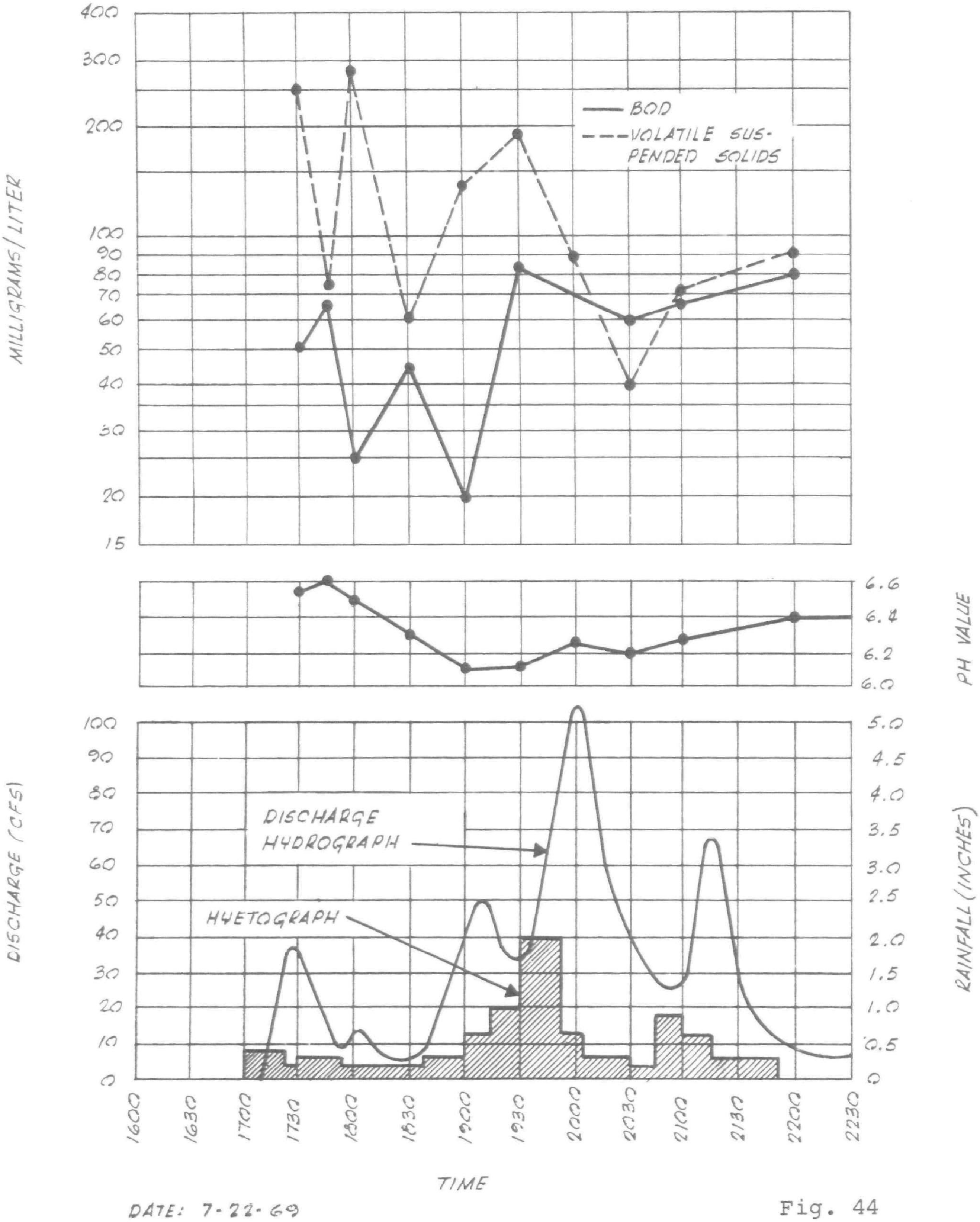
DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 6-2-69
6-3-69

Fig. 43

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 7-22-69

Fig. 44

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

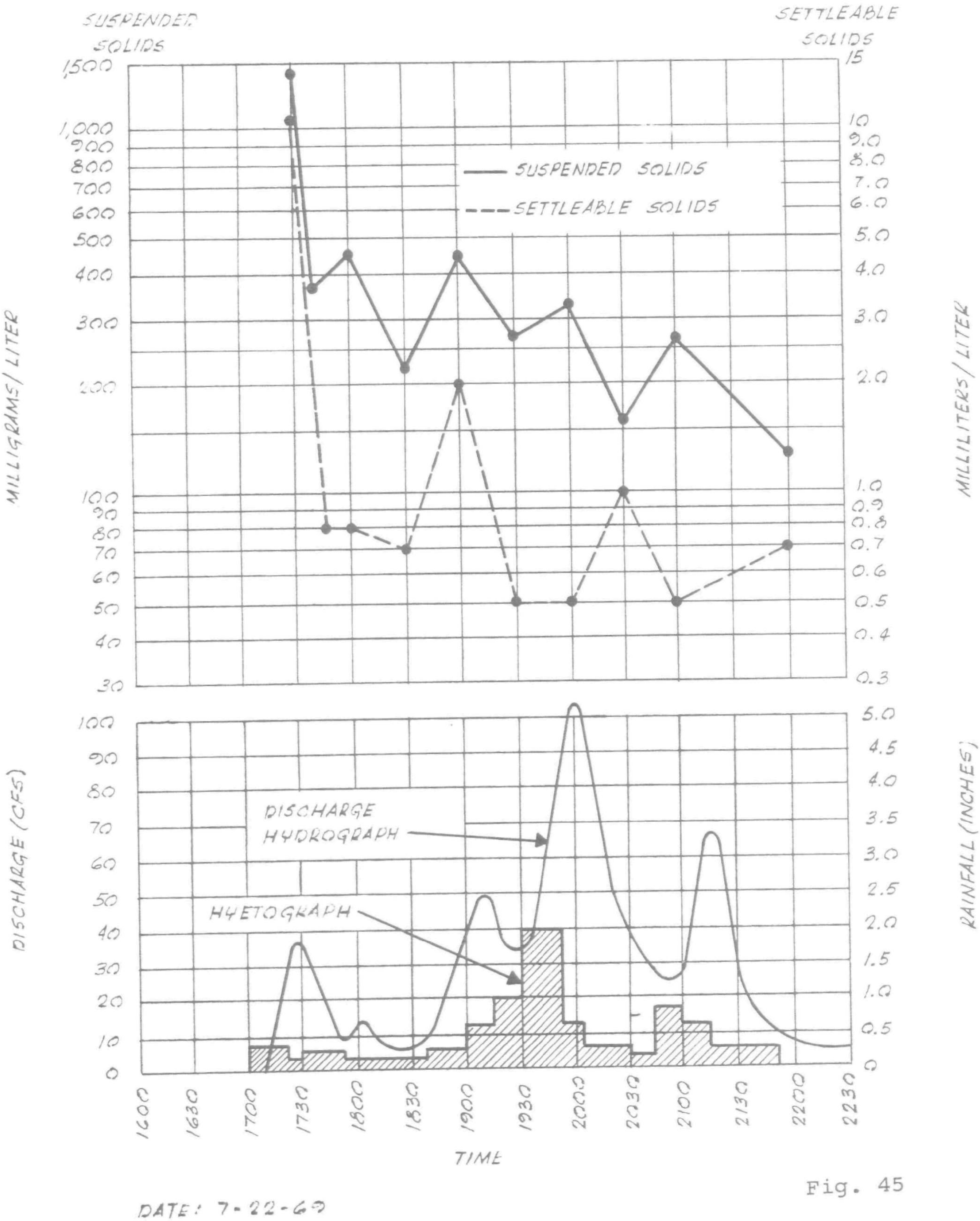
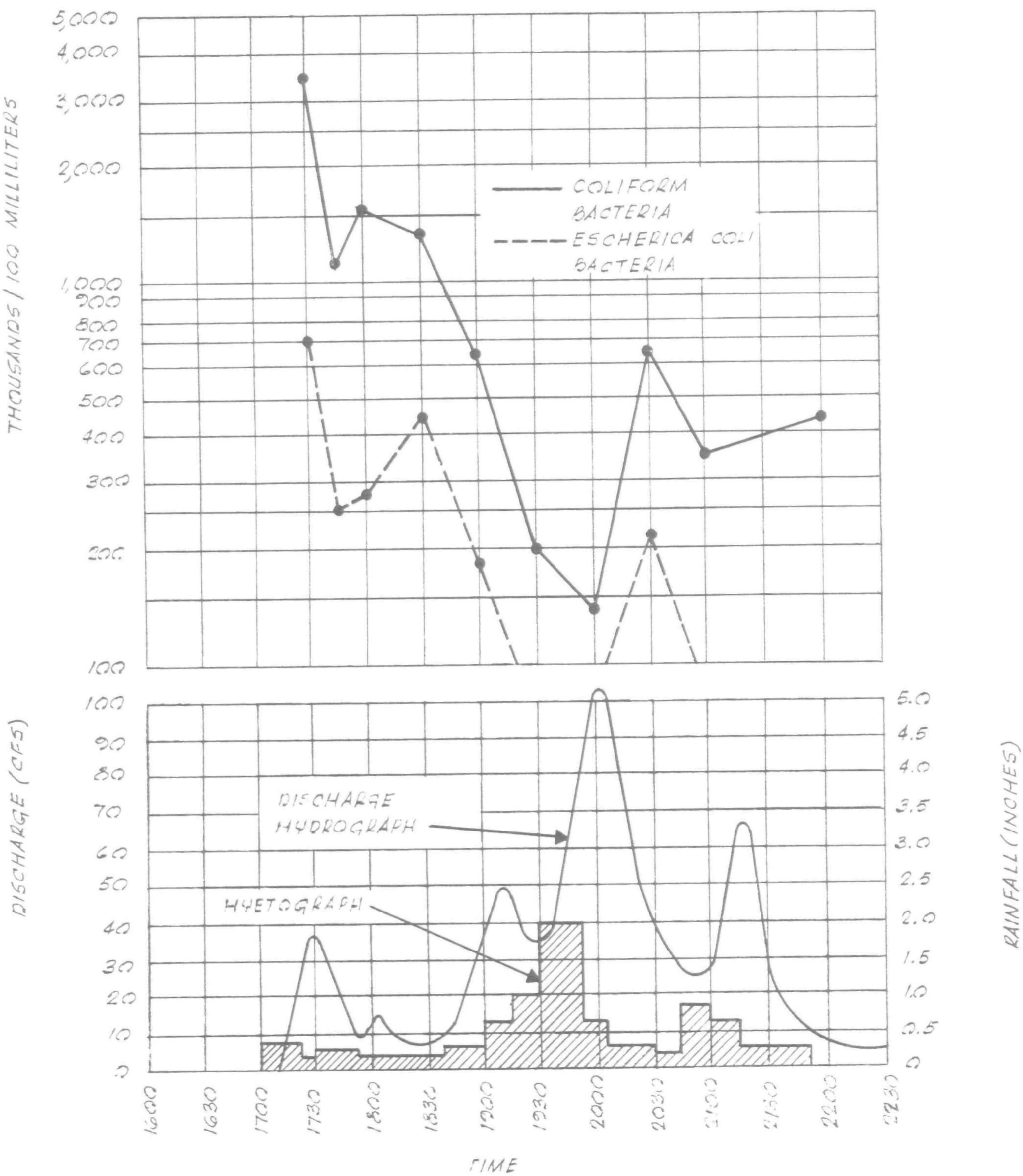


Fig. 45

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 7-22-69

Fig. 46

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

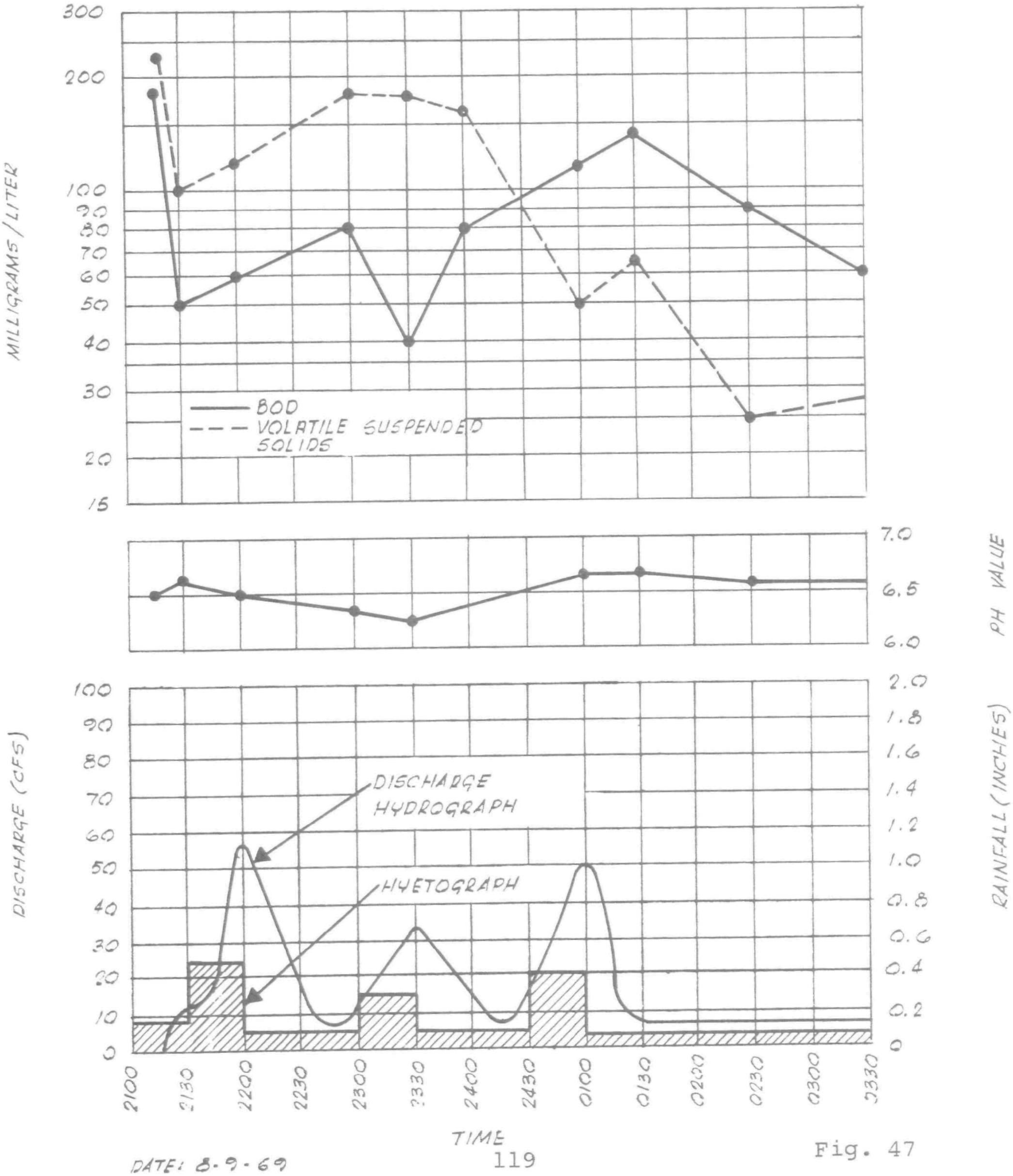


Fig. 47

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

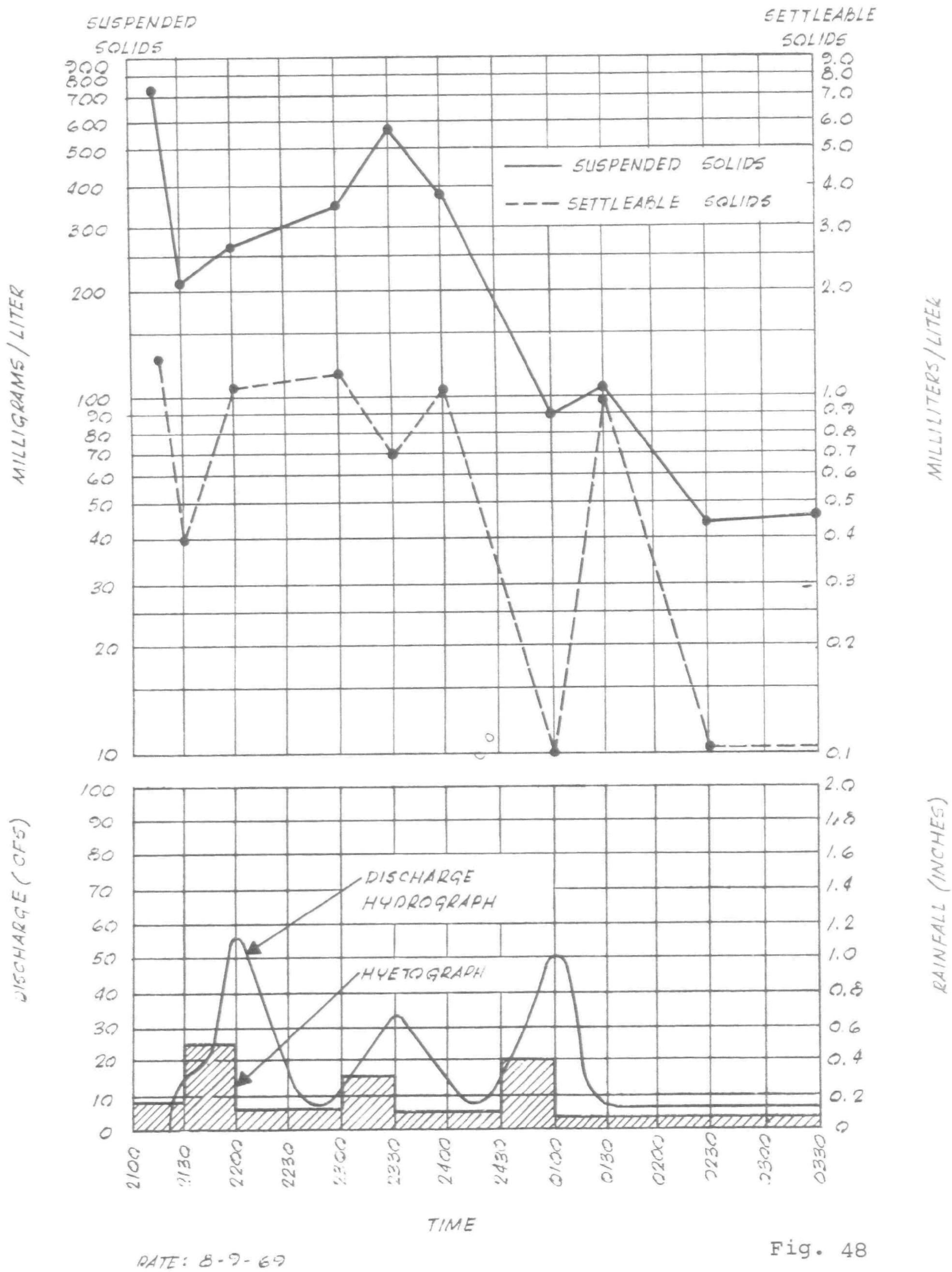


Fig. 48

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

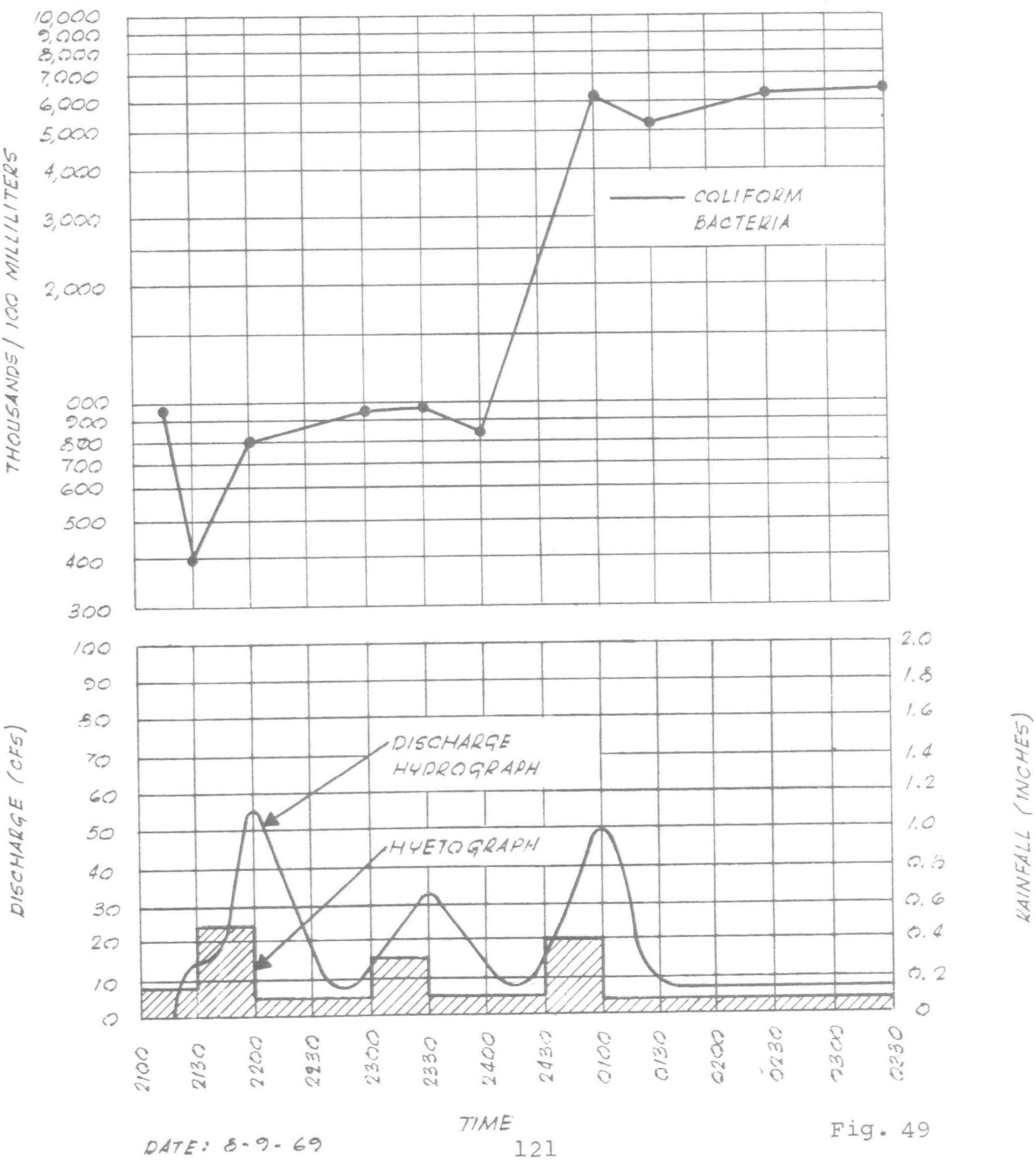


Fig. 49

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

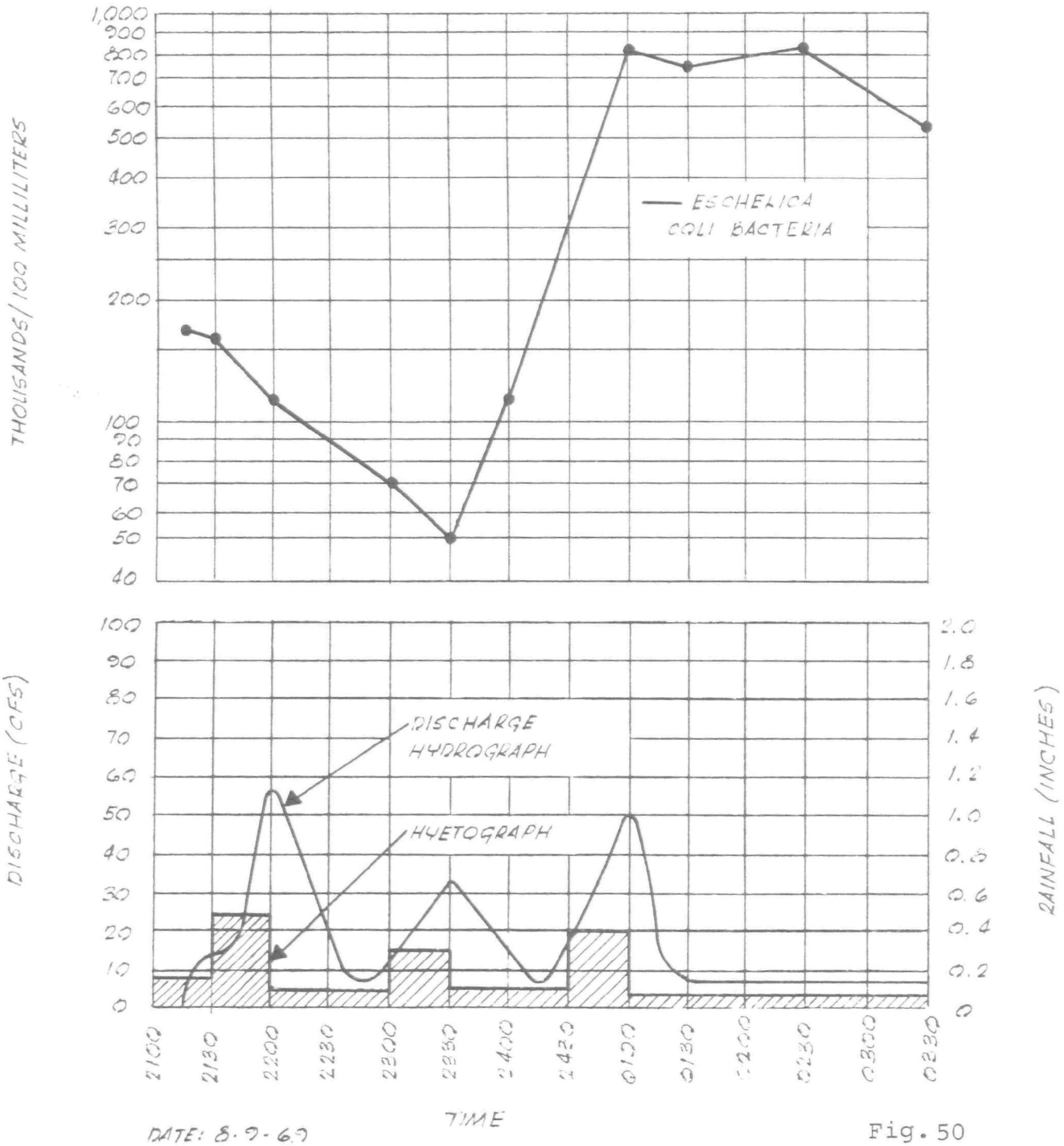


Fig. 50

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

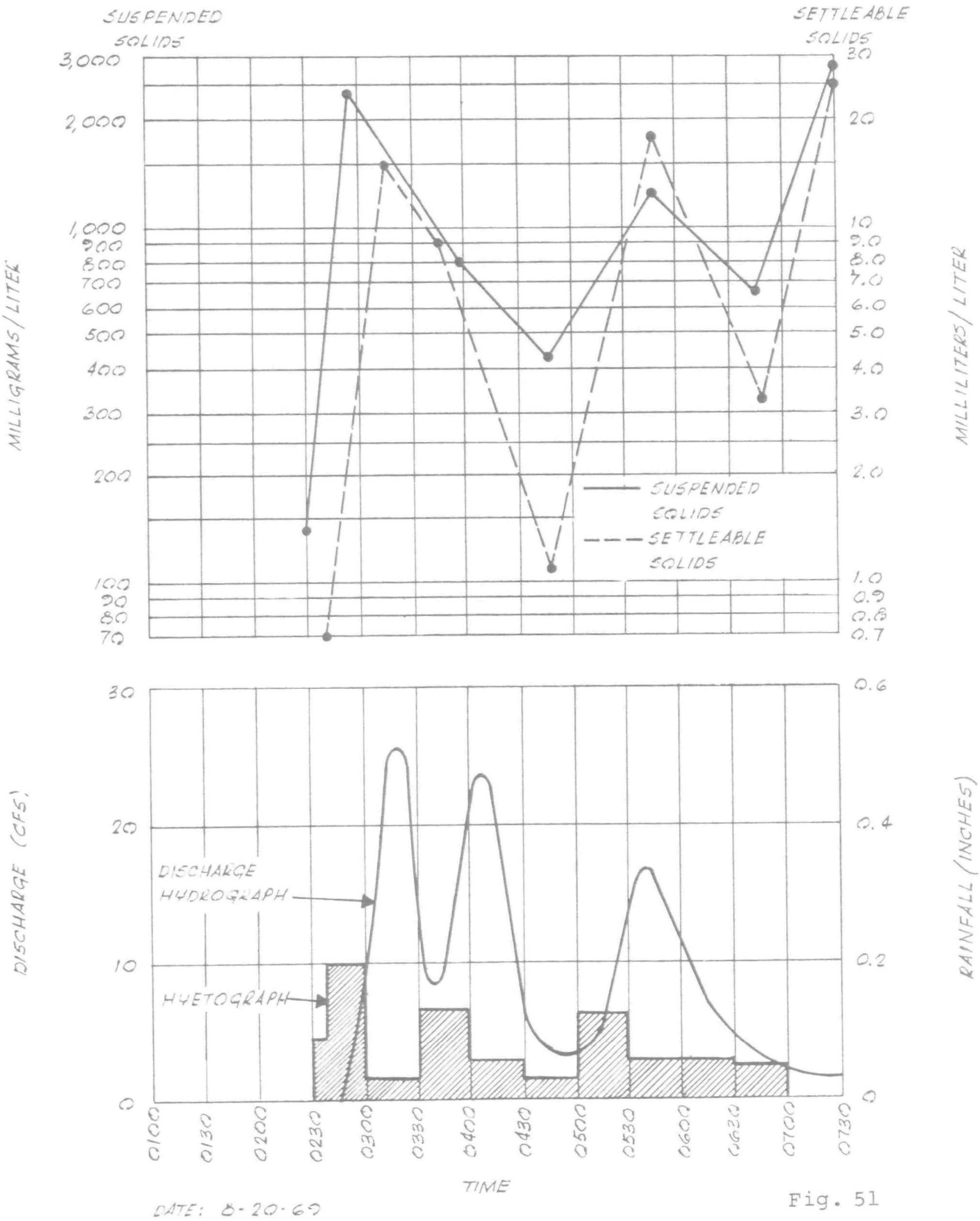


Fig. 51

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

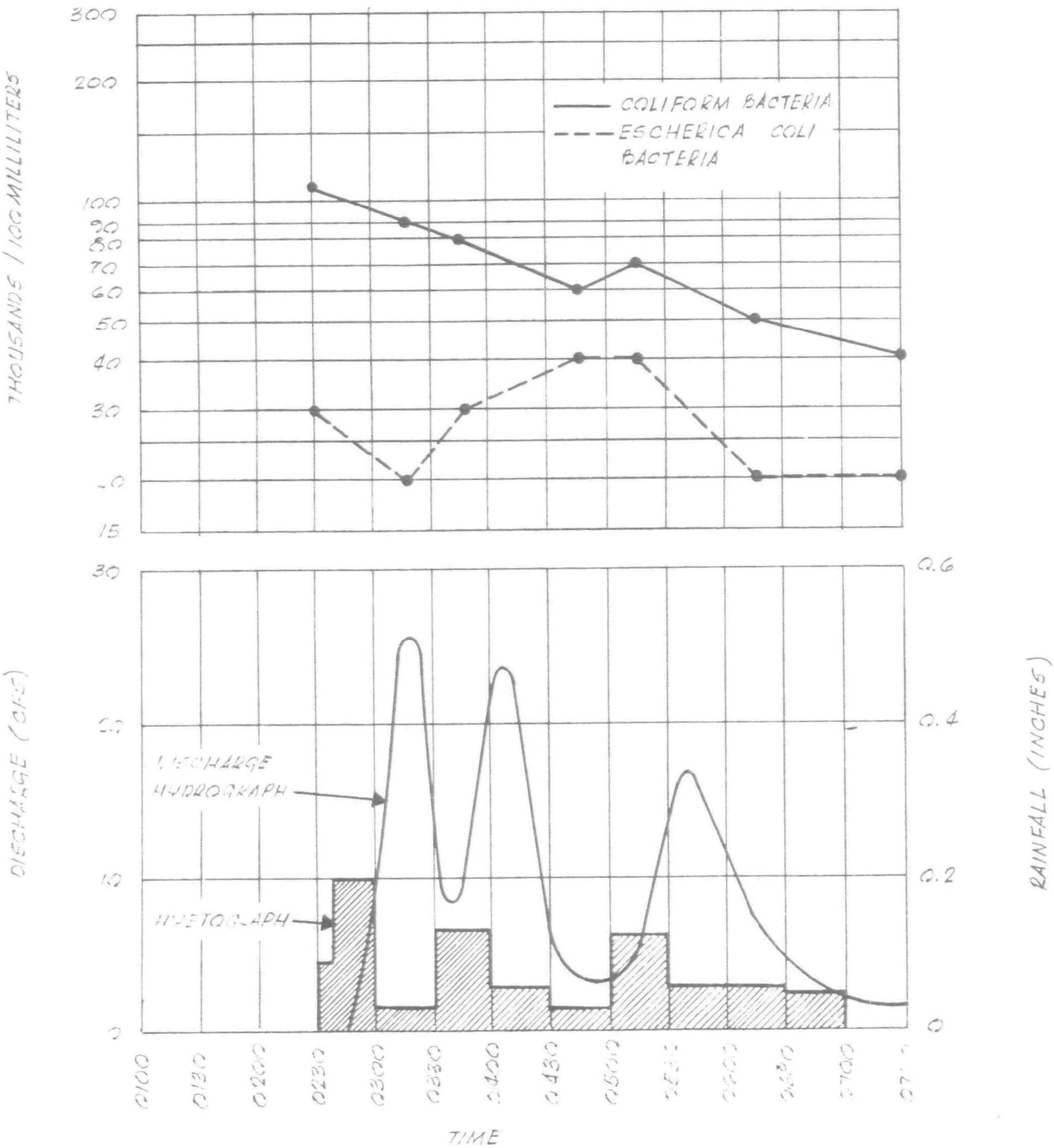
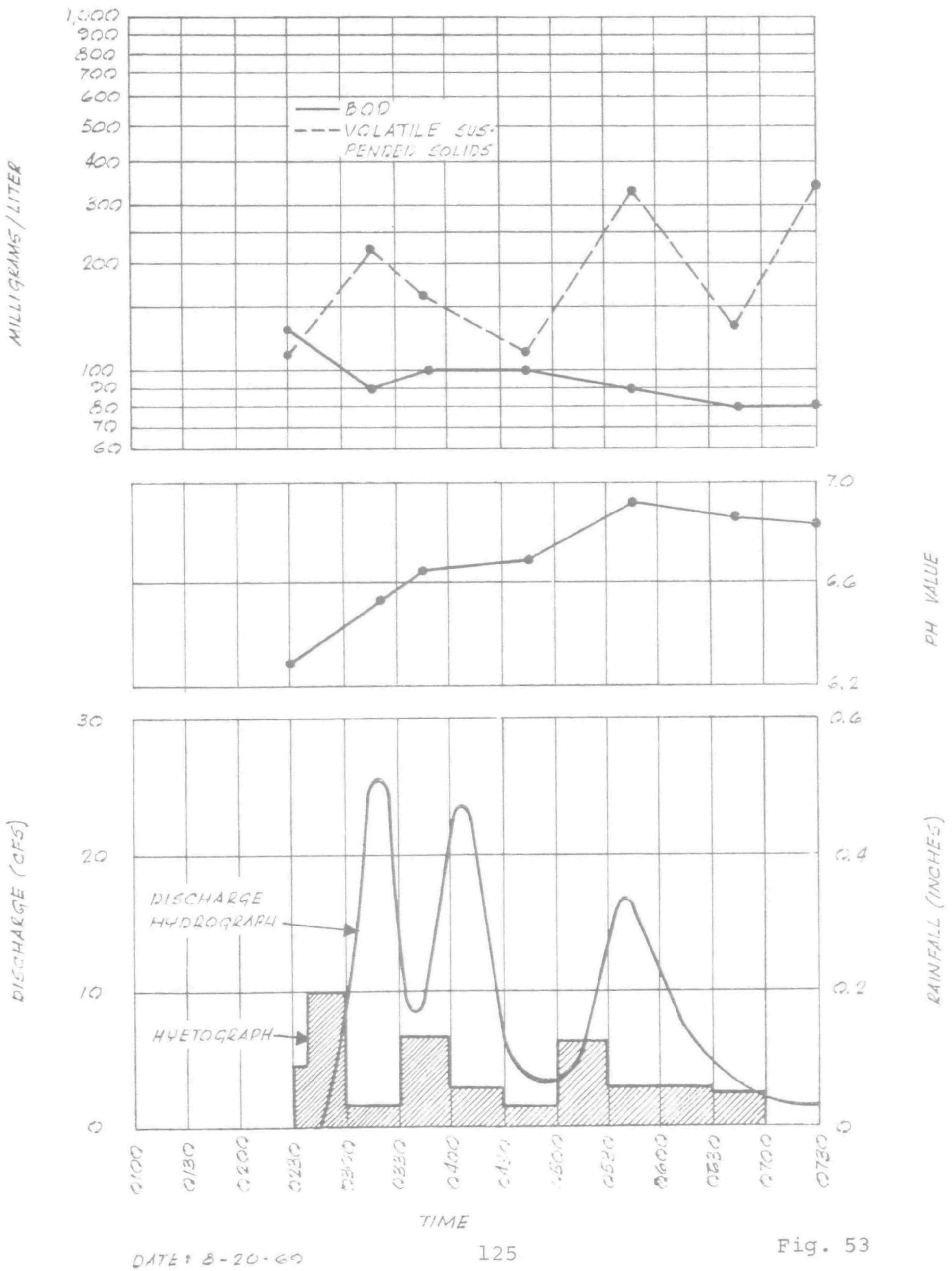


Fig. 52

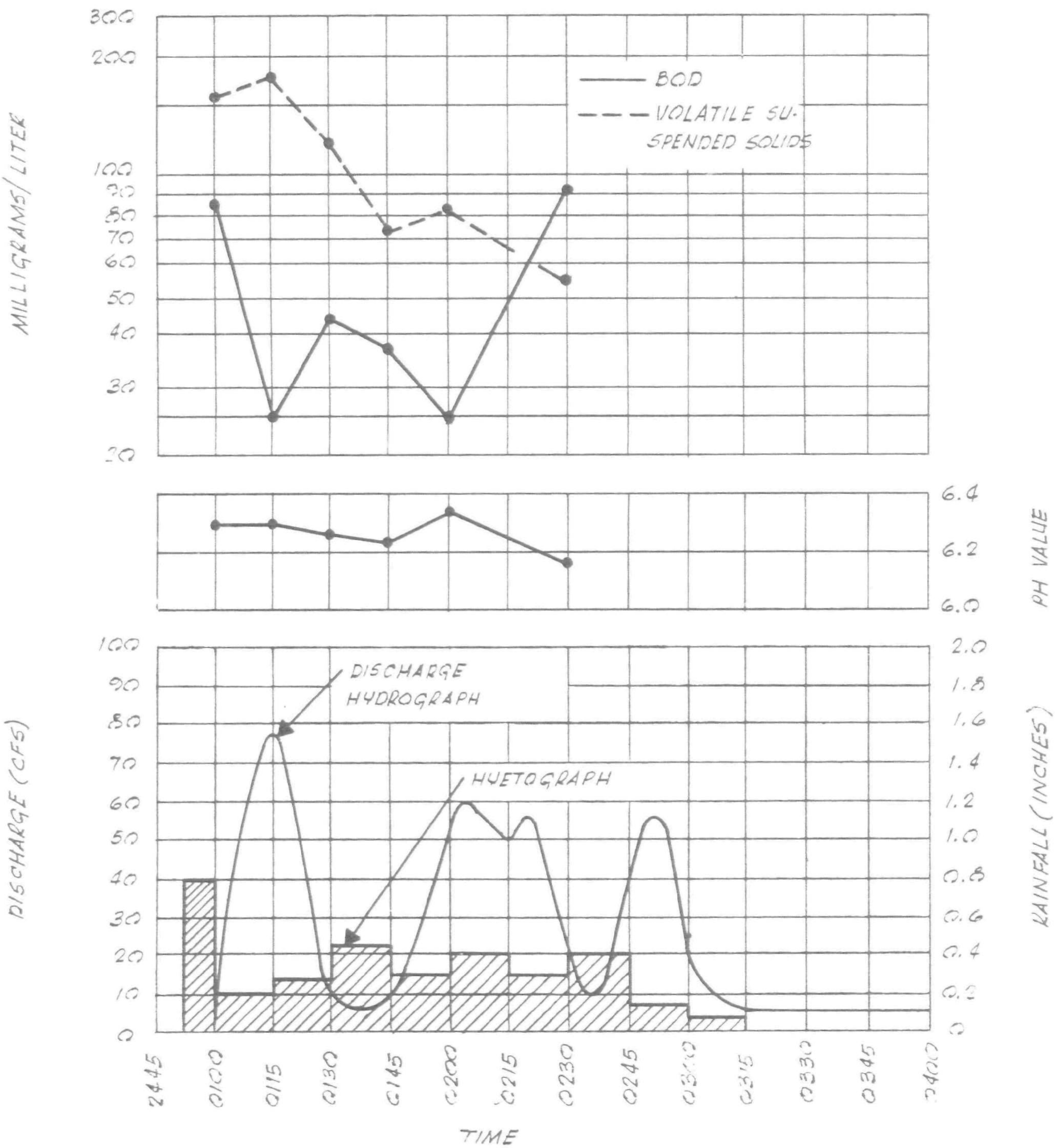
DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 8-20-60

Fig. 53

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS



DATE: 9-8-69

Fig. 54

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

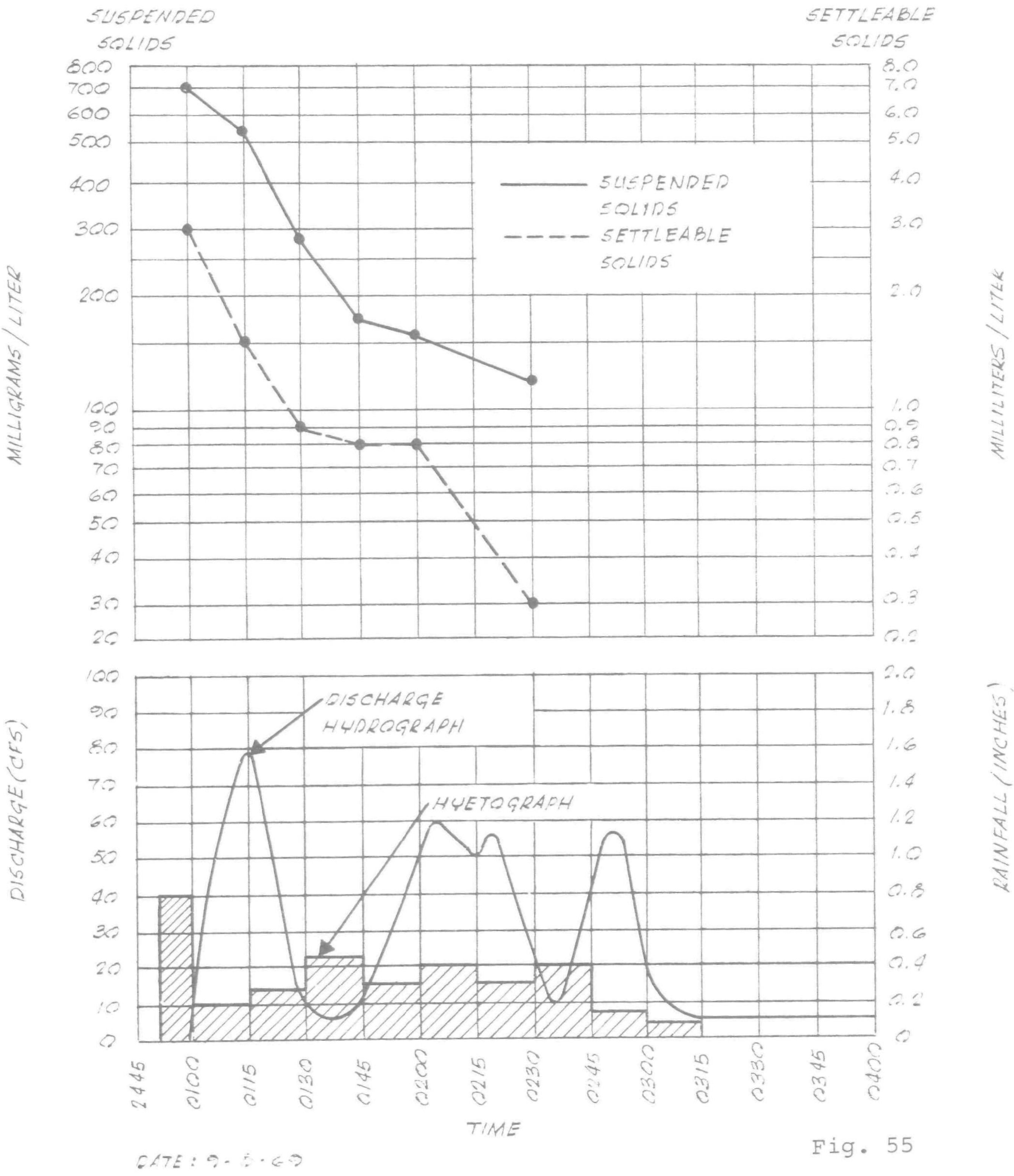


Fig. 55

DISCHARGE HYDROGRAPH, HYETOGRAPH
AND OVERFLOW WASTE WATER SAMPLE ANALYSIS

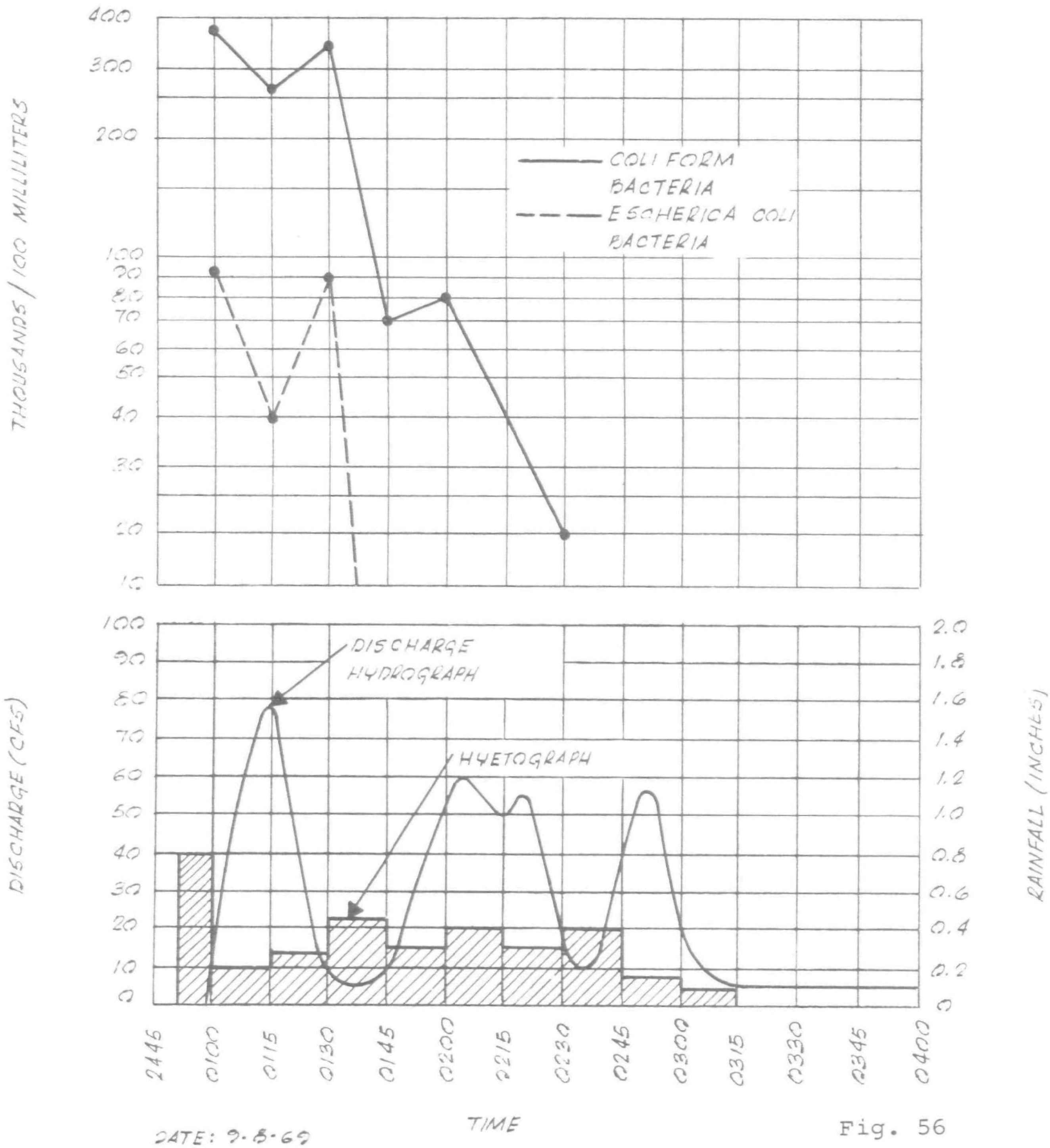


Fig. 56

SECTION 13

ACKNOWLEDGEMENTS

This demonstration facility was carried out by the Underwater Storage, Inc., Silver, Schwartz, Ltd. Joint Venture under contract No. 14-12-139 for the Federal Water Pollution Control Administration, Department of the Interior.

The particular concept of underwater storage of combined sewer overflow as herein reported was originally conceived by Dr. Harold G. Quase, President of Underwater Storage, Inc. Proprietary items used in the formulation of this project are based on patents assigned to Underwater Storage, Inc., by Dr. Quase.

Acknowledgement is made of the support and assistance of those who participated directly in this effort:

Mr. Grover E. Steele and Mr. Robert Viklund of the National Capitol Park Service, Department of the Interior, for their efforts in all aspects of construction and operation of the project.

The Goodyear Tire and Rubber Company, Industrial Products Division for the expeditious manner in which the underwater storage tanks were fabricated and delivered to the site.

Commander John A. Dearden, United States Coast Guard, for his assistance in establishing navigational aids at the project.

Mr. G. J. Maliszewski of the Potomac Electric Power Company for his personal efforts in providing electric service to the site on short notice.

Dr. Harold M. Windlan of The C. W. England Laboratories, Inc. for giving the project immediate service in processing of chemical analysis of waste samples.

The Analytical Services Laboratory at the Federal Water Pollution Control Administration Pilot Plant, Washington, D.C. for its assistance in chemical analyses through the initial phase of the project.

Mr. Howard L. Keller of Scullen, Keller and Marchigiani for structural design of land structures.

Mr. Duncan Gray, Consulting Engineer, for structural design of underwater facilities.

Mr. James J. Schnabel of Schnabel Engineering Associates for his report on soils and foundations.

Mr. Herbert G. McDonald of McDonald, Williams and Marshall for architectural design of pump house.

Penniman and Browne, Incorporated for test borings and river soundings.

Special thanks are given to Mr. George Kirkpatrick and Mr. William Rosenkranz of Federal Water Pollution Control Administration for their comments during the course of the program, which provided valuable guidance in the evaluation of the system.

The project was administered and supervised by Underwater Storage, Inc.; Dr. Harold G. Quase was Project Director, and Mr. H. C. John Russell was Project Supervisor.

The project was designed and operated by Silver, Schwartz, Ltd. Mr. Sidney A. Silver, P.E. was Chief Engineer; Mr. Harold Schwartz, P.E. was Design Engineer, and Mr. Irving T. Read was the Field Engineer.

SECTION 14

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SECTION 15

PATENTS AND PAPERS

Patent No. 3,114,468 - H. G. Quase assignor to Underwater Storage, Inc. "Collapsible Container," dated December 17, 1963.

Patent No. 3,114,384 - H. G. Quase assignor to Underwater Storage, Inc. "Underwater Storage System," dated December 17, 1963.

Patent No. 3,155,380 - H. G. Quase assignor to Underwater Storage, Inc. "Buoyant Flexible Container and Underwater Anchorage Therefor," dated November 3, 1964.

Patent No. 3,187,793 - H. G. Quase assignor to Underwater Storage, Inc. "Amphibious Underwater Storage System," dated June 8, 1965.

Paper prepared by Underwater Storage, Inc., "Underwater Storage of Overflow from Combined Sanitary and Storm Water Sewers."

Paper prepared by Underwater Storage, Inc., "Demonstration Underwater Storm Sewer Overflow and Storage Facility."

SECTION 16

APPENDIX

Grading Plan

Borings and Tank Location Plan

Storage Tank Pile and Frame Layout

Meter and Pump House Elevations

Project Piping Distribution

Overflow Sewer Plan and Site Plan

Parshall Flume Plan and Sections

Parshall Flume Details

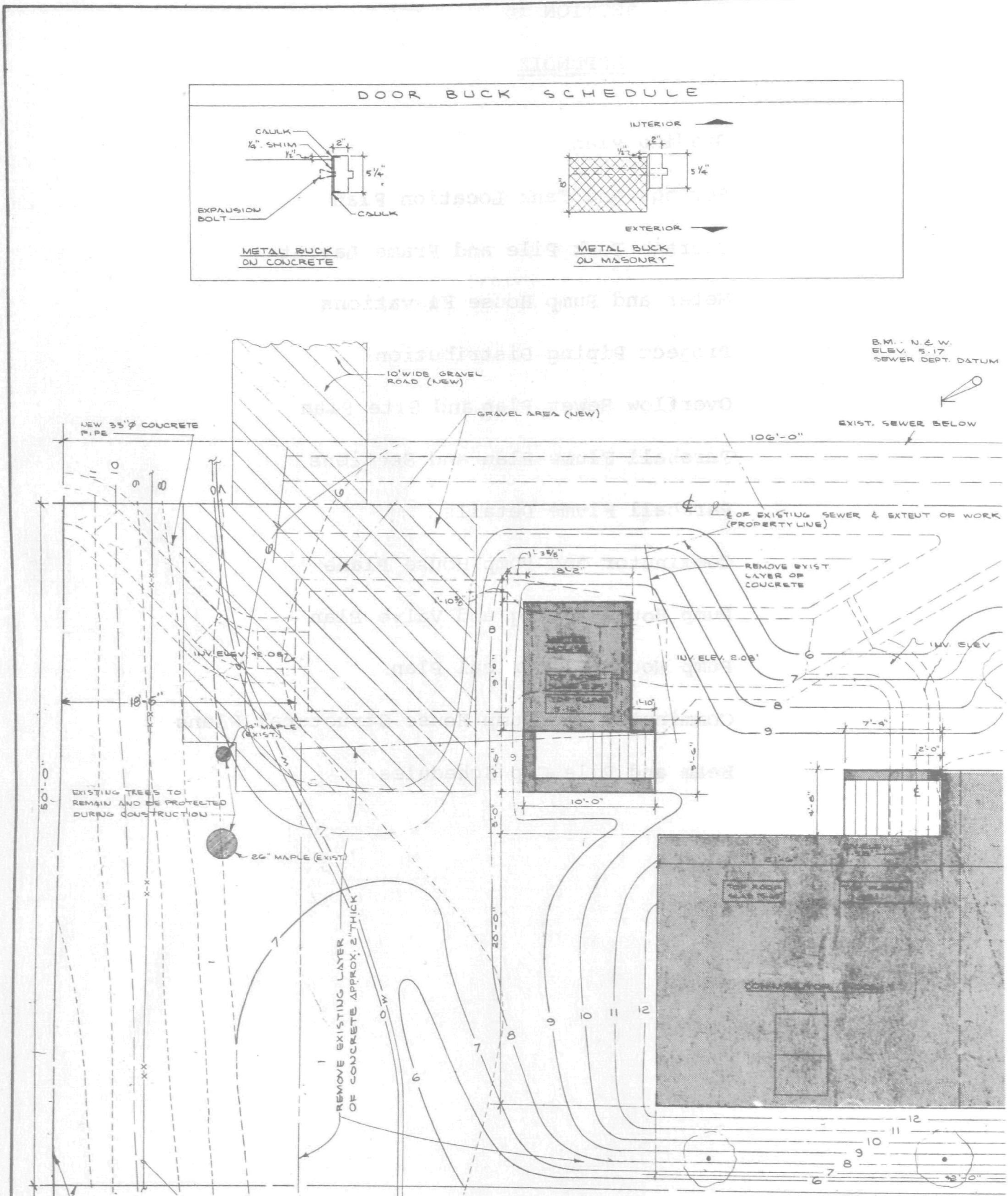
Comminutor and Pump House Plans

Pump House Piping and Valve Plan

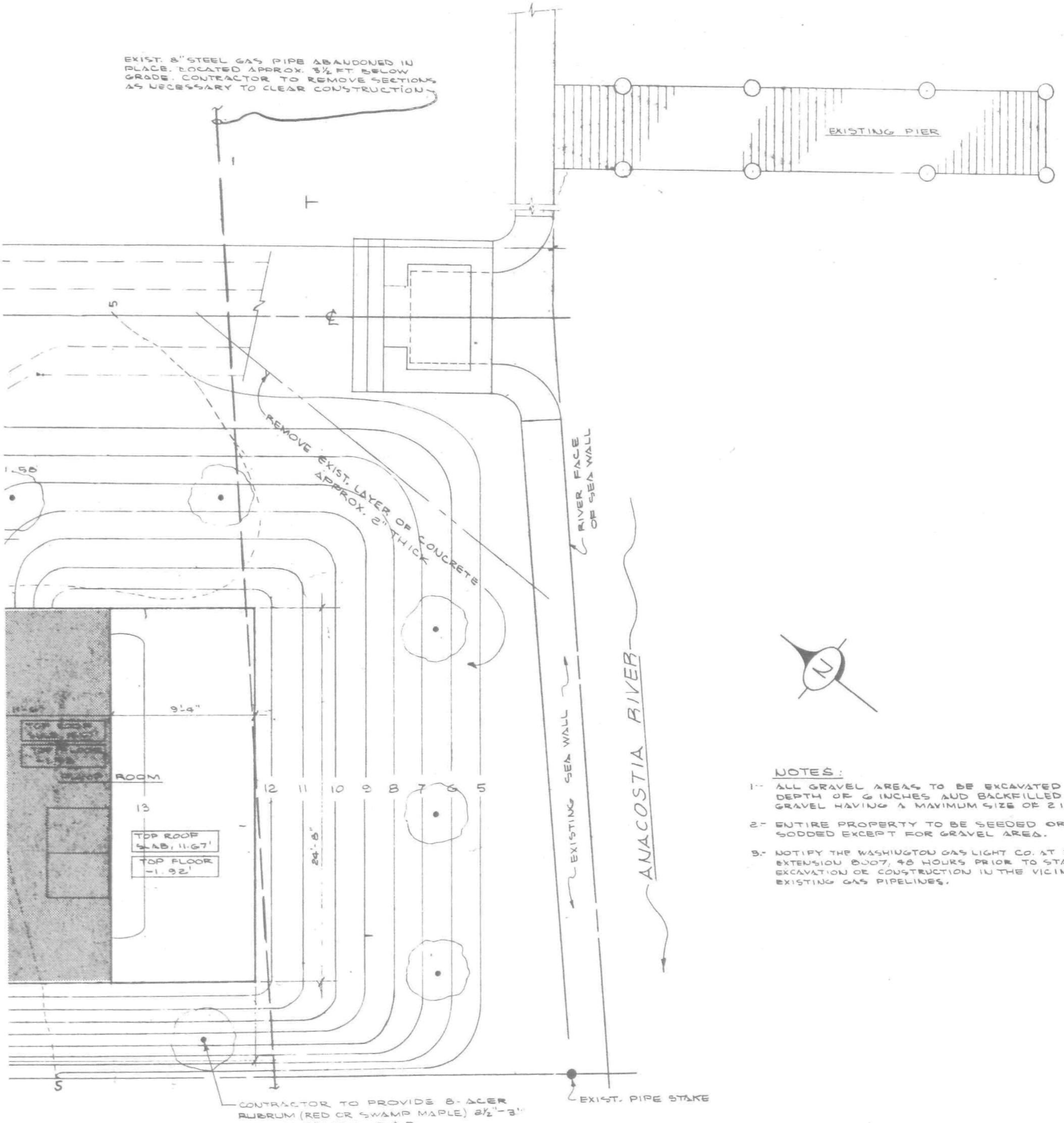
Pump House Electrical Plan

Comminutor and Pump House Structural Plans

Beam and Pile Cap Schedules


DOOR BUCK SCHEDULE
CAULK
¼" SHIM
½"
2"
5¼"
EXPANSION BOLT
CAULK
METAL BUCK ON CONCRETE
INTERIOR
EXTERIOR
8"
METAL BUCK ON MASONRY
B.M. - N.E.W.
ELEV. 5.17
SEWER DEPT. DATUM
10' WIDE GRAVEL ROAD (NEW)
GRAVEL AREA (NEW)
NEW 33"Ø CONCRETE PIPE
EXIST. SEWER BELOW
106'-0"
℄ OF EXISTING SEWER & EXTENT OF WORK (PROPERTY LINE)
REMOVE EXIST LAYER OF CONCRETE
INV. ELEV. 2.05'
INV. ELEV
18'-6"
EXISTING TREES TO REMAIN AND BE PROTECTED DURING CONSTRUCTION
26" MAPLE (EXIST.)
50'-0"
10'-0"
7'-4"
2'-0"
4'-8"
REMOVE EXISTING LAYER OF CONCRETE APPROX. 2" THICK
20'-0"
PROPERTY LINE & R.R. RIGHT OF WAY
℄ 12" H.P. GAS LINE (EXIST.)
℄ 4" C.I. WATERMAIN (EXIST.)
℄ 8" OIL LINE (EXIST.)
GRADING PLAN
SCALE: ¼" = 1'-0"

FINISH SCHEDULE				
AREA	FLOOR	WALLS	CEILING	REMARKS
METER ROOM	UNFINISHED CONCRETE FLUME	PAINTED CMU	UNFINISHED CONCRETE	
PUMP ROOM	UNFINISHED CONCRETE WITH HARDENER	PAINTED CMU.	UNFINISHED CONCRETE	
GRIT CHAMBERS & COMMINUTOR ROOM	LIQUID TILE COATING ON CONCRETE	LIQUID TILE COATING ON CONCRETE	UNFINISHED CONCRETE	TILE COATING PER SPECIFICATIONS SCHED.
CONCRETE ROOFS & WALLS ABOVE GRADE	NOT APPLICABLE	NOT APPLICABLE	NOT APPLICABLE	PAINT PER SPECIFICATION SCHEDULE



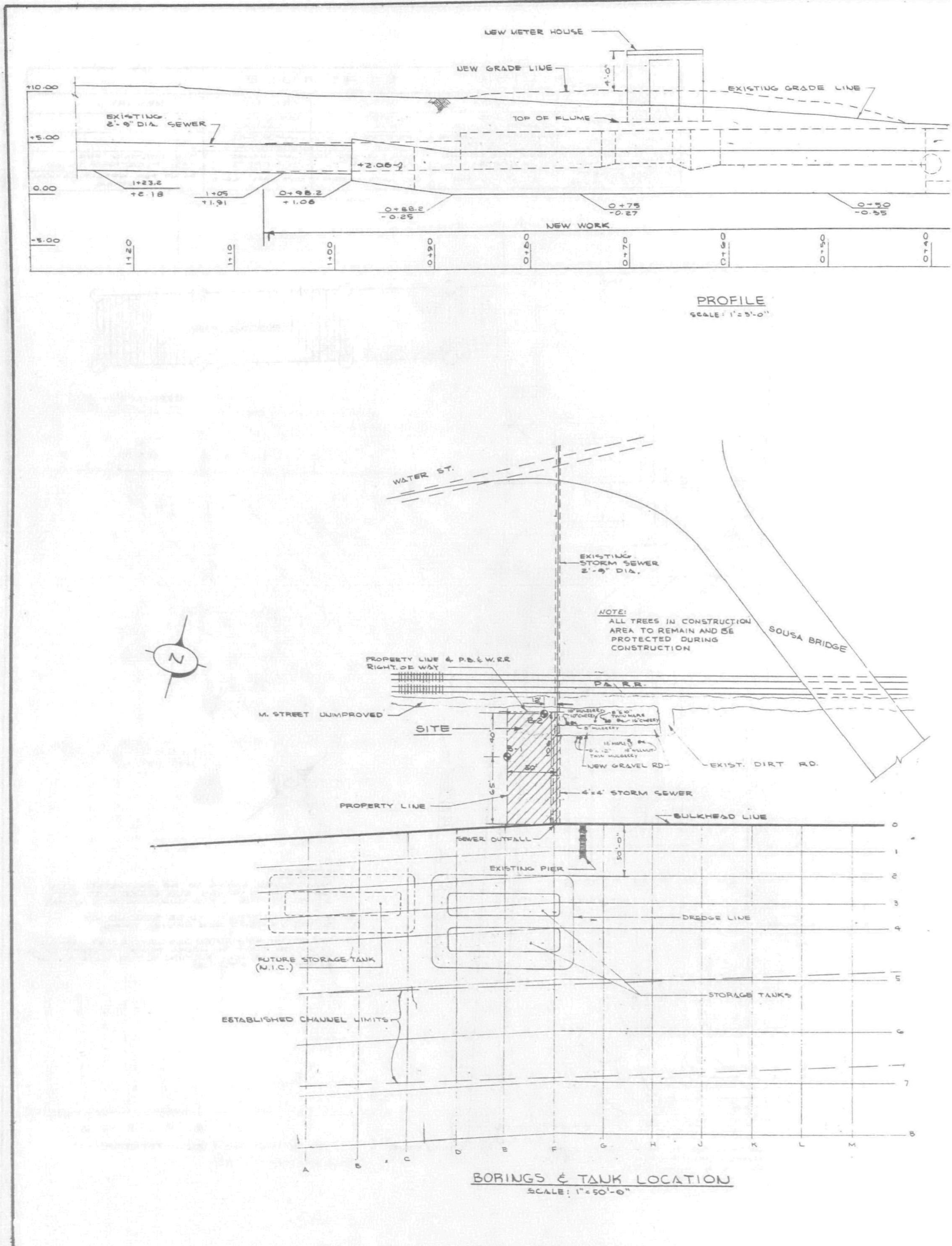
NOTES:

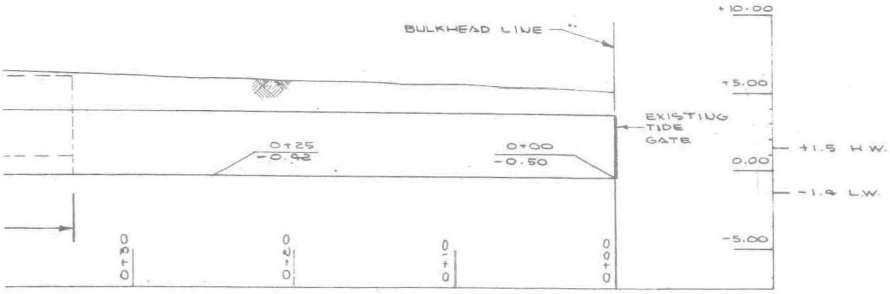
1.- ALL GRAVEL AREAS TO BE EXCAVATED TO A DEPTH OF 6 INCHES AND BACKFILLED WITH GRAVEL HAVING A MAXIMUM SIZE OF 2 INCHES

2.- ENTIRE PROPERTY TO BE SEEDED OR SODDED EXCEPT FOR GRAVEL AREA.

3.- NOTIFY THE WASHINGTON GAS LIGHT CO. AT 783-5225 EXTENSION 8007, 48 HOURS PRIOR TO START OF ANY EXCAVATION OR CONSTRUCTION IN THE VICINITY OF EXISTING GAS PIPELINES.

Borings and Tank Location Plan





BORING NO B-1
DATE TAKEN 7-17-67
WATER LEVEL UPON COMPLETION - 2'-0"
WATER LEVEL MEASURED FROM GROUND SURFACE

DEPTH	SAMPLE NO	SAMPLE BLOWS/6"	VISUAL CLASSIFICATION
0			
3.5	1	1-2-2	CINDER & MISC. FILL SOME CLAY-BLACK
8.5	2	1-½-½	
11			DARK GRAY SILT-SOME CLAY TRACE ORGANICS & SHELLS
13.5	3	½-1-1	
18.5	4	2-1-2	
20			GRAY SILT-SOME CLAY TRACE ORGANICS & SHELLS
23.5	5	HAMMER	
25			DARK GRAY SILT-SOME CLAY TRACE ORGANICS & SHELLS
28.5	6	1-1-2	
31			BROWN CLAY-SOME SILT & SAND ORGANICS & SHELLS
33.5	7	2-2-3	
37			RED & GRAY CLAY-SOME SILT TRACE SAND-ORGANIC LENSES BROWN & GRAY FINE SAND SOME SILT-TRACE CLAY SEAMS
38.5	8	3-4-5	
39			
42			
43.5	9	4-7-8	RED & GRAY CLAY-TRACE SAND & SILT TRACE WOOD-SILT LENSES
48.5	10	8-10-15	
53.5	11	10-15-18	
55			GAS @ 38'-FILL VERY LOOSE & SATURATED-HEAVY WATER FLOW IN 6" SEAM @-4'

ELEV.: 105, 100, 95, 90, 85, 80, 75, 70, 65, 60, 55, 50

BORING NO. B-2
DATE TAKEN 7-17-67
WATER LEVEL UPON COMPLETION 3'-0"
WATER LEVEL MEASURED FROM GROUND SURFACE

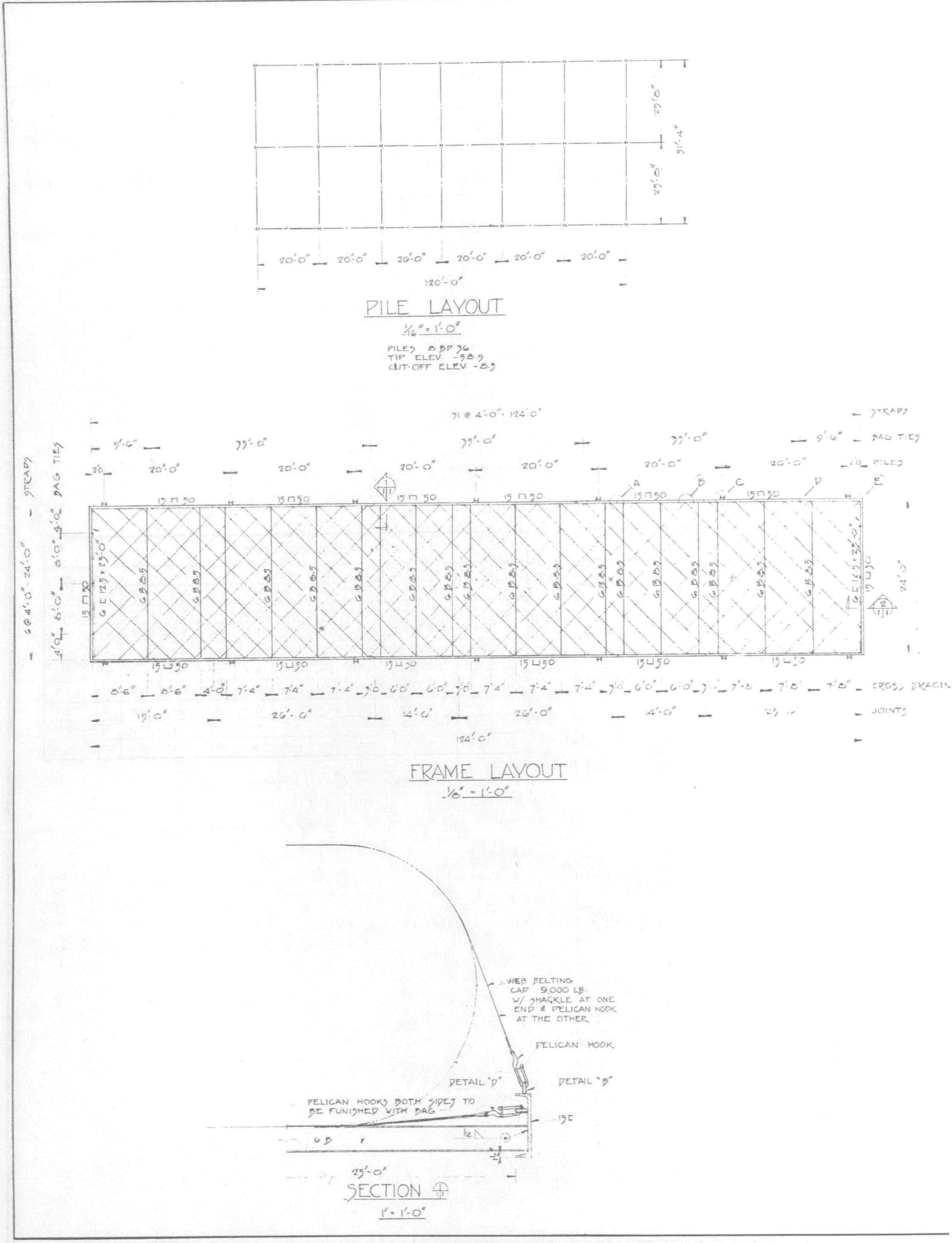
DEPTH	SAMPLE NO	SAMPLE BLOWS/6"	VISUAL CLASSIFICATION
0			
3.5	1	1-1-2	CINDER-WOOD-ORGANIC SILT MISC. FILL-BLACK.
5			GRAY SILT-SOME CLAY TRACE ORGANICS & SHELLS
8.5	2	1-2-2	
13.5	3	1-2-3	
14.5			BROWN SILT SAND & GRAVEL TRACE CLAY
16			DARK GRAY SILT-SOME CLAY TRACE ORGANICS & SHELLS
18.5	4	1-1-2	
23.5	5	1-1-1	
28.5	6	2-8-5	
29			GRAY SAND & GRAVEL SOME SILT TRACE CLAY & ORGANICS
32			BROWN SAND & GRAVEL SOME SILT & CLAY
33.5	7	12-11-12	
36			RED & GRAY CLAY-TRACE SAND & SILT TRACE WOOD SILT LENSES
38.5	8	6-11-15	
43.5	9	7-10-13	
45.0			HEAVY WATER FLOW @ 5' RED & GRAY CLAY HAS SOFT LENSES THROUGHOUT SAMPLES FAILED INSIDE SPOON.

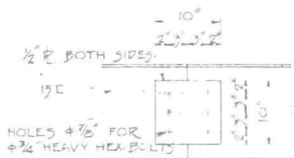
NOTE: SPOON O.D - 2"
SPOON I.D. - 1½"
HAMMER WEIGHT - 140 LBS.
HAMMER FALL - 30"

RIVER SOUNDINGS

	A	B	C	D	E	F	G	H	J	K	L	M
0	1.4'	1.2'	1.1'	0.9'	0.8'	0.6'	0.6'	0.6'	0.6'	0.5'	0.5'	0.5'
1	8.3'	6.2'	4.3'	4.2'	4.2'	4.2'	4.2'	4.8'	5.4'	6.3'	6.3'	6.7'
2	8.8'	5.0'	9.3'	9.3'	9.4'	9.3'	9.4'	9.4'	9.5'	8.4'	8.4'	9.5'
3	11.0'	10.2'	10.1'	10.0'	8.9'	8.8'	8.8'	9.1'	9.7'	10.2'	10.2'	10.4'
4	12.0'	10.2'	10.1'	10.0'	10.5'	9.4'	9.4'	9.4'	9.5'	9.6'	9.6'	10.2'
5	11.8'	11.3'	11.0'	11.0'	10.4'	10.4'	10.4'	10.6'	10.6'	10.5'	10.5'	10.9'
6	10.4'	10.3'	10.2'	10.2'	10.4'	10.4'	10.4'	10.4'	10.4'	10.4'	10.4'	10.7'
7	10.3'	10.3'	10.4'	10.1'	9.7'	9.5'	9.5'	9.5'	9.5'	10.4'	10.4'	9.8'
8	9.8'	9.9	9.7'	9.9'	10.4'	10.5'	10.5'	10.5'	10.2'	10.0'	10.0'	10.0'

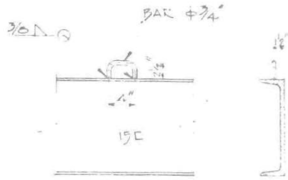
NOTE:
4 FOOT SEWER INVERT REFERENCE - 0
DEPTH CORRECTED FOR TIDAL FLUCTUATIONS.


20'-0"
20'-0"
20'-0"
20'-0"
20'-0"
20'-0"
120'-0"
25'-8"
25'-8"
51'-4"
PILE LAYOUT
1/16" = 1'-0"
PILES 8 BP 36
TIP ELEV. -58.5
CUT-OFF ELEV. -8.5
31 @ 4'-0" - 124'-0"
STRAPS
BAG TIES
PILES
9'-6"
33'-0"
33'-0"
33'-0"
9'-6"
20'-0"
A
B
C
D
E
15 ⊓ 50
15 ⊔ 50
GE 12.5 x 25'-0"
GB 8.5
CROSS BRACING
JOINTS
8'-6"
8'-6"
4'-0"
7'-4"
6'-0"
19'-0"
26'-0"
14'-0"
26'-0"
14'-0"
124'-0"
24'-0"
FRAME LAYOUT
1/8" = 1'-0"
WEB BELTING
CAP. 9,000 LB.
W/ SHACKLE AT ONE
END & PELICAN HOOK
AT THE OTHER
PELICAN HOOK
DETAIL "D"
DETAIL "B"
PELICAN HOOKS BOTH SIDES TO
BE FUNISHED WITH BAG
15 C
6 B
25'-0"
SECTION
1" = 1'-0"



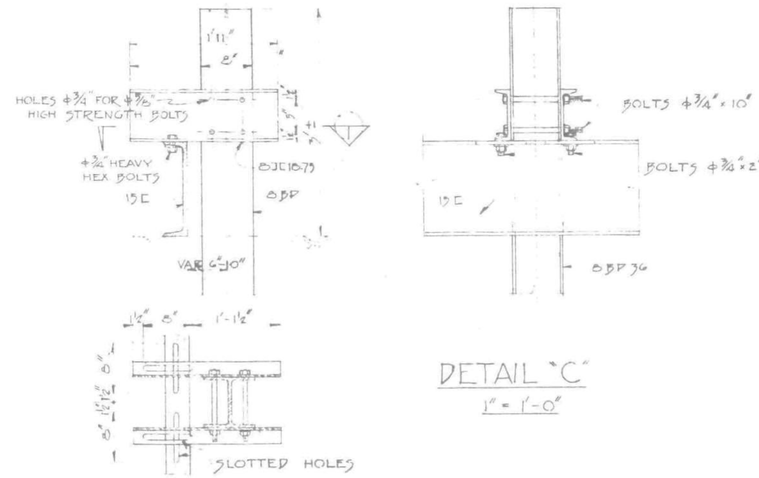
DETAIL "A"

1" = 1'-0"



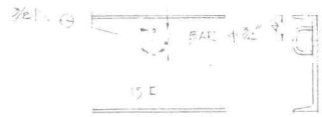
DETAIL "B"

1" = 1'-0"



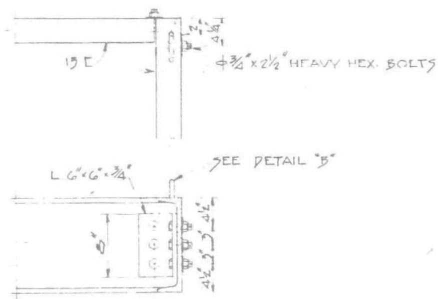
DETAIL "C"

1" = 1'-0"



DETAIL "D"

1" = 1'-0"

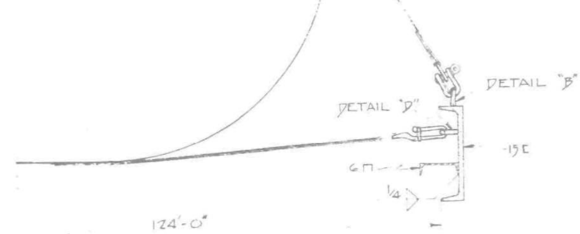


DETAIL "E"

1" = 1'-0"

STRUCTURAL STEEL

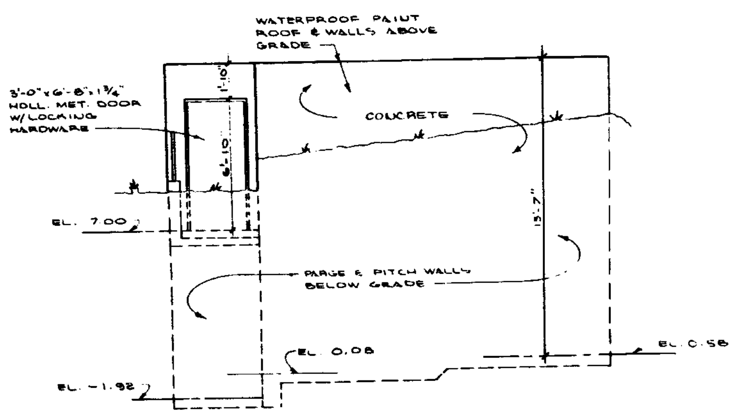
1. STRUCTURAL STEEL SHALL BE DESIGNED, FABRICATED AND ERECTED IN ACCORDANCE WITH SPECIFICATIONS OF THE AISC CODE OF 1963.
2. WELDING SHALL BE IN ACCORDANCE WITH SPECIFICATIONS OF THE AWS.
3. STRUCTURAL SHAPES AND PLATES SHALL BE ASTM A36 UNLESS NOTED OTHERWISE ON PLANS OR DETAILS.
4. FIELD CONNECTIONS SHALL BE MADE USING 3/4" HIGH STRENGTH ASTM A325, UNLESS NOTED OTHERWISE.
5. ALL STRUCTURAL STEEL SHALL RECEIVE ONE SHOP COAT OF RUST INHIBITIVE PAINT EXCEPT ON SURFACES WHERE FIELD CONNECTIONS ARE TO BE MADE. ERECTED STEEL SHALL RECEIVE ONE TOUCH-UP COAT AND ONE FIELD COAT OF RUST INHIBITIVE PAINT.
6. ENTIRE FRAME TO BE ASSEMBLED ON SITE OR ON BARGES BEFORE BEING LOWERED INTO RIVER.



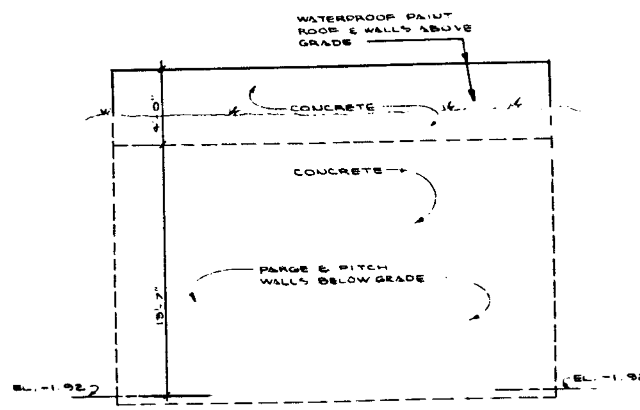
SECTION

1" = 1'-0"

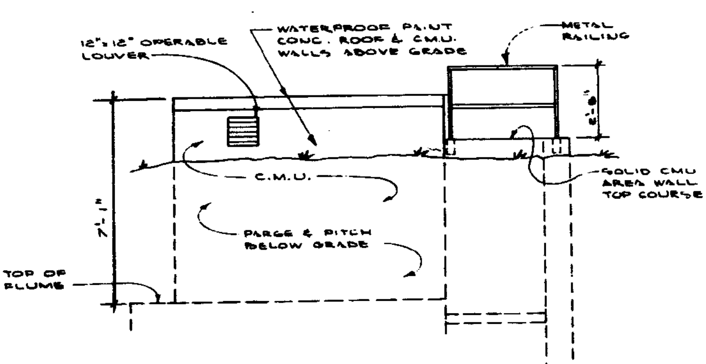
Meter and Pump House Elevations



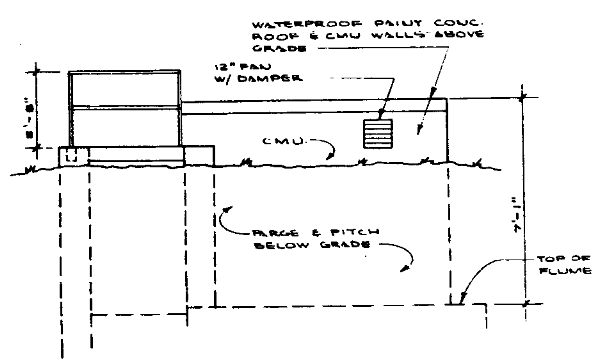
ENTRANCE (FRONT) ELEVATION
SCALE 1/4" = 1'-0"



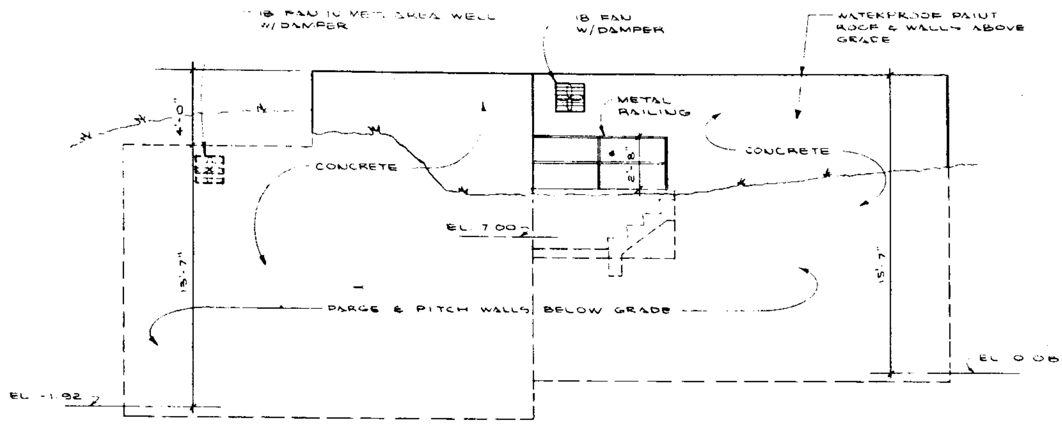
REAR ELEVATION
SCALE: 1/4" = 1'-0"



METER HOUSE - LEFT ELEVATION
SCALE: 3/8" = 1'-0"

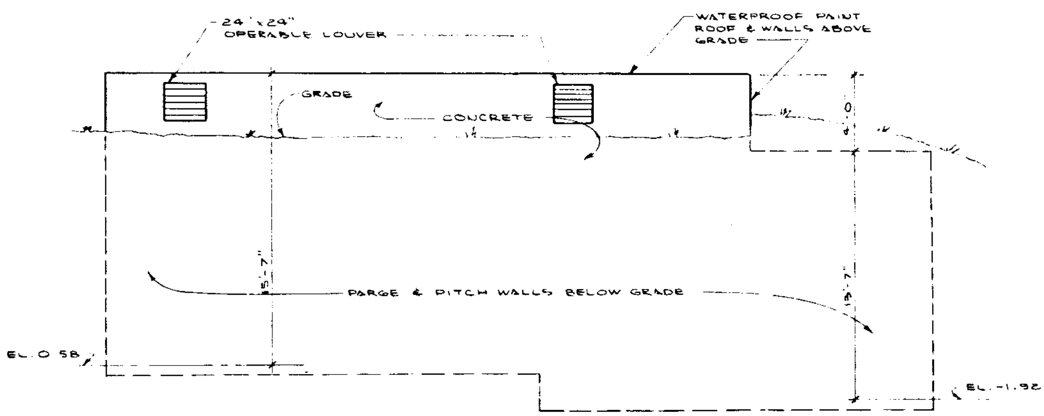


METER HOUSE - RIGHT ELEV.
SCALE: 3/8" = 1'-0"



PUMP & COMMINUTOR ROOM (LEFT) ELEVATION

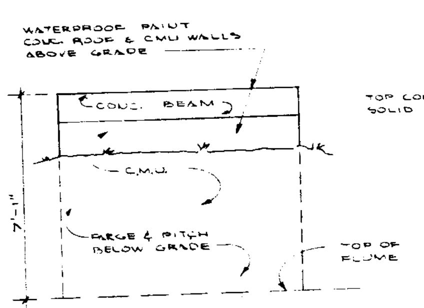
SCALE: 1/4"=1'-0"



RIGHT ELEVATION

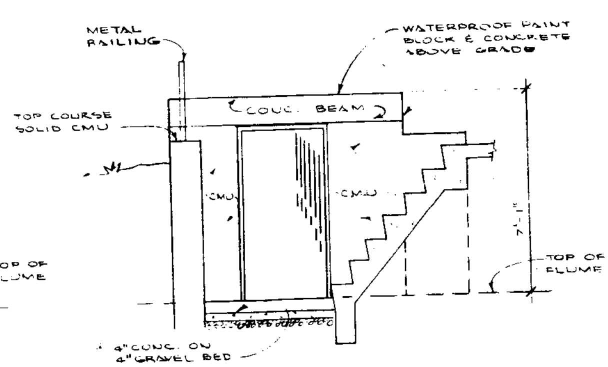
SCALE 1/4"=1'-0"

NOTE:
1. ALL PARGING TO STOP 6" BELOW GRADE ELEVATION



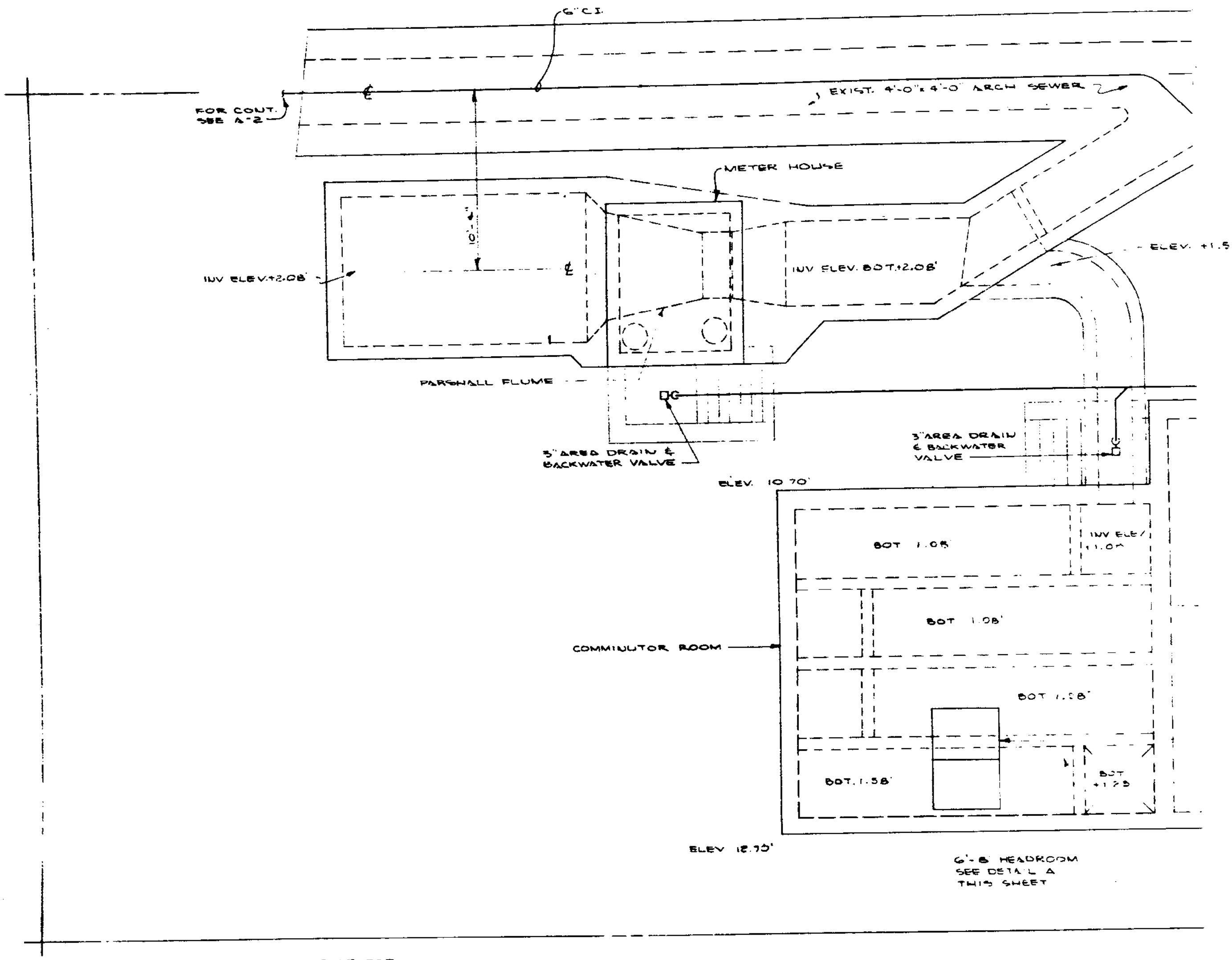
METER HOUSE
REAR ELEV.

SCALE: 3/8"=1'-0"

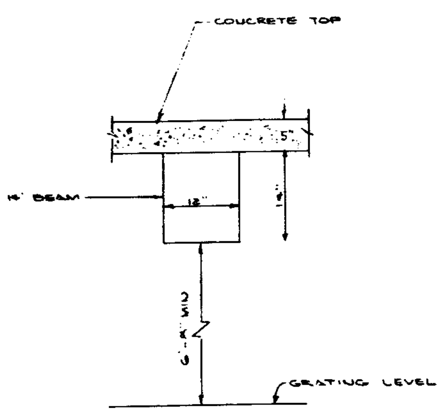


METER HOUSE
FRONT ELEV.

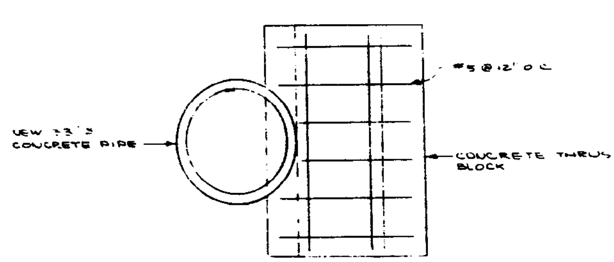
SCALE: 3/8"=1'-0"



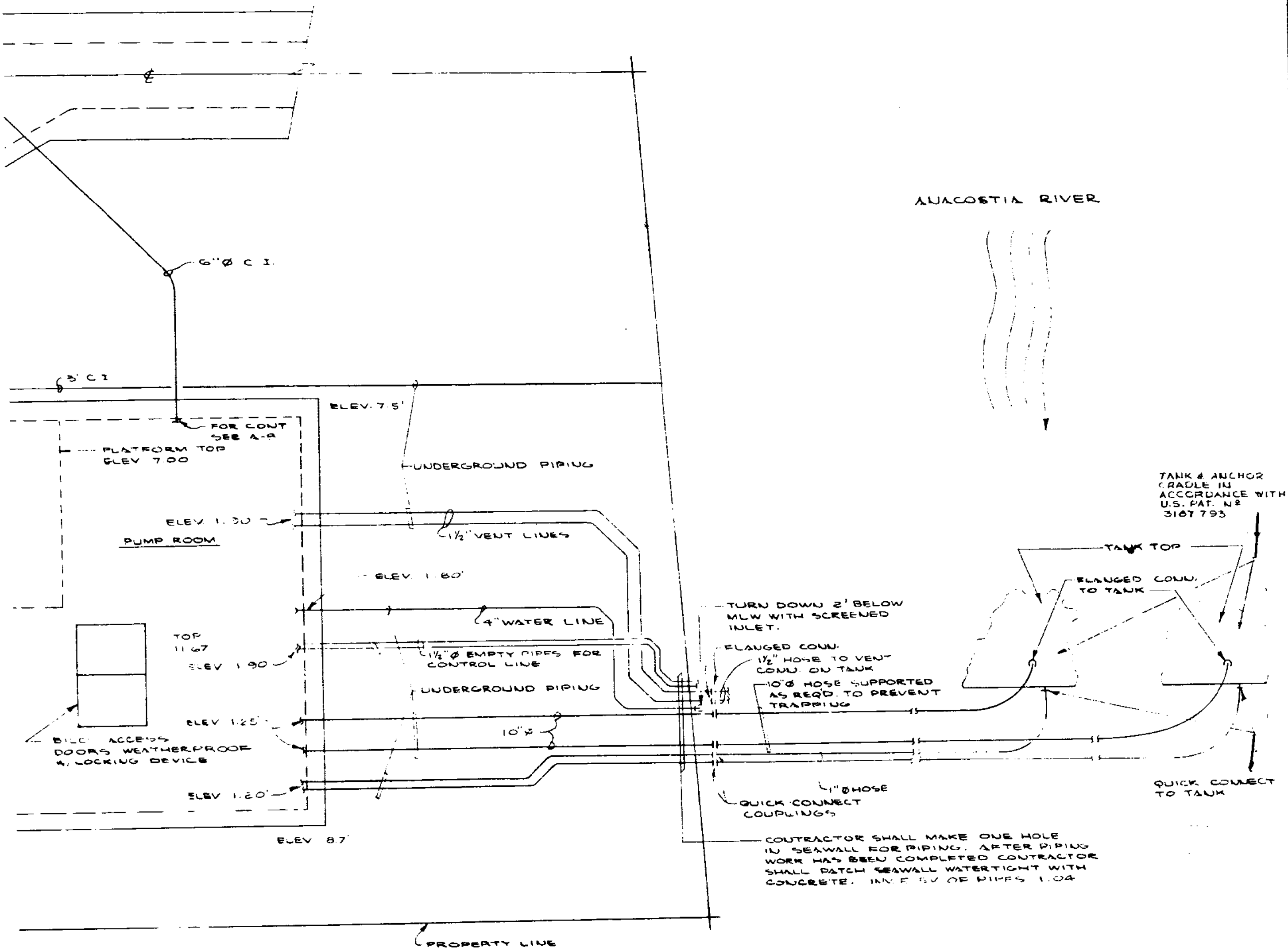
LOCATION PLAN
SCALE 1/4"=1'-0"

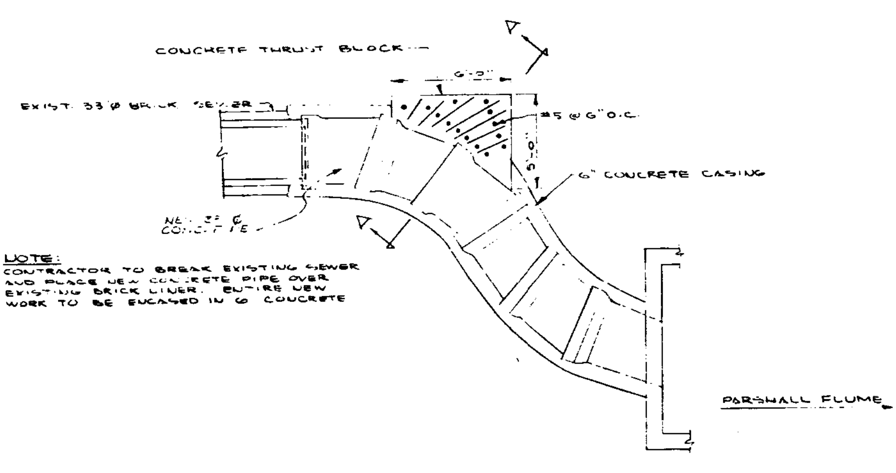


DETAIL A
SCALE 1"=1'-0"

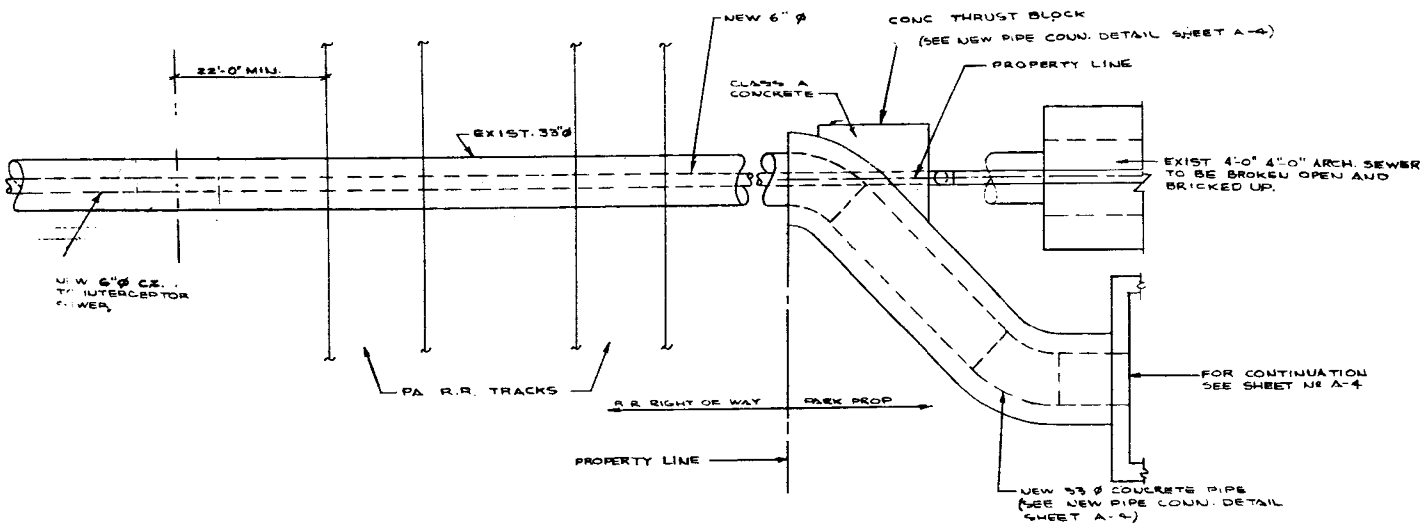


SECTION A-A
SCALE 1/2"=1'-0"

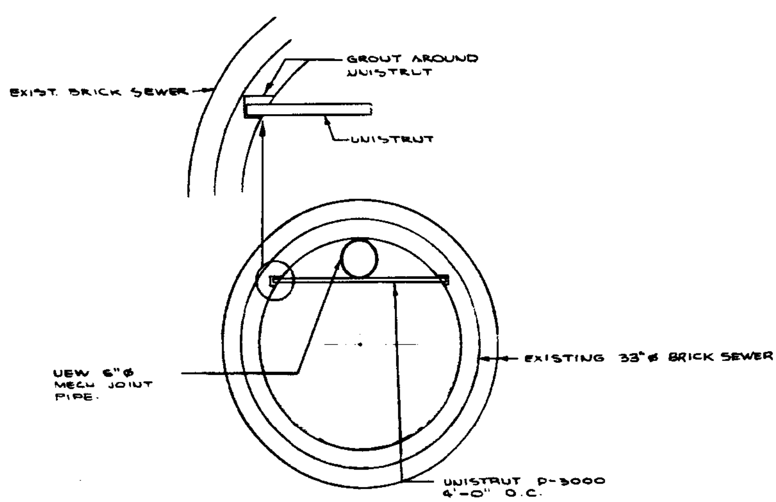




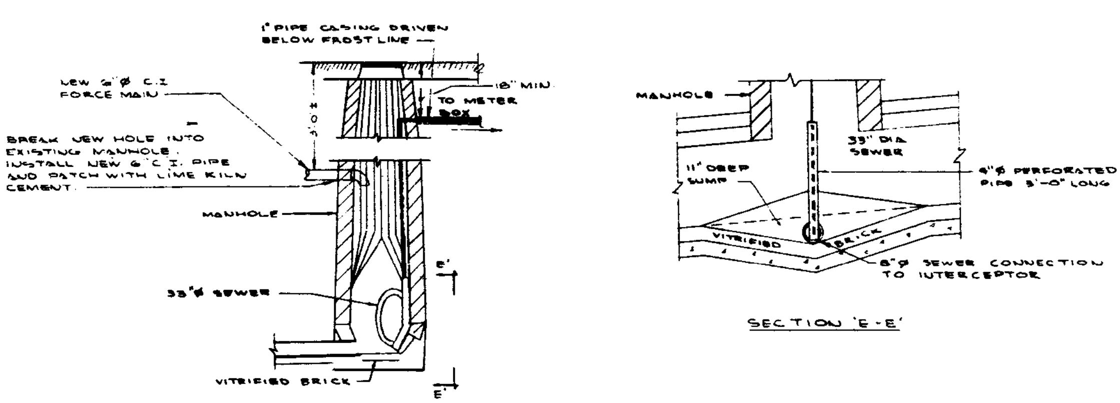
NEW PIPE CONNECTION DETAIL
SCALE: ¼" = 1'-0"



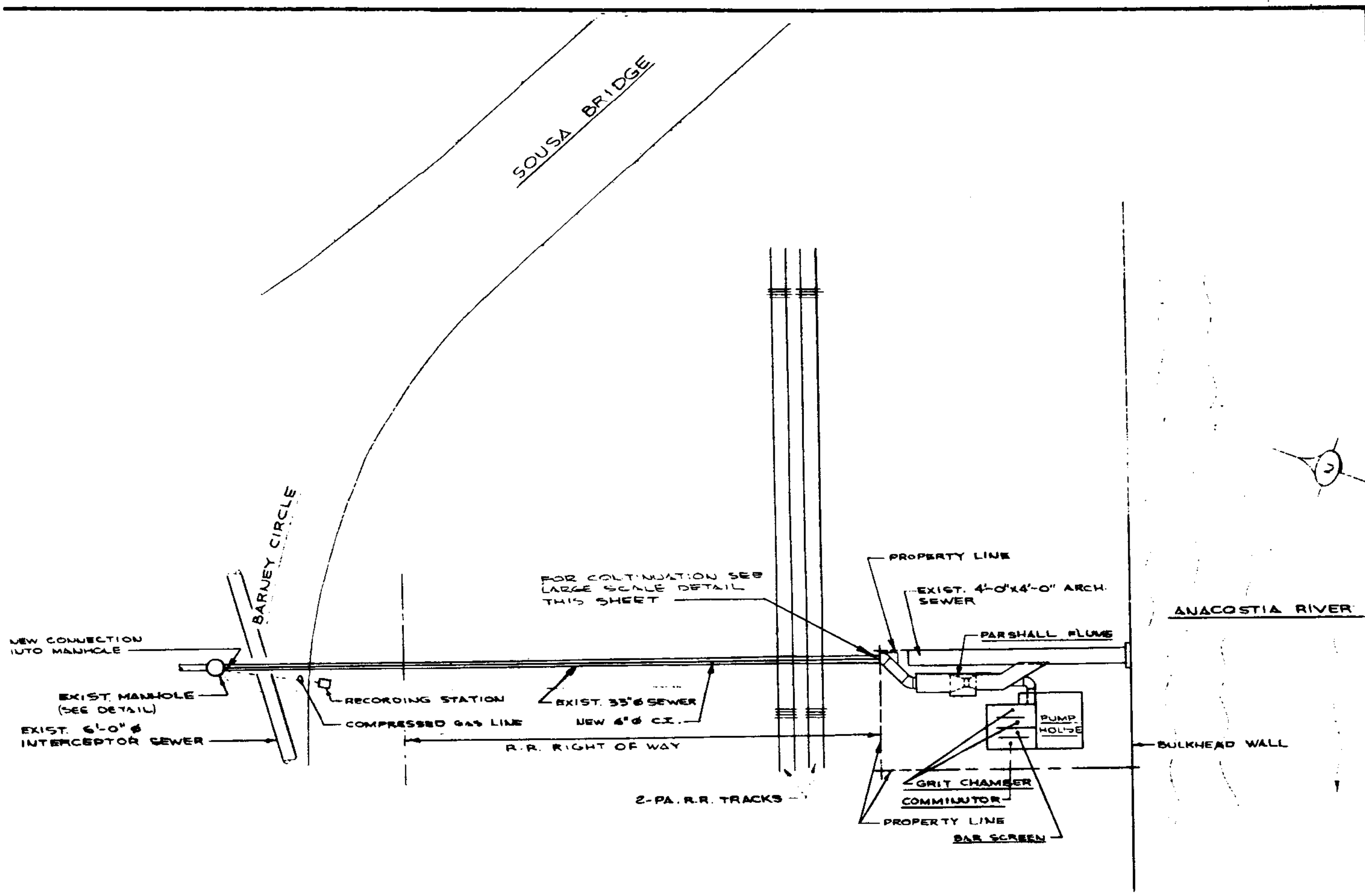
LARGE SCALE DETAIL
SCALE: 1/4" = 1'-0"



SECTION THRU 33" ⌀ SEWER
SCALE 1" = 1'-0"

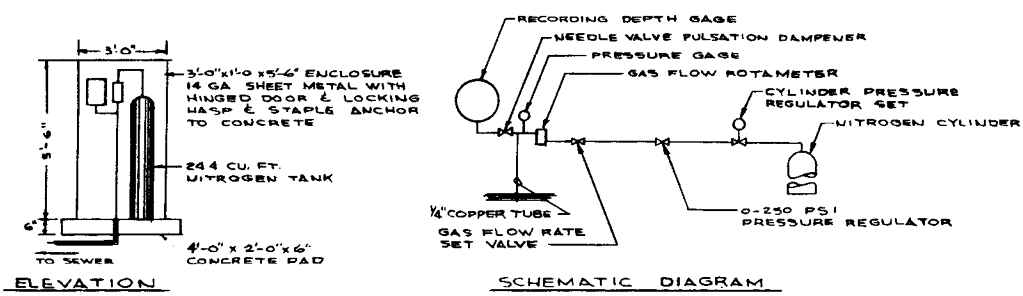


EXISTING MANHOLE DETAIL
NO. SCALE



SITE PLAN

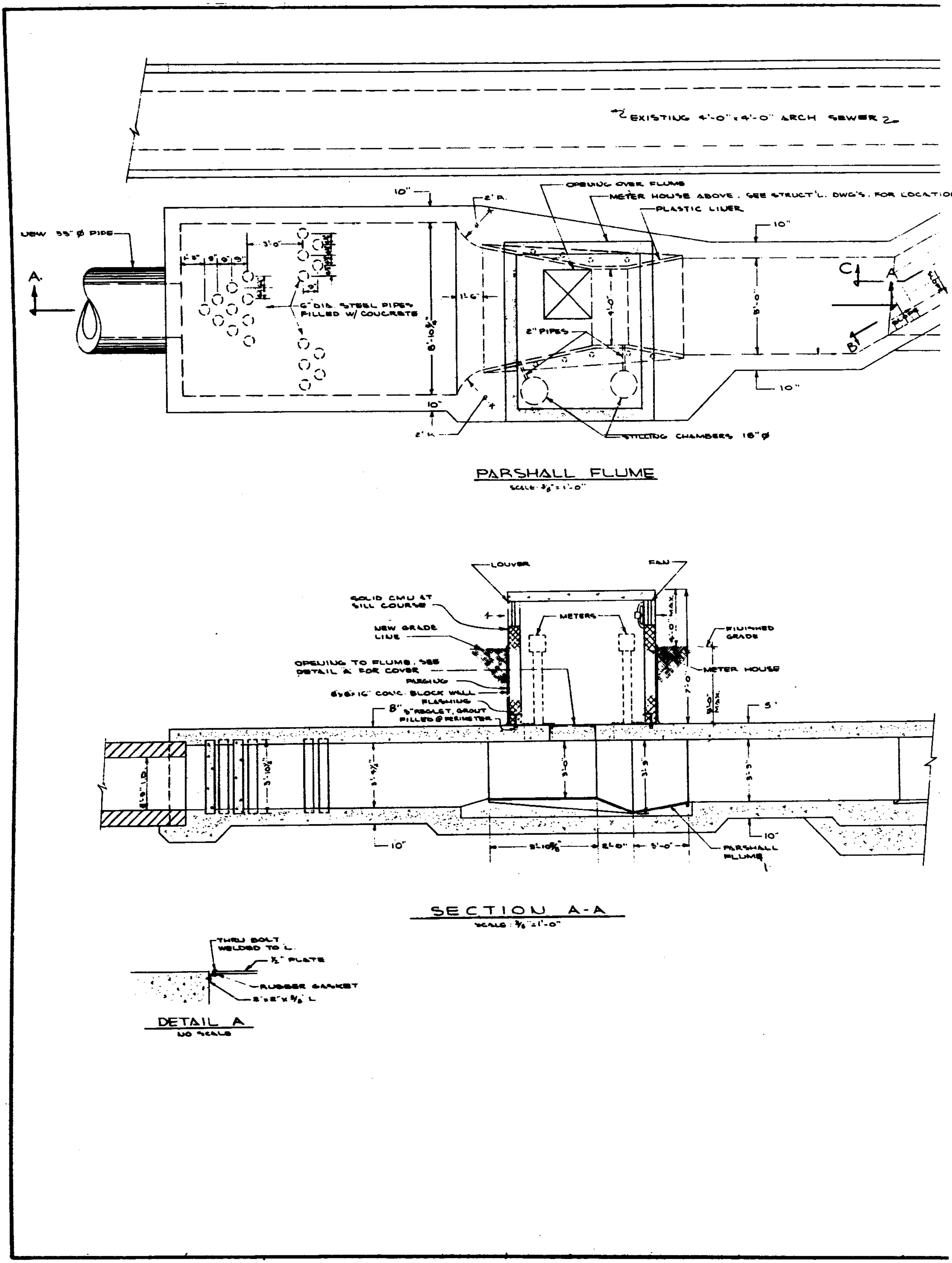
SCALE 1"=30'-0"

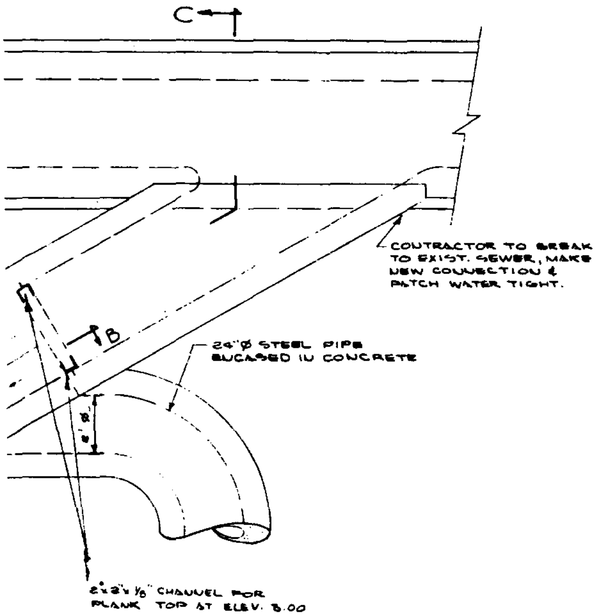


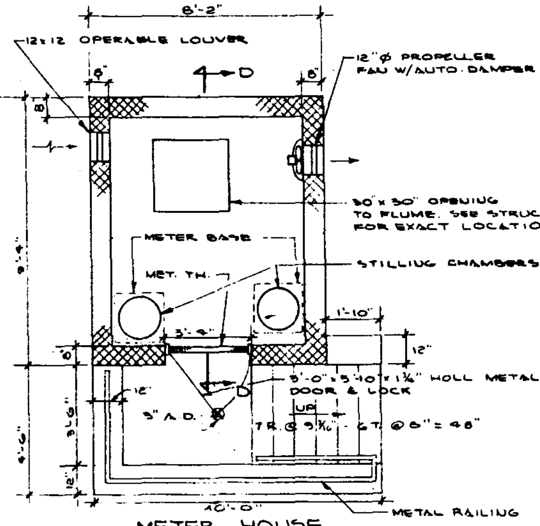
LIQUID LEVEL RECORDING STATION

NO SCALE

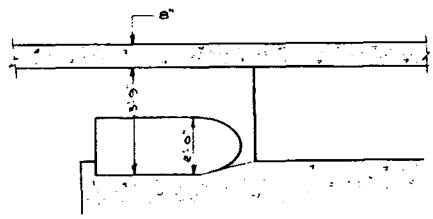
Parshall Flume Plan and Sections



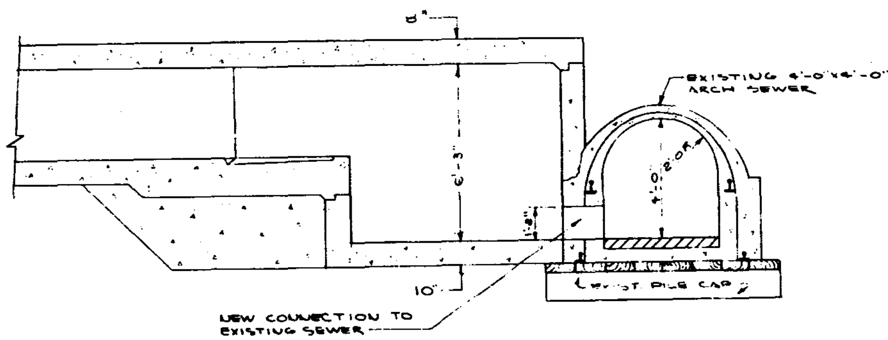




METER HOUSE
SCALE: 3/8"=1'-0"

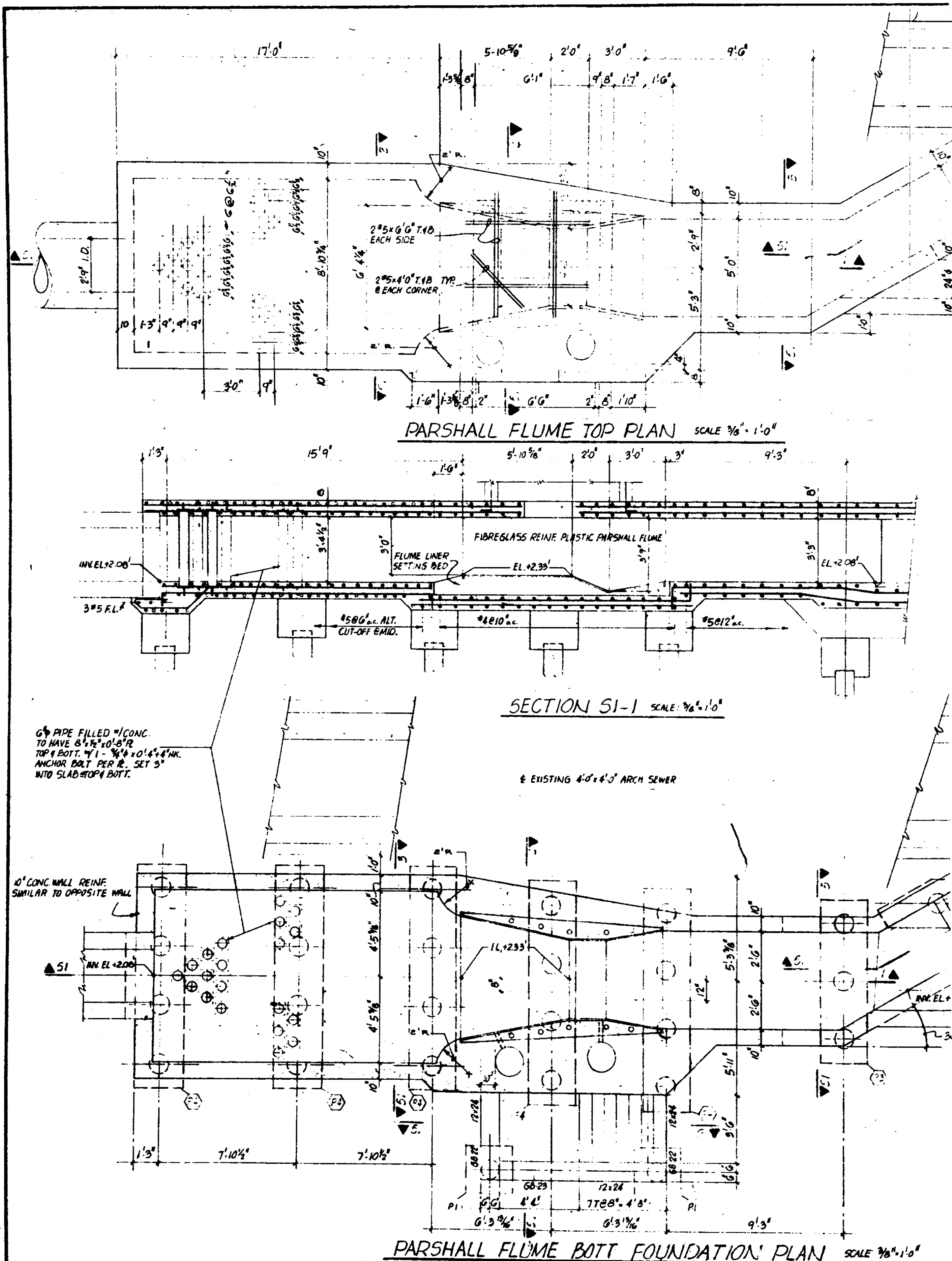


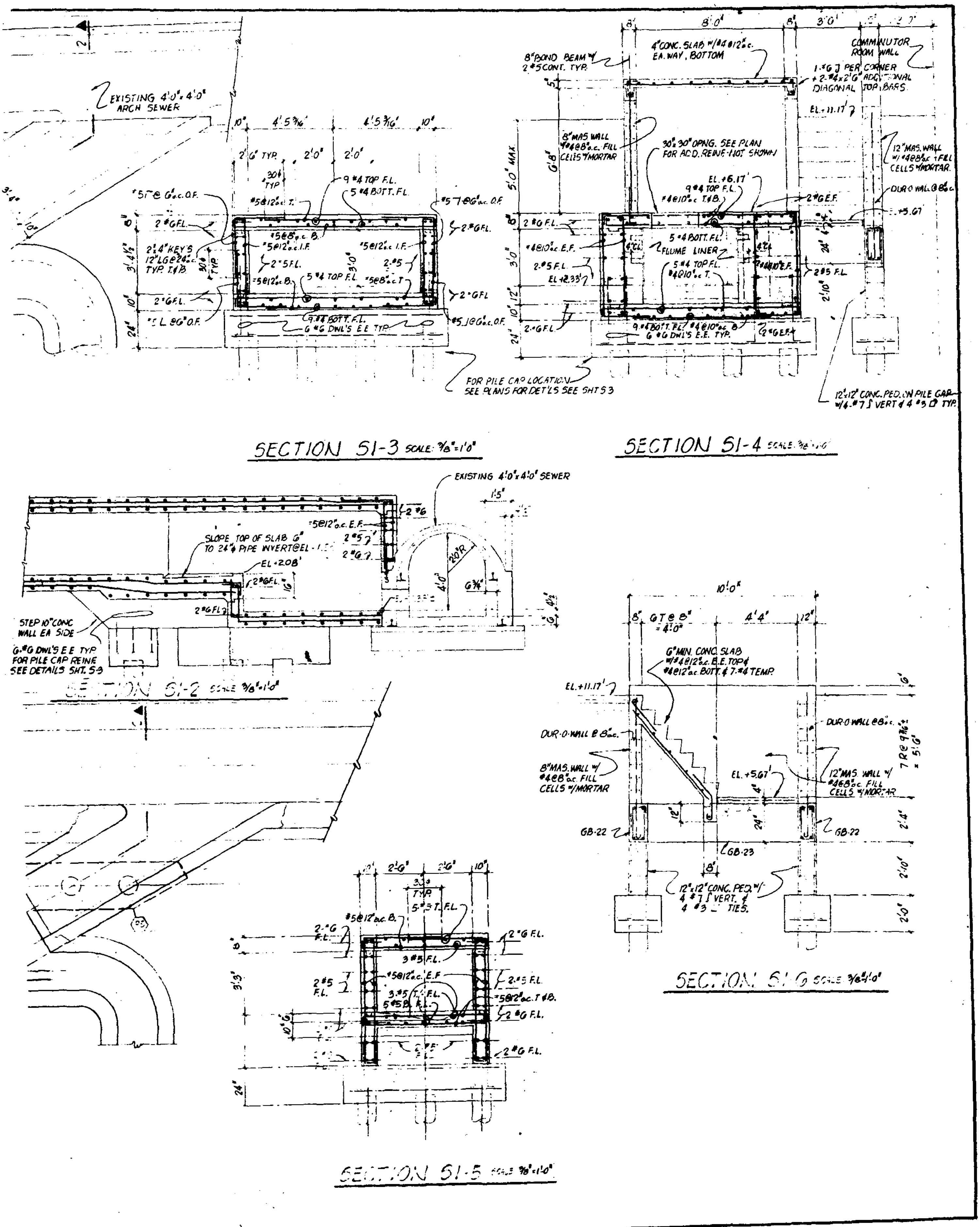
SECTION B-B
SCALE: 3/8"=1'-0"

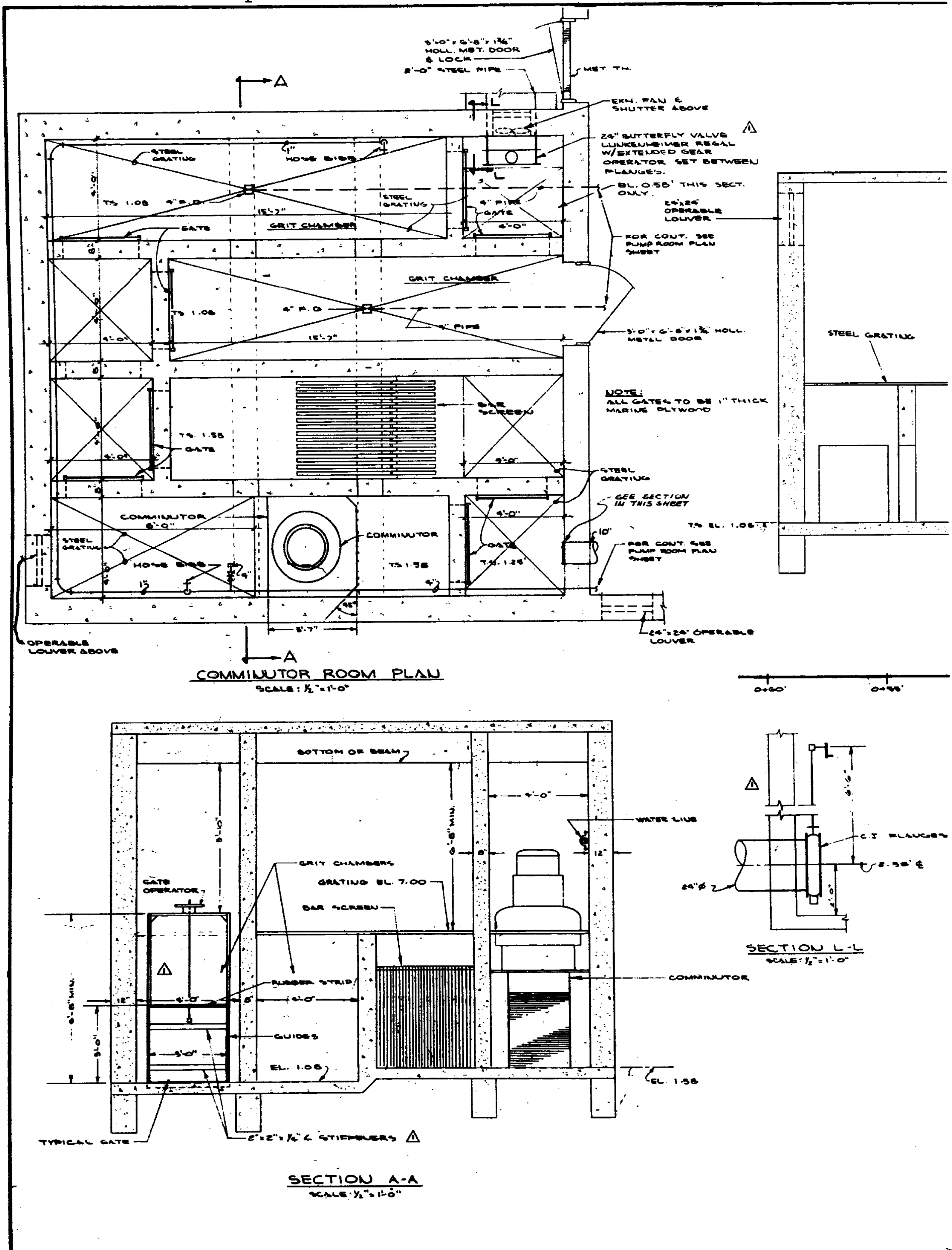


SECTION C-C
SCALE: 3/8"=1'-0"

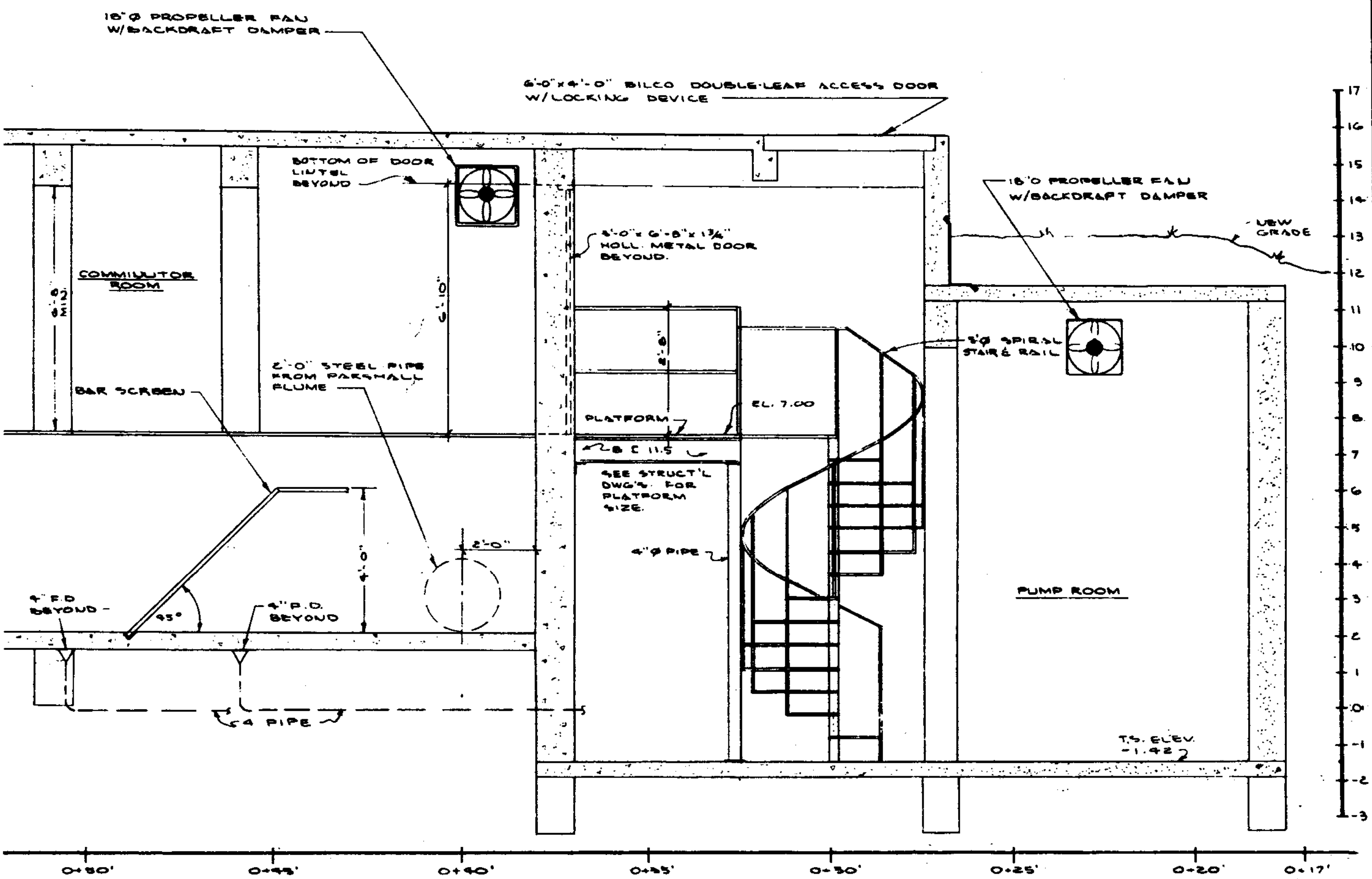
ITEMS TO BE FURNISHED BY OTHERS TO BE INSTALLED BY CONTRACTOR:
1.- PARSHALL FLUME LINER
2.- STILLING WELL METERS
3.- SAMPLING DEVICE
4.- METERING EQUIPMENT


PARSHALL FLUME TOP PLAN SCALE 3/8" = 1'-0"
2 #5 x 6'6" T.&B. EACH SIDE
2 #5 x 4'0" T.&B. TYP. @ EACH CORNER
FIBREGLASS REINF. PLASTIC PARSHALL FLUME
FLUME LINER SETTING BED
INV. EL. +2.08'
EL. +2.33'
EL. +2.08'
SECTION S1-1 SCALE: 3/8" = 1'0"
6" PIPE FILLED W/CONC. TO HAVE 8" x 1/2" x 0'-8" PL. TOP & BOTT. W/ 1 - 3/4" x 0'4" + 4" HK. ANCHOR BOLT PER PL. SET 3" INTO SLAB @ TOP & BOTT.
℄ EXISTING 4'0" x 4'0" ARCH SEWER
10" CONC. WALL REINF. SIMILAR TO OPPOSITE WALL
INV. EL. +2.08
EL. +2.33'
INV. EL. +1.58'
PARSHALL FLUME BOTT FOUNDATION PLAN SCALE 3/8" = 1'0"

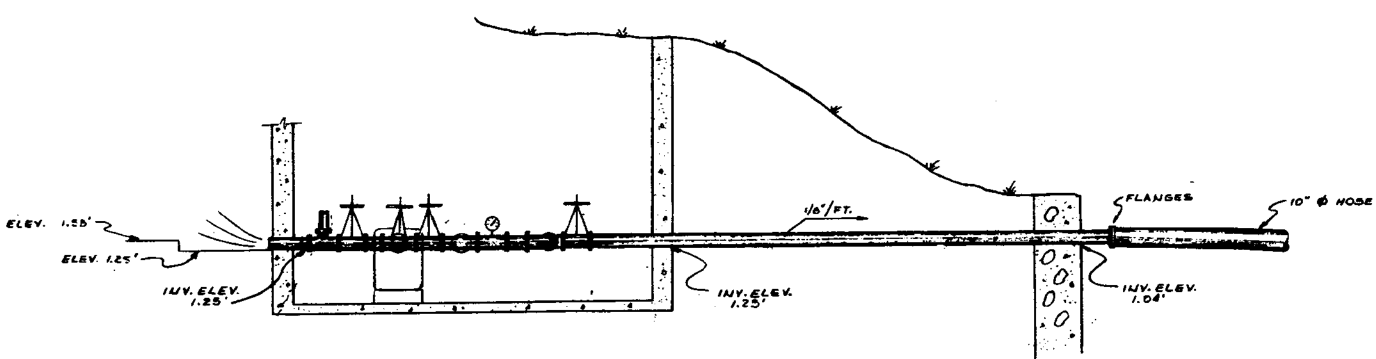

SECTION S1-3 SCALE: 3/8"=1'0"
SECTION S1-4
SECTION S1-2 SCALE 3/8"=1'0"
SECTION S1-5 SCALE 3/8"=1'0"
SECTION S1-6 SCALE 3/8"=1'0"
EXISTING 4'0" x 4'0" ARCH SEWER
EXISTING 4'0" x 4'0" SEWER
FOR PILE CAP LOCATION SEE PLANS FOR DET'LS SEE SHT S3
8" BOND BEAM w/ 2 #5 CONT. TYP.
4" CONC. SLAB w/ #4 @ 12" o.c. EA. WAY, BOTTOM
COMMINUTOR ROOM WALL
8" MAS WALL w/ #4 @ 8" o.c. FILL CELLS w/ MORTAR
30" x 30" OPNG. SEE PLAN FOR ADD. REINF. NOT SHOWN
12" MAS. WALL w/ #4 @ 8" o.c. FILL CELLS w/ MORTAR
FLUME LINER
12" x 12" CONC. PED. ON PILE CAP w/ 4 #7 VERT & 4 #3 TIES TYP.
SLOPE TOP OF SLAB 6" TO 24" PIPE INVERT
STEP 10" CONC WALL EA SIDE
6 #6 DWL'S E.E. TYP FOR PILE CAP REINF. SEE DETAILS SHT. S3
6" MIN. CONC. SLAB w/ #4 @ 12" o.c. E.E. TOP & #4 @ 12" o.c. BOTT. & 7 #4 TEMP.
DUR-O-WALL @ 8" o.c.
8" MAS. WALL w/ #4 @ 8" o.c. FILL CELLS w/ MORTAR
12" MAS. WALL w/ #4 @ 8" o.c. FILL CELLS w/ MORTAR
12" x 12" CONC. PED. w/ 4 #7 VERT. & 4 #3 TIES.
GB-22
GB-23
EL. +11.17'
EL. +5.67'
EL. +6.17'



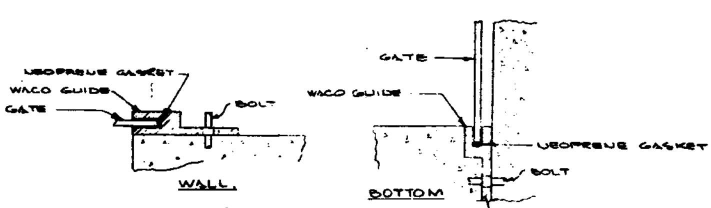
3'-0" x 6'-8" x 1¾" HOLL. MET. DOOR & LOCK
2'-0" STEEL PIPE
MET. TH.
EXH. FAN & SHUTTER ABOVE
24" BUTTERFLY VALVE LUNKENHEIMER REGAL W/EXTENDED GEAR OPERATOR SET BETWEEN FLANGES.
EL. 0.58' THIS SECT. ONLY.
24"x24" OPERABLE LOUVER
FOR CONT. SEE PUMP ROOM PLAN SHEET
STEEL GRATING
HOSE BIBB
GATE
GRIT CHAMBER
4" F.D.
4" PIPE
15'-7"
3'-0" x 6'-8" x 1¾" HOLL. METAL DOOR
NOTE:
ALL GATES TO BE 1" THICK MARINE PLYWOOD
BAR SCREEN
SEE SECTION IN THIS SHEET
COMMINUTOR
OPERABLE LOUVER ABOVE
24"x24" OPERABLE LOUVER
COMMINUTOR ROOM PLAN
SCALE: ½"=1'-0"
BOTTOM OF BEAM
WATER LINE
GRIT CHAMBERS
GRATING EL. 7.00
BAR SCREEN
GATE OPERATOR
RUBBER STRIP
GUIDES
EL. 1.08
EL. 1.58
TYPICAL GATE
2"x2"x¼" ∠ STIFFENERS
SECTION A-A
SCALE: ½"=1'-0"
C.I. FLANGES
24"Ø
SECTION L-L
SCALE: ½"=1'-0"



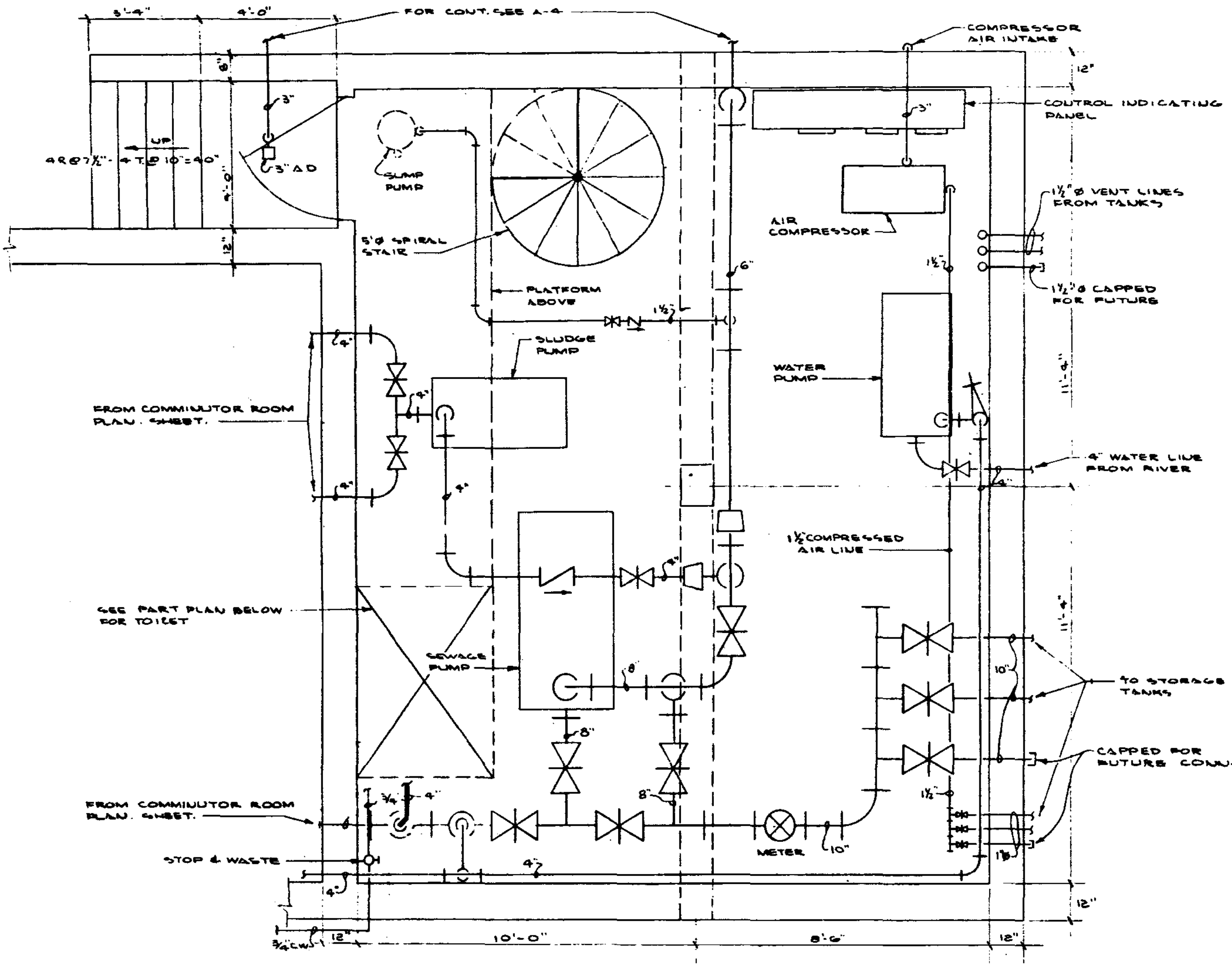
SECTION THRU COMMINUTOR ROOM & PUMP ROOM
SCALE: ½"=1'-0"



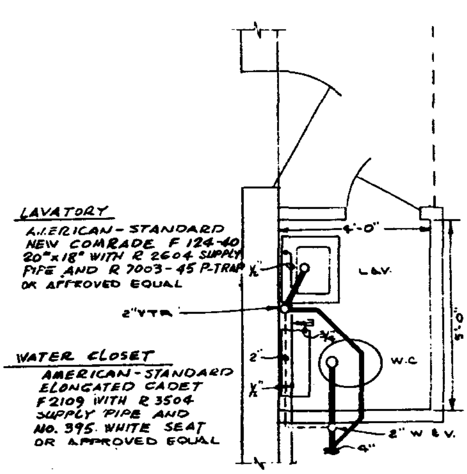
SECTION
SCALE: 1/4"=1'-0"



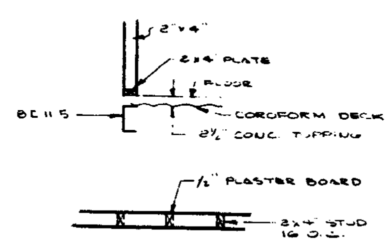
TYPICAL GATE DETAILS
NO SCALE



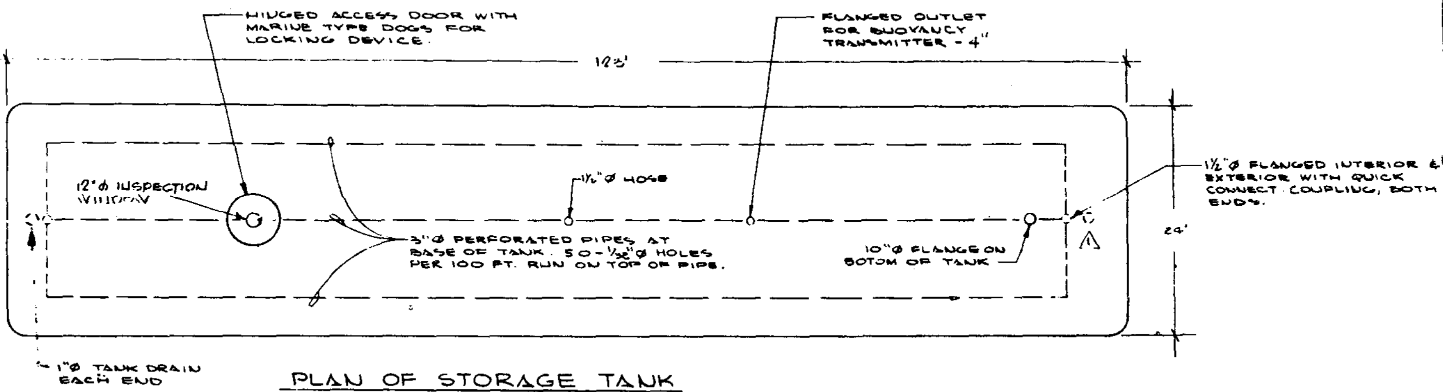
PUMP ROOM PLAN
SCALE: ½" = 1'-0"



PART PLAN TOILET
SCALE ½" = 1'-0"

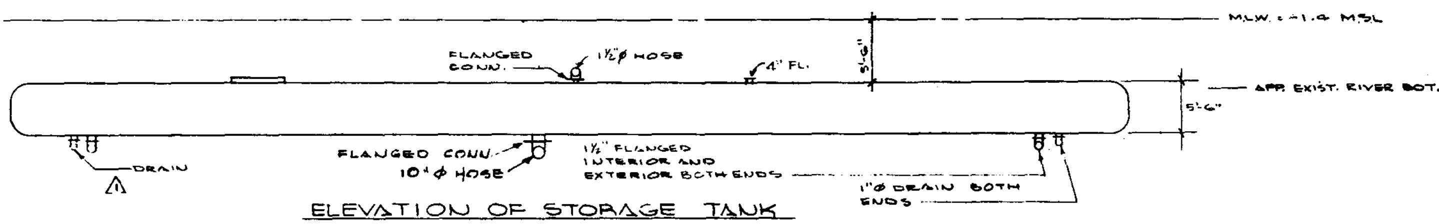


TOILET PARTITION DETAIL
SCALE: ½" = 1'-0"



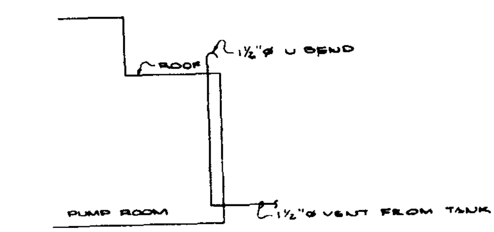
PLAN OF STORAGE TANK

SCALE: ⅛" = 1'-0"



ELEVATION OF STORAGE TANK

SCALE: ⅛" = 1'-0"



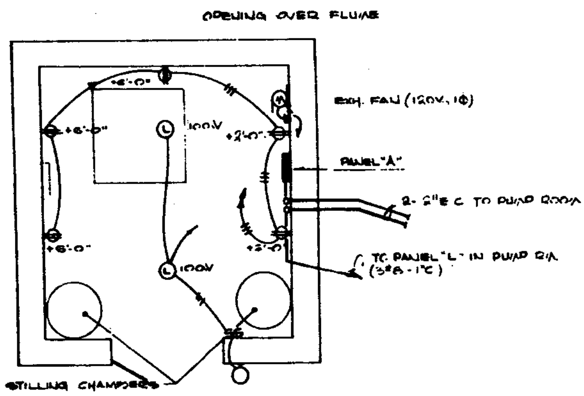
TYPICAL VENT PIPE

NOT TO SCALE

EQUIPMENT SCHEDULE	
SEWAGE PUMP 1000 GPM 70 FT. HEAD 40 H.P. 8" x 6" 1200 RPM SUCT. LIFT 9 NPSH	AIR COMPRESSOR [illegible] CFM - ACTUAL 20 PSI - 25 H.P. 213 CFM PISTON DISPLACEMENT TANK 24" x 6'-0" 19 CU. FT.
SLUDGE PUMP 200 GPM 70 FT HEAD 10 H.P. 2000 RPM 4" x 4"	SUMP PUMP (SUBMERSIBLE) 10 GPM 70 FT HEAD 1 HP. 3,500 RPM
WATER PUMP 200 GPM 30 FT HEAD SUCT. LIFT 5 NPSH 2 HP 1750 RPM	

SYMBOL LIST

	CHECK VALVE
	BUTTERFLY VALVE
	BUTTERFLY VALVE
	METER
	REDUCER OR INCREASER
A.D.	AREA DRAIN
N.I.C.	NOT IN CONTRACT



FLOOR PLAN — METER HOUSE
SCALE: 1/2"=1'-0"

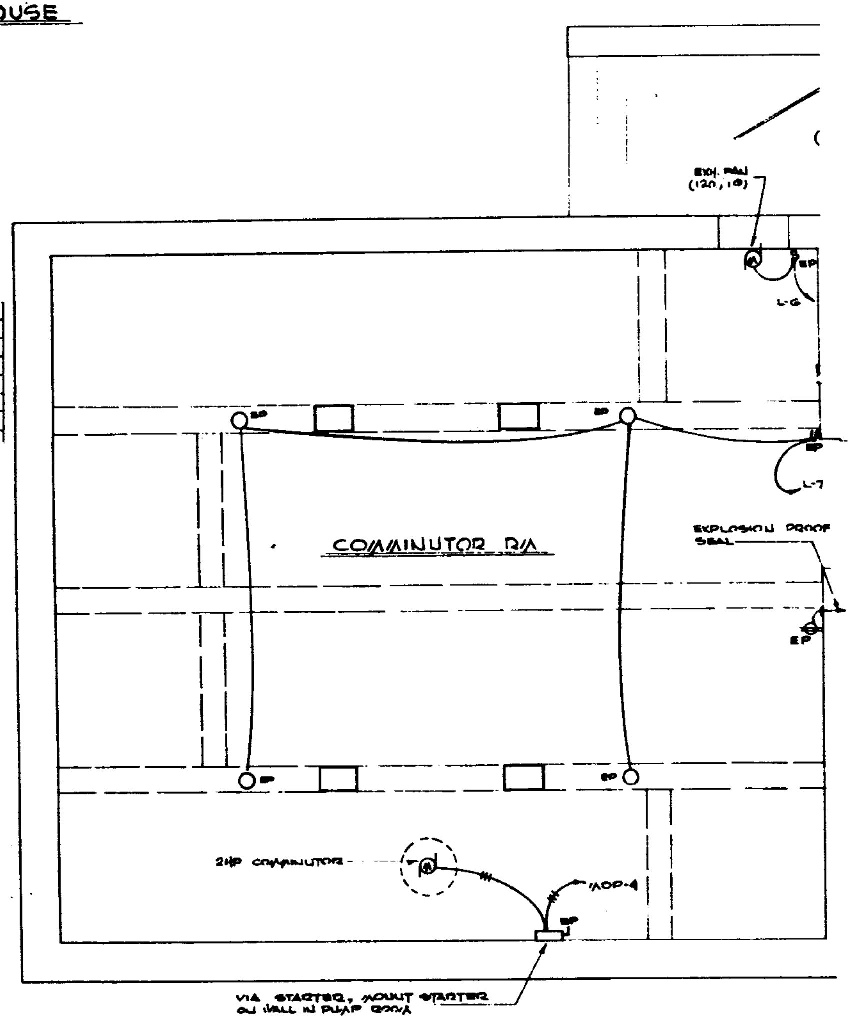
PANEL "MDP" FEDERAL PACIFIC TYPE "CDP"
400A. - 3Φ - 4W, M.L.O.

CKT	SERVING	POLES	FRAME	TRIP	FEEDER
1	40 HP SEWAGE PUMP	3	UFJ	175	3#4/0 - 2"C.
2	10 HP SLUDGE PUMP	3	UEF	50	3#8 - 1"C.
3	25 COMPRESSOR	3	UFJ	125	3#2 - 1 1/4"C.
4	2 HP COMMINUTOR	3	UEF	20	3#12 - 3/4"C.
5	2 HP WTR PUMP	3	UEF	20	3#12 - 3/4"C.
6	PANEL "L"	2	UEF	100	3#2 - 1 1/4"C.

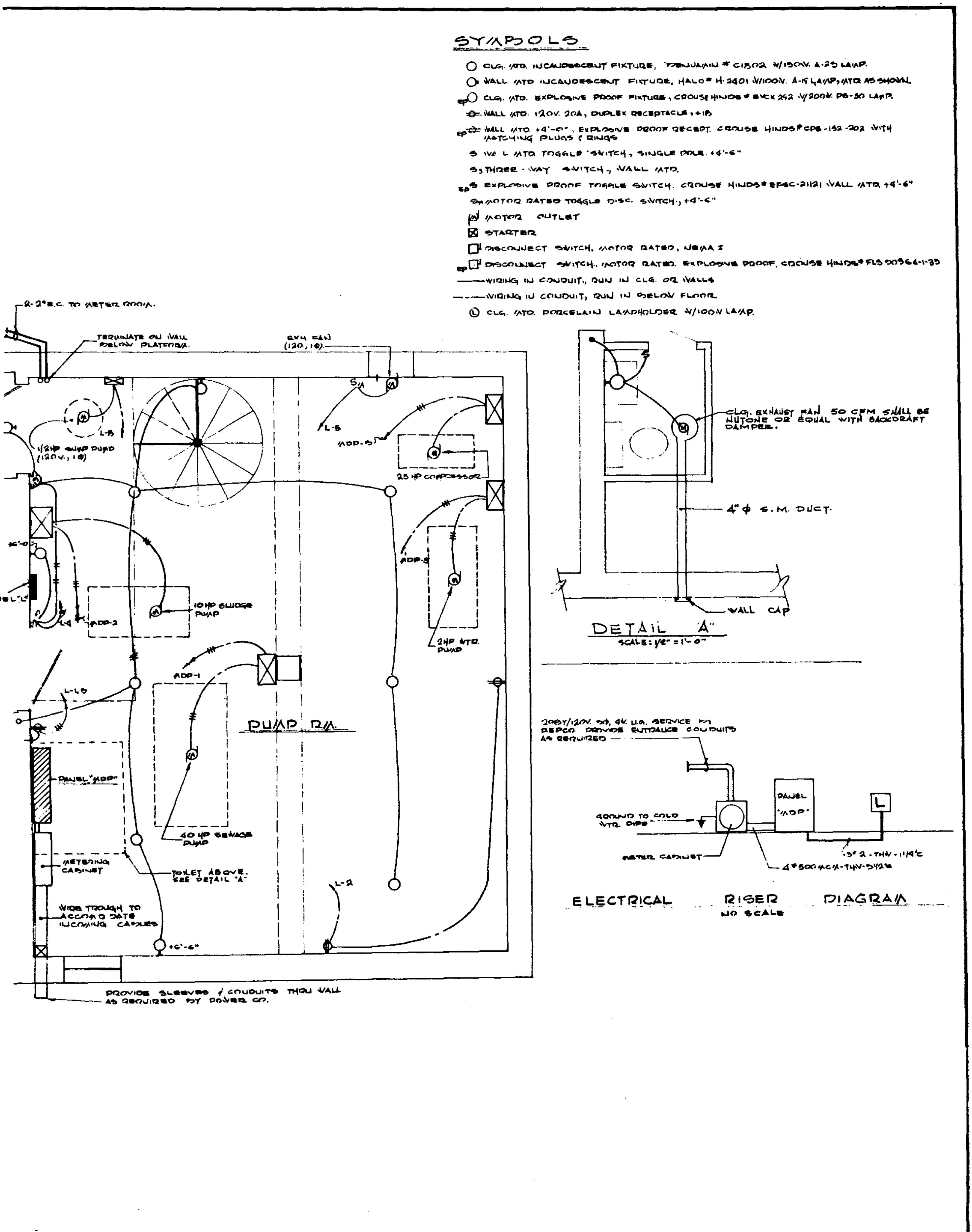
PANEL "L" FEDERAL PACIFIC TYPE "NQLP"
100A, 1Φ, 3W, M.L.O.
11 - 1P - 20A. C.B.'s
1 - 2P - 40A. C.B. (PANEL "A")

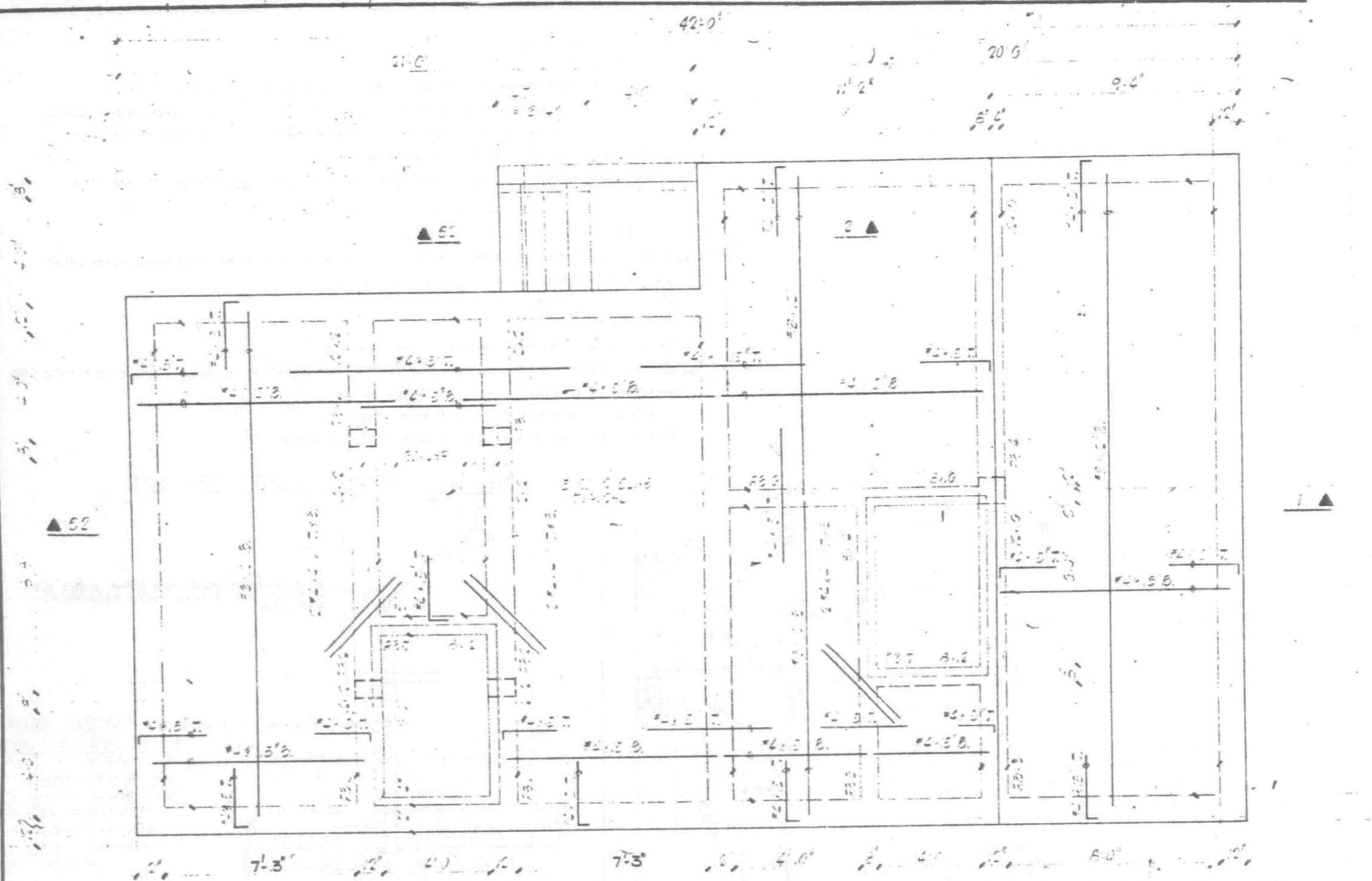
PANEL "A" FEDERAL PACIFIC "STAB LOCK"
100A., 1Φ, 3W., M.L.O.
6 - 1P - 20A C.B.'s

NOTE:
ALL EQUIPT. IN COMMINUTOR ROOM SHALL BE APPROVED FOR CLASS I GROUP "D" OCCUPANCY AND ALL WORK SHALL BE INSTALLED IN ACCORDANCE WITH N.E. CODE FOR THIS TYPE OF THIS TYPE OF OCCUPANCY.

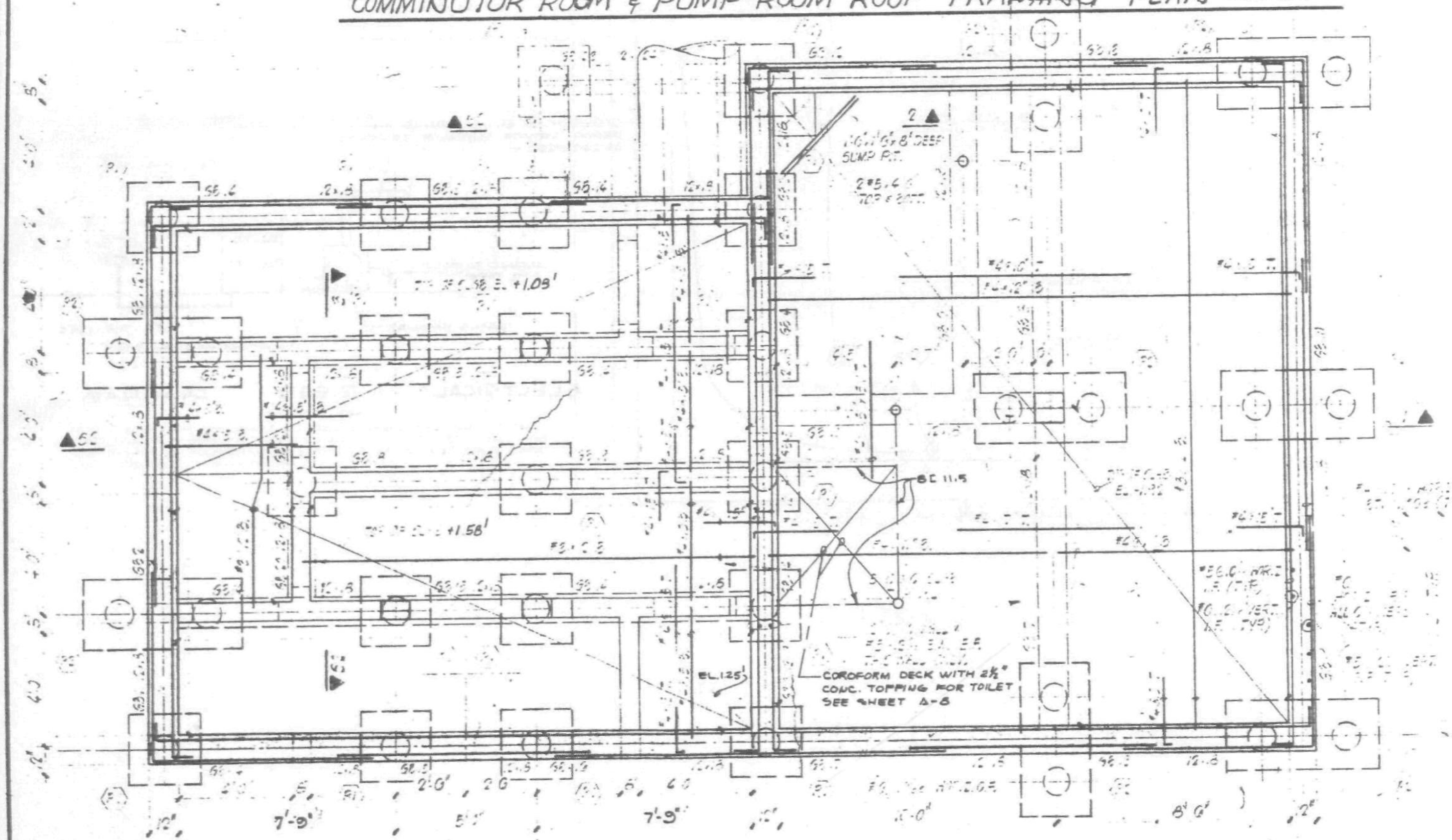


COMMINUTOR & PUMP ROOM PLAN
SCALE: 1/2"=1'-0"

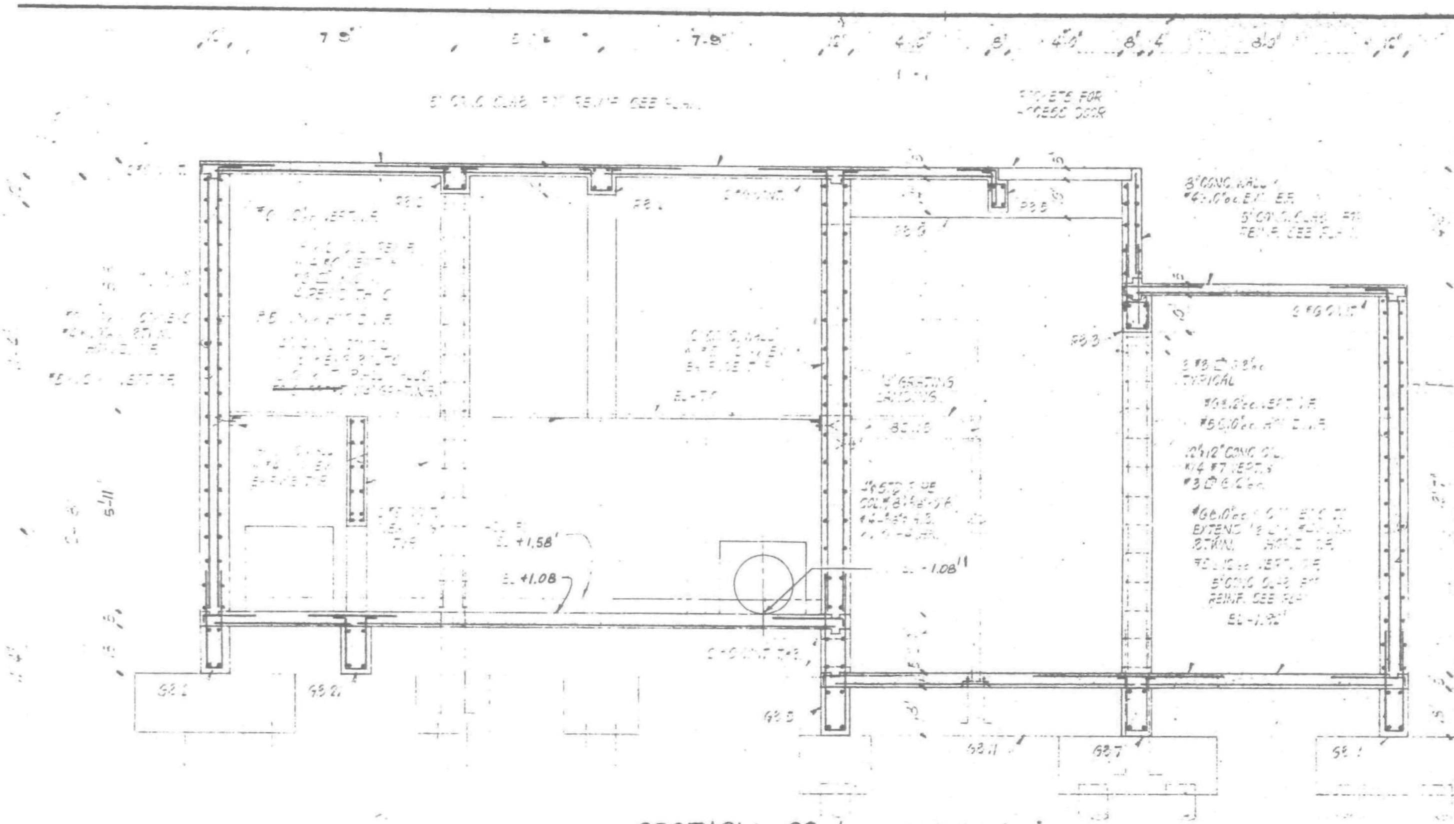

SYMBOLS
CLG. MTD. INCANDESCENT FIXTURE, PENUMAIN # C1802 W/150W. A-25 LAMP.
WALL MTD INCANDESCENT FIXTURE, HALO # H-2401 W/100W. A-19 LAMP, MTD AS SHOWN.
CLG. MTD. EXPLOSIVE PROOF FIXTURE, CROUSE HINDS # EVCX 252 W/200W. PS-30 LAMP.
WALL MTD. 120V. 20A, DUPLEX RECEPTACLE, +18
EP WALL MTD. +4'-0", EXPLOSIVE PROOF RECEPT. CROUSE HINDS # CPS-152-202 WITH MATCHING PLUGS & RINGS
S WALL MTD TOGGLE SWITCH, SINGLE POLE. +4'-6"
S3 THREE-WAY SWITCH, WALL MTD.
EP S EXPLOSIVE PROOF TOGGLE SWITCH. CROUSE HINDS # EFSC-21121 WALL MTD. +4'-6"
SM MOTOR RATED TOGGLE DISC. SWITCH, +4'-6"
MOTOR OUTLET
STARTER
DISCONNECT SWITCH, MOTOR RATED, NEMA 1
EP DISCONNECT SWITCH, MOTOR RATED. EXPLOSIVE PROOF, CROUSE HINDS # FLS 90964-1-35
WIRING IN CONDUIT, RUN IN CLG. OR WALLS
WIRING IN CONDUIT, RUN IN BELOW FLOOR.
CLG. MTD. PORCELAIN LAMPHOLDER W/100W LAMP.
2-2" E.C. TO METER ROOM.
TERMINATE ON WALL BELOW PLATFORM.
EXH. FAN (120, 1Ø)
1/2 HP SUMP PUMP (120V., 1Ø)
L-5
MDP-5
25 HP COMPRESSOR
L-3
+6'-0"
PANEL "L"
10 HP SLUDGE PUMP
L-4
MDP-2
MDP-3
2 HP WTR. PUMP
MDP-1
L-L5
PUMP RM.
PANEL "MDP"
40 HP SEWAGE PUMP
METERING CABINET
TOILET ABOVE. SEE DETAIL "A"
WIDE TROUGH TO ACCOMODATE INCOMING CABLES
L-2
+6'-6"
PROVIDE SLEEVES & CONDUITS THRU WALL AS REQUIRED BY POWER CO.
CLG. EXHAUST FAN 50 CFM SHALL BE NUTONE OR EQUAL WITH BACKDRAFT DAMPER.
4" Ø S.M. DUCT.
WALL CAP
DETAIL "A"
SCALE: 1/8" = 1'-0"
208Y/120V. 3Ø, 4W. U.G. SERVICE BY PEPCO. PROVIDE ENTRANCE CONDUITS AS REQUIRED
GROUND TO COLD WTR. PIPE
METER CABINET
PANEL "MDP"
L
3 # 2 - THW - 1 1/4"C
4 # 500 MCM - THW - 3 1/2"C
ELECTRICAL RISER DIAGRAM
NO SCALE



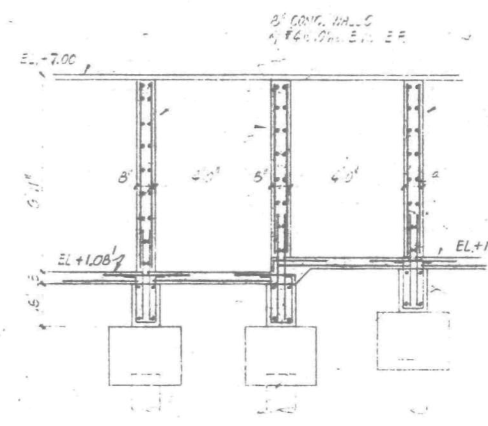
COMMINUTOR ROOM & PUMP ROOM ROOF FRAMING PLAN SCALE: 3/8"=1'-0"



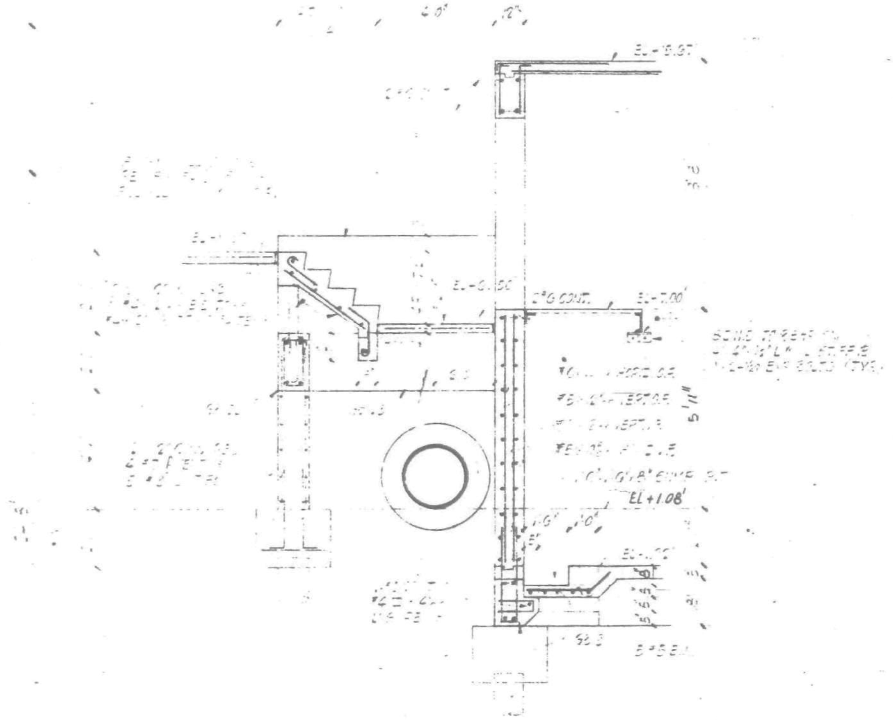
COMMINUTOR ROOM & PUMP ROOM FLOOR & FOUNDATION PLAN SCALE: 3/8"=1'-0"



SECTION S2-1 SCALE 3/8"=1'-0"



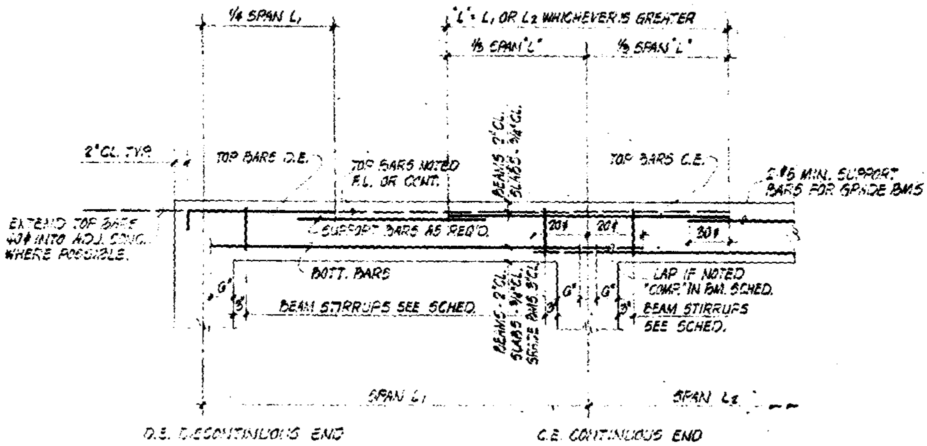
SECTION S2-3



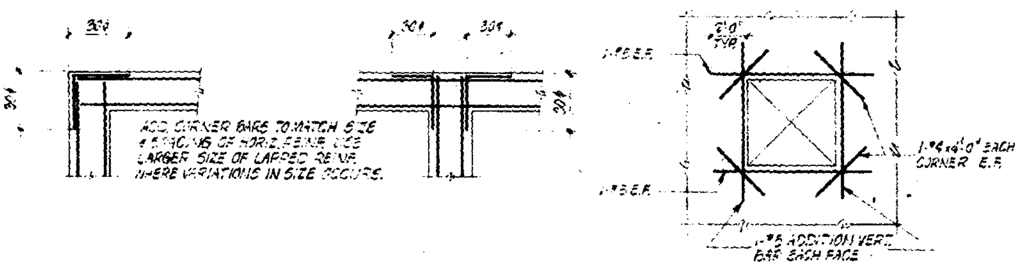
SECTION S2-2

BEAM SCHEDULE

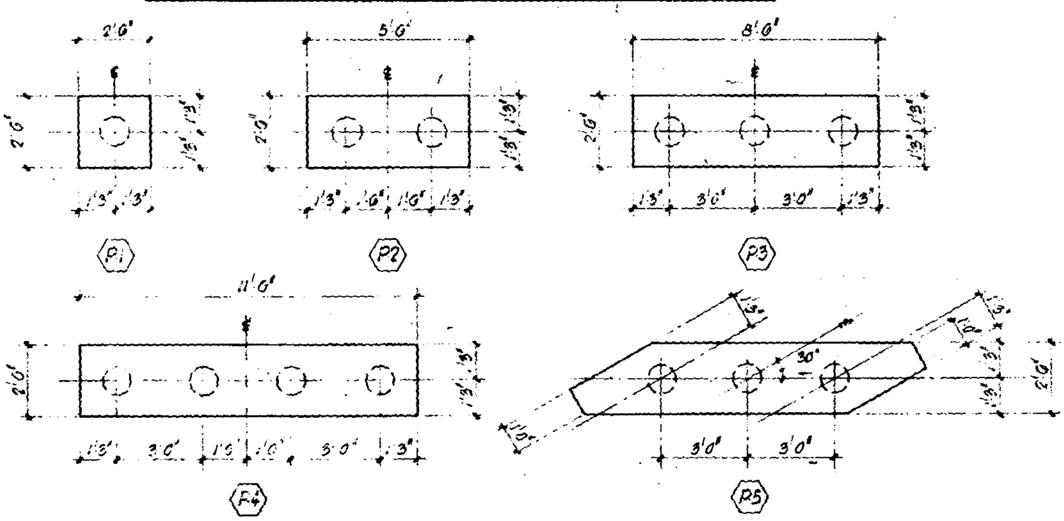
MARK	SIZE		REINFORCING		STIRRUPS			REMARKS
	B	D	TOP	BOTTOM	SIZE	TYPE	SPACING	
RB-1	12	12	2-#5 F.L. EXTEND 1/3"L" INTO RB-2	2-#6 FROM RB-2	#3	Ⴎ	1@3", R@12" E.E.	
RB-2	12	12	E.E. FROM RB-1	2-#6 EXTEND E.E. THRU RB-1		Ⴎ	1@3", R@12" E.E.	
RB-3	12	10	2-#6 D.E. 2-#8 C.E.	2-#8		ƅ	1@3", 2@6", R@12" E.E.	
RB-4	12	16	2-#6 D.E. C.E. FROM RB-3	3-#7		ƅ	1@3", 2@6", R@12" E.E.	
RB-5	8	16	2-#5 E.E.	2-#7		ƅ	1@3", 2@6", R@12" E.E.	
RB-6	8	16	2-#5 E.E.	2-#6		ƅ	1@3", R@12" E.E.	
RB-7	8	12	2-#5 F.L.	2-#5	#3	ƅ	1@3" R@12" E.E.	
GB-1	12	18	2-#8 F.L. EXTEND 1/3"L" INTO GB-2	2-#7 FROM GB-2	#3	U	1@3", 1@6", R@12" E.E.	
GB-2	12	18	E.E. FROM GB-1	2-#7 EXTEND E.E. THRU GB-1			1@3", 1@6", R@12" E.E.	
GB-3	12	18	2-#5 D.E. C.E. FROM GB-4	2-#5			1@3", R@12" E.E.	
GB-4	12	18	2-#6 E.E.	2-#5			1@3", R@12" E.E.	
GB-5	12	18	E.E. FROM GB-4	2-#5		U	1@3", R@12" E.E.	
GB-6	12	18	2-#5 F.L.	2-#7		Ⴎ	1@3", 1@6", R@12" E.E.	
GB-7	12	18	2-#6 D.E. 2-#8 C.E.	2-#7		Ⴎ	1@3", 1@6" R@12" E.E.	
GB-8	12	18	2-#6 D.E. C.E. FROM GB-7	2-#6		Ⴎ	1@3", 1@6", R@12" E.E.	
GB-9	12	18	2-#7 D.E. 2-#9 C.E.	2-#8		U	1@3", 4@6", R@12" E.E.	
GB-10	12	18	2-#7 D.E. C.E. FROM GB-9	2-#8		U	1@3", 4@6", R@12" E.E.	
GB-11	12	18	2-#5 F.L.	2-#7		Ⴎ	1@3", 1@6", R@12" E.E.	
GB-12	12	18	2-#7 D.E. 3-#8 C.E.	3-#8 COMP.		U	1@3", 2@6", R@12" E.E.	
GB-13	12	18	2-#6 D.E. C.E. FROM ADJ. GB.	2-#6 COMP.		U	1@3", 1@6", R@12" E.E.	
GB-14	12	18	2-#7 D.E. C.E. FROM GB-15	2-#6		U OR Ⴎ	1@3", 1@6", R@12" E.E.	
GB-15	12	18	2-#7 F.L. EXTEND 1/3"L" INTO ADJ. GB'S	2-#6		U OR Ⴎ	1@3", 1@6", R@12" E.E.	
GB-16	12	18	FROM ADJ. BM'S	2-#6		U	1@3", 1@6", R@12" E.E.	
GB-17	12	18	2-#8 @ GB-15 2-#7 @ GB-16	2-#6 COMP.		U	1@3", 1@6", R@12" E.E.	
GB-18	12	18	2-#5 D.E. 2-#7 C.E.	2-#5		Ⴎ	1@3", 1@6", R@12" E.E.	
GB-19	12	18	2-#5 D.E. C.E. FROM GB-18	2-#5		Ⴎ	1@3", 1@6", R@12" E.E.	
GB-20	12	18	2-#5 F.L.	2-#5		Ⴎ	1@3", R@12"	
GB-21	12	18	2-#5 F.L.	2-#5		Ⴎ	1@3", R@12"	
GB-22	12	24	2-#5 F.L.	2-#6		Ů	1@3", R@12" E.E.	
GB-23	12	24	2-#6 F.L.	2-#7	#3	Ů	1@3", R@12" E.E.	



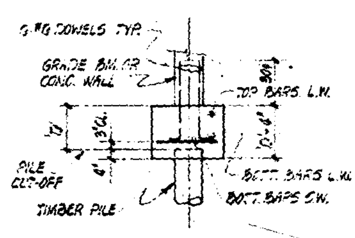
TYPICAL BEAM & ONE WAY SLAB REINF. DETAIL



TYPICAL WALL REINFORCING DETAILS



PILE CAP SCHEDULE					
MARK	"D"	REINFORCING		NO. OF PILES	REMARKS
		TOP	BOTTOM		
P1	20"	NONE	3-#5 L.W. 3-#5 S.W.	1	
P2	20"	NONE	5-#6 L.W. 7-#4 S.W.	2	
P3	20"	6-#6 L.W.	6-#6 L.W. 7-#6 S.W.	3	
P4	20"	6-#6 L.W.	9-#6 L.W. 10-#4 S.W.	4	
P5	20"	6-#6 L.W.	6-#6 L.W. 7-#4 S.W.	3	



TYP. PILE CAP DETAIL

BIBLIOGRAPHIC: Underwater Storage, Inc., Silver, Schwartz, Ltd., Joint Venture. Control of Pollution by Underwater Storage FWPCA Publication DAST 29.

ABSTRACT: A pilot plant was designed, constructed and operated to asses the feasibility of providing a facility for the collection, treatment, storage and final disposition of a portion of the storm overflow from a combined sewer system serving a thirty-acre drainage area in Washington, D.C. A Parshall flume was installed in the overflow line for measurement of flow rates and determination of total overflow volume. A portion of the overflow was diverted to the pilot plant through grit chambers and a comminutor. Flow was stored in two 100,000-gallon underwater bags fabricated of nylon reinforced synthetic rubber and fastened to the river bed by a system of patented anchors. During the period of storage, compressed air was delivered to the tanks for agitation of the solids. Following cessation of the storm, contents of the bags were pumped to the interceptor sewer for delivery to the District of Columbia Sewage Treatment Plant at Blue Plains. Flow into and out of each underwater storage tank was metered and recorded. Samples of the combined sewage overflow discharged to the bags and pumped discharge from the bags were collected and subjected to laboratory analyses. During the operation period from January through September, 1969, a total of 1,600,000-gallons of diverted overflow from 38-storms was stored in the tanks. In addition, 600,000-gallons of river water was pumped into the underwater storage tanks for testing during dry weather periods. The total amount stored was pumped to the interceptor sewer in 26-separate pump out periods. The cost of the pilot plant was $341,480.00, or $1.70 per gallon of storage. This included facilities for testing, samples and flow measurement. Estimates for larger installations, without these special requirements range from 28.2¢ to 14.6¢ per gallon for plants with storage from two to twenty million gallons. The project demonstrated that temporary storage of overflow from combined sewers in underwater rubber storage tanks is feasible and may, under suitable conditions, be effective in eliminating direct, untreated discharge of combined sewage into surface waters during storm periods. Drainage area to be served, land use, nature of storm events, and other factors must be considered when planning an underwater storage facility.

ACCESSION NO:

KEY WORDS:

Storm Overflow

Combined Sewers

Underwater Storage

Hydrology

Pumping Stations

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Pumping Stations

BIBLIOGRAPHIC: Underwater Storage, Inc., Silver, Schwartz, Ltd., Joint Venture. Control of Pollution by Underwater Storage FWPCA Publication DAST 29.

ABSTRACT: A pilot plant was designed, constructed and operated to asses the feasibility of providing a facility for the collection, treatment, storage and final disposition of a portion of the storm overflow from a combined sewer system serving a thirty-acre drainage area in Washington, D.C. A Parshall flume was installed in the overflow line for measurement of flow rates and determination of total overflow volume. A portion of the overflow was diverted to the pilot plant through grit chambers and a comminutor. Flow was stored in two 100,000-gallon underwater bags fabricated of nylon reinforced synthetic rubber and fastened to the river bed by a system of patented anchors. During the period of storage, compressed air was delivered to the tanks for agitation of the solids. Following cessation of the storm, contents of the bags were pumped to the interceptor sewer for delivery to the District of Columbia Sewage Treatment Plant at Blue Plains. Flow into and out of each underwater storage tank was metered and recorded. Samples of the combined sewage overflow discharged to the bags and pumped discharge from the bags were collected and subjected to laboratory analyses. During the operation period from January through September, 1969, a total of 1,600,000-gallons of diverted overflow from 38-storms was stored in the tanks. In addition, 600,000-gallons of river water was pumped into the underwater storage tanks for testing during dry weather periods. The total amount stored was pumped to the interceptor sewer in 26-separate pump out periods. The cost of the pilot plant was $341,480.00, or $1.70 per gallon of storage. This included facilities for testing, samples and flow measurement. Estimates for larger installations, without these special requirements range from 28.2¢ to 14.6¢ per gallon for plants with storage from two to twenty million gallons. The project demonstrated that temporary storage of overflow from combined sewers in underwater rubber storage tanks is feasible and may, under suitable conditions, be effective in eliminating direct, untreated discharge of combined sewage into surface waters during storm periods. Drainage area to be served, land use, nature of storm events, and other factors must be considered when planning an underwater storage facility.

ACCESSION NO:

KEY WORDS:

Storm Overflow

Combined Sewers

Underwater Storage

Hydrology

Pumping Stations

☆ U. S. GOVERNMENT PRINTING OFFICE : 1970 O - 376-902