CORPS OF ENGINEERS, U. S. ARMY

SURGES IN SOUTHERN OUTFALL SEWER AND FLOW CONDITIONS IN STATE FAIRGROUNDS (WESTERN PARKWAY) PUMPING PLANT LOUISVILLE, KENTUCKY

HYDRAULIC MODEL INVESTIGATION

TECHNICAL MEMORANDUM NO. 2-367

CONDUCTED FOR

LOUISVILLE DISTRICT, CORPS OF ENGINEERS

BY

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

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CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DIRECTOR
WATERWAYS EXPERIMENT STATION
VICKSBURG, MISSISSIPPI

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23 JUN 1954

SUBJECT: Transmittal of T. M. 2-367, "Surges in Southern Outfall Sewer and Flow Conditions in State Fairgrounds (Western Parkway) Pumping Plant, Louisville, Kentucky; Hydraulic Model Investigation."

TO: The Chief of Engineers (ENGKW)
Department of the Army
Washington 25, D. C.

T. M. 2-367, "Surges in Southern Outfall Sewer and Flow Conditions in State Fairgrounds (Western Parkway) Pumping Plant, Louisville, Kentucky; Hydraulic Model Investigation," is forwarded herewith for your information and retention.

C. H. DUNN
Colonel, Corps of Engineers
Director

1 Incl (in dup)
T. M. 2-367

PREFACE

Model studies of surges in the Southern Outfall Sewer and of flow conditions in the State Fairgrounds Pumping Plant*, Louisville, Kentucky, were authorized by the Chief of Engineers, Department of the Army, in second indorsement dated 6 February 1947, to a letter from the District Engineer, Louisville District, CE, dated 15 January 1947. Model studies were conducted in the Hydraulics Division of the Waterways Experiment Station during the period December 1947-July 1950. Personnel actively connected with the model study were Messrs. F. R. Brown, T. E. Murphy, T. J. Buntin, and G. B. Sims.

Messrs. C. L. Cowan, J. R. Hamilton, and R. E. Karlen of the Louisville District, and Mr. R. L. Irwin of the Ohio River Division, visited the Waterways Experiment Station to discuss test results and to correlate these results with design work being carried on concurrently in the District Office. Messrs. F. U. Druml and H. B. Willis** of the District and Irwin** did all the analysis work in the application of the model data to the prototype problem. This work is summarized in appendices A and B to this report.

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* Name of plant later changed to Western Parkway Pumping Plant.
** Presently of the Office, Chief of Engineers, and Washington District, respectively.

CONTENTS

CONTENTS (Cont'd)

SUMMARY

Model investigations of the surges occurring in sewer lines and of the flow conditions in a typical pumping plant, used to dispose of drainage during high river stages, were conducted as a part of the local flood-protection project for Louisville, Kentucky. Studies conducted on a 1:32-scale simplified model of the Southern Outfall Sewer provided general information on the relative amount of relief from surges to be expected from the pumping station and manholes in the sewer line upstream. It was determined that the open sump absorbed all water-hammer effects with no increase in pressure occurring upline from the sump. It also was determined that the time of valve closure (pump stoppage) had little effect upon the maximum height of surge rise. A decrease in the size of the sump increased the maximum height of rise; an increase in the area of the risers had little or no effect on the height of rise, although increasing the area greatly lengthened the time of rise. The height of surge upline was reduced by allowing one of the lower manholes to be overtopped and the excess water to go into storage above ground. The time of concentration tests indicated that with a surcharged sewer the time of travel of an additional increment of water from the point of entry to the point of exit was practically instantaneous; whereas, with a sewer flowing partially full, the additional increment traveled with the normal velocity of the water, thus greatly increasing the time of concentration of flow at the lower end of the sewer. The above studies provided data that formed the basis for a comprehensive analysis of the surge problem in sewers.

Tests of the State Fairgrounds Pumping Plant were made on a 1:16-scale model and provided information on plant performance for gravity and pumped flow. Flow conditions were improved throughout the plant by revising the intake and outlet transitions and removing the baffle walls in the discharge channel. It was determined from the model studies that an approximate depth of 2 ft of water was required over the lower end of the inlet transition to prevent the passage of air upstream resulting from surges in the main sump.

SURGES IN SOUTHERN OUTFALL SEWER AND

FLOW CONDITIONS IN STATE FAIRGROUNDS (WESTERN PARKWAY) PUMPING PLANT

LOUISVILLE, KENTUCKY

Hydraulic Model Investigation

PART I: INTRODUCTION

The Problem

1. The city of Louisville, Kentucky, is located on the left bank of the Ohio River, approximately 200 miles northeast of the river's mouth (fig. 1). At present the city's sanitary sewage and the storm water drainage both empty into the Ohio River through a series of combined sewer systems. These systems are of the gravity flow type, and no pumping facilities are provided for forcing the sanitary sewage and storm water into the river during flood stages. Accordingly, it is proposed to build a series of pumping stations near the outlet ends of existing sewer systems or on diversion sewers to pump sewage and runoff across the line of protection into the river during flood stages.

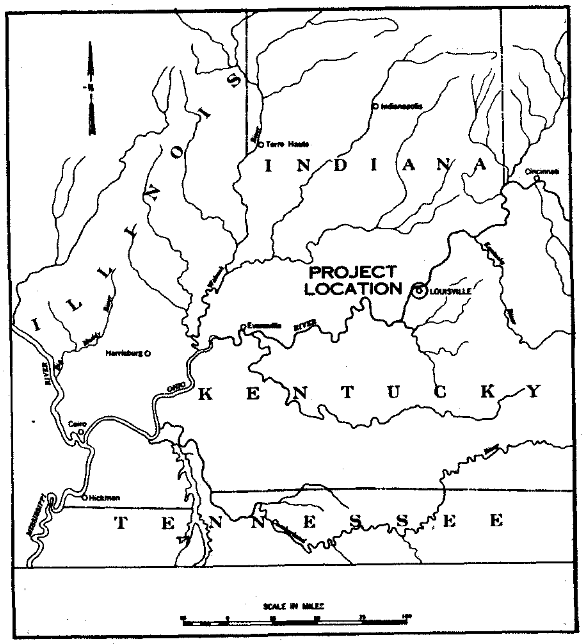

Fig. 1. Vicinity map

2. Preliminary design of the internal drainage facilities of the city of Louisville indicated that, in several instances, pumping stations would be operating with a considerable length of sewer flowing under a surcharge. It appeared that any sudden change in pump discharge, such as might be caused by a power failure, while the station was operating under a surcharge would create surges in the sump which might result in serious damage to the system. This surge problem is apparently analagous to that of a penstock, although the mathematical

solution in connection with sewers is more difficult because of the numerous inlet points and the frequent changes in sewer size. The need for information on surge effects was considered critical enough by the Ohio River Division that studies having application to the Louisville internal drainage problem were initiated. These tests were extended to include the study of flow conditions at the entrance to and through a typical pumping station.

Pertinent Features of the Prototype

3. The Southern Outfall Sewer System* was selected by the Louisville District, CE, as the prototype on which to base the model. This system drains an area of about 4,540 acres, and has a total length of main-line sewer of approximately 33,000 ft (fig. 2) with accompanying laterals and manholes. The main sewer is of concrete construction, varying from a 5-ft-3-in. semielliptical shape to a 15-ft-6-in. horseshoe shape. The surcharged capacity of the sewer is approximately 1700 cfs.

4. A pumping plant (plate 1), designated as the State Fairgrounds Pumping Plant, is to be constructed at the lower end of the Southern Outfall Sewer line for disposal of storm water when the river elevation is higher than that of the sewer. The plant will be constructed of concrete with top of discharge chamber at elevation 462**, and is designed to carry a flow of 1800 cfs when the river is at elev 427.2, and 910 cfs when the river is at elev 458.3. Capacities for model testing were 1800 cfs at river elev 430 and 1200 cfs at river elev 458.3. The average velocity in the sewer during the maximum flow will be approximately 20 ft per sec. The pumping plant was planned to be constructed in an offset position from the existing concrete horseshoe-shaped sewer, which is 10 ft 7 in. by 10 ft 1-1/2 in., and connected to the existing sewer by a transition on each end. The final design provides for "over-sewer" location. The upper transition will be flared so as to decrease the velocity of flow before it enters the sump chamber. The storm-water

* See appendix B for more detailed descriptions of sewer system.
** All elevations are in feet above mean sea level.

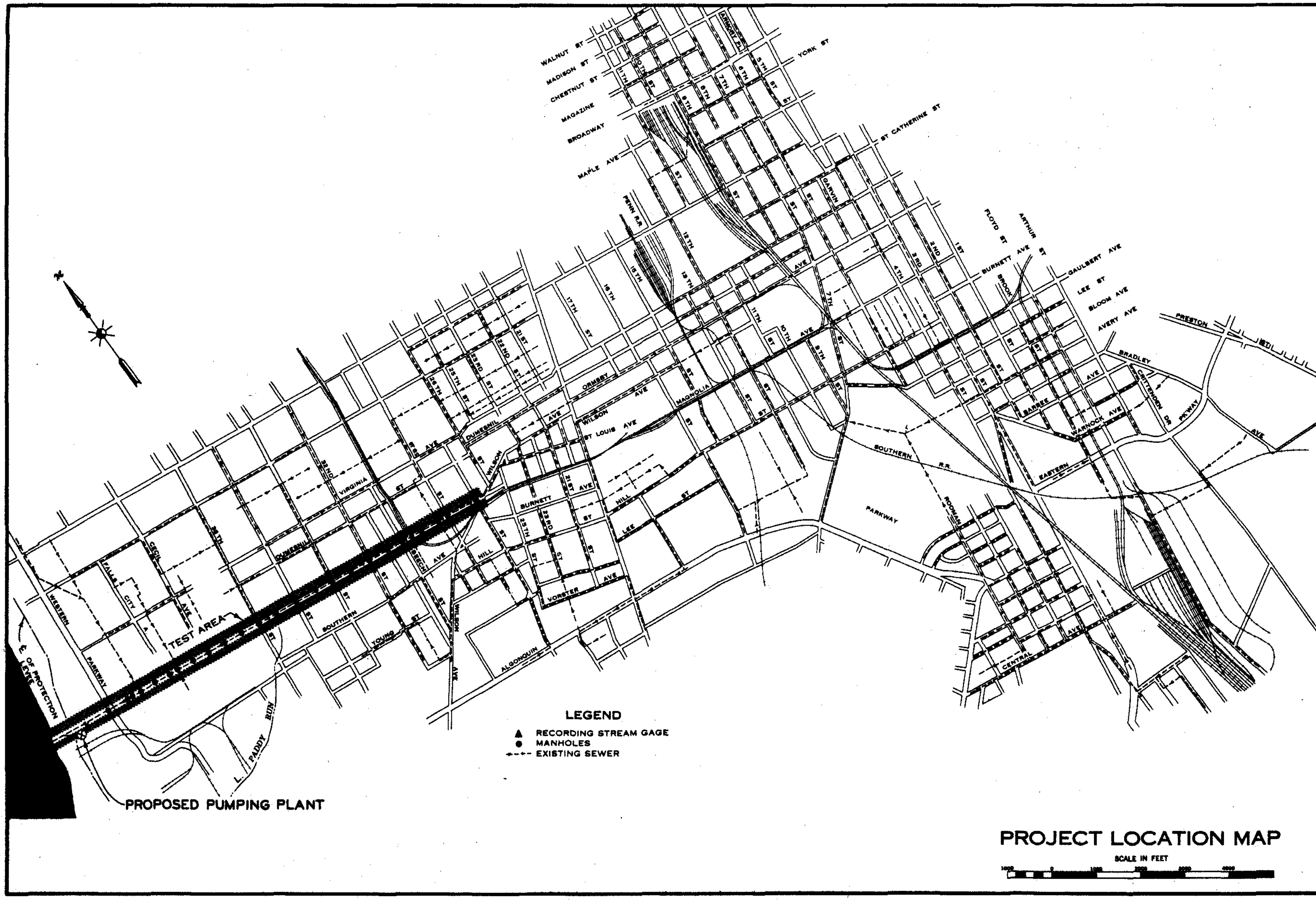
PROJECT LOCATION MAP
SCALE IN FEET
LEGEND
RECORDING STREAM GAGE
MANHOLES
EXISTING SEWER
TEST AREA
PROPOSED PUMPING PLANT
PADDY RUN
SOUTHERN R.R.
PARKWAY
ST CATHERINE ST
BROADWAY
MAGNOLIA
ALGONQUIN
CENTRAL AVE
EASTERN PKWAY
PRESTON ST

Fig. 2.

pumps will be open-pit type installations with each intake opening into a trash rack well. Each trash rack well will be connected to the sump chamber by a gated opening. The pump installation tested consists of four 1500-hp (386 cfs) pumps and two 500-hp (128 cfs) pumps. Final design of this pumping plant provides for four 1250-hp (360 cfs) and three 500-hp (120 cfs) pumps. The river end of the sump chamber will be connected through two gated openings to the outfall sewer, so that the transition section and sump chamber will serve as a part of the gravity outlet of the sewer when the pumping plant is not in operation.

5. It is proposed to maintain the water-surface elevation in the sump chamber within a 5-ft range when the pumps are operating. The pumps will be manually controlled and turned on consecutively, starting with the riverward pump and progressing landward.

Purposes of the Model Studies

6. Two models were considered necessary to study the principal problems involved in designing an efficient and safe sewer system and pumping plant for the conditions existing at Louisville. A simplified model of a part of the main-line sewer is designated in this report as the "surge model" since it was used to determine (a) the water-hammer pressures in the main-line sewer, (b) the surges that might occur at the pumping plant and in the lower reaches of the connecting sewer, and (c) the effects of surcharging the sewer upon the time of concentration of flows at the pumping station. Results of tests from the simplified model could not be applied directly to the Louisville sewer system but were to be used to evaluate the effects of certain variables as a basis for a more comprehensive mathematical development of the design. The second or "pumping plant model" was used to study flow conditions in and through the pumping plant for gravity and pumped flow.

PART II: THE MODELS

Design Considerations

7. Considerable attention was given to the selection of the type models that would allow the desired data to be obtained with minimum expense. After all factors involved had been carefully weighed, it was decided that a simplified model of a portion of the main sewer and the more important laterals would permit collection of data that could be applied analytically to the over-all sewer design. A 1:32 scale was selected for the surge model. Its use permitted the cross-sectional area of the lower part of the main sewer to be simulated by means of a 6-in.-diameter circular pipe; a 4-in.-diameter circular pipe simulated the 10-ft horseshoe-shaped section of sewer in the vicinity of the pumping plant. A 1:16 scale was selected for the pumping plant model in order to permit more detailed study of the flow conditions in the sump and transition areas.

8. Fluid motion in both models was effected predominantly by the force of gravity, which permitted the variation of all hydraulic quantities in their proper relationships as derived from the Froude law. Since it was desired to reproduce water-hammer phenomena in the surge model, and since the velocity of propagation of a pressure wave is a function of the elasticity of the fluid medium and of the conduit walls, elastic forces were also a factor for consideration and Cauchy's number was of primary importance. Therefore, in order to insure closer relationships with regard to elastic forces, the model sewers were constructed of plastic. This material possesses the elasticity required to bring the velocity of pressure-wave propagation about into consonance with the Froudian velocity scale governing discharge in the model.* The use of plastic tubing also aided in approximating the desired roughness values. Relationships for transference of model data to prototype equivalents,

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* For verification see Waterways Experiment Station TM No. 185-1, "Model Study of Hydraulic Characteristics of Power Tunnel, Fort Peck Dam," November 1941.

or vice versa, are listed in the following tabulation where the subscript r represents the model-to-prototype ratio:

Dimension	Ratio	Scale Relationship: Surge Model	Scale Relationship: Pumping Plant
Length	L_r	1:32	1:16
Area	$A_r = L_r^2$	1:1024	1:256
Time	$T_r = L_r^{1/2}$	1:5.66	1:4
Velocity	$V_r = L_r^{1/2}$	1:5.66	1:4
Discharge	$Q_r = L_r^{5/2}$	1:5793	1:1024
Roughness	$n_r = L_r^{1/6}$	1:1.782	1:1.586

Description

9. The 1:32-scale model reproduced approximately 10,000 ft of the lower part of the main line of the Southern Outfall Sewer (fig. 3 and plate 2). No laterals or risers were reproduced initially; however, provision was made for installing a short section of each of five main laterals during the testing program. The entire model was constructed initially of 6-in. plastic tubing (model); after the initial tests the lower 1600 ft was replaced by 4-in. tubing (model). The wall thickness of the 6-in. and 4-in. plastic tubing was 1/8 in. and 1/4 in., respectively. The pumping plant was represented in this model by a rectangular, plastic box equal in area to the sump chamber. The sudden stoppage of flow was produced in the surge model by means of a quick-closing valve on which an electrical measuring device was installed so that the length of time required to effect closure could be measured and recorded. Sections of plastic tubing for reproducing the risers or manholes were provided for installation when required.

10. The 1:16-scale model of the State Fairgrounds Pumping Plant reproduced approximately 175 ft of the existing sewer upline from the pumping plant; the entire pumping plant, including the intake and outlet transitions; and approximately 45 ft of the existing and proposed sewers below

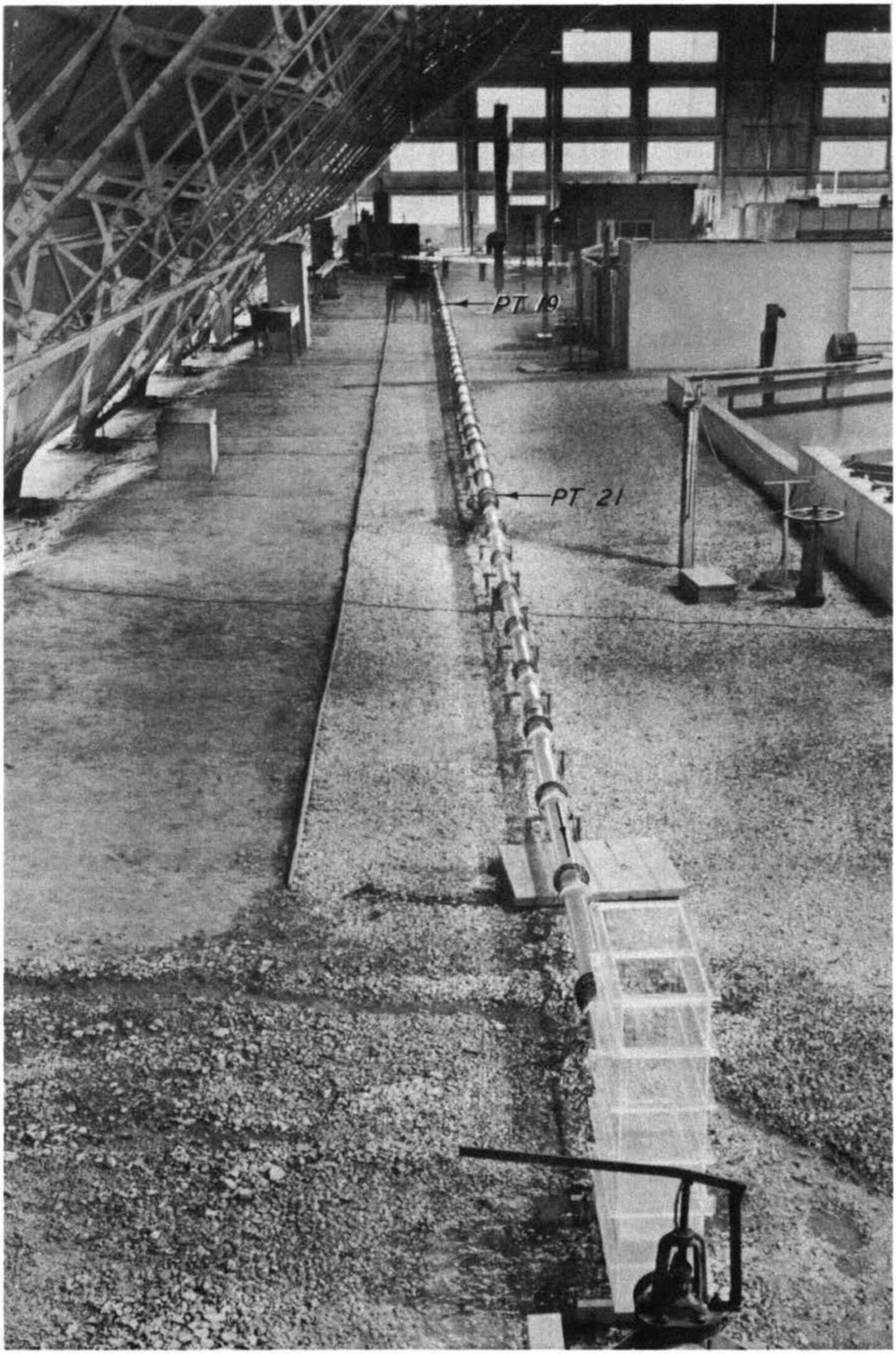

Fig. 3. Model of Southern Outfall Sewer

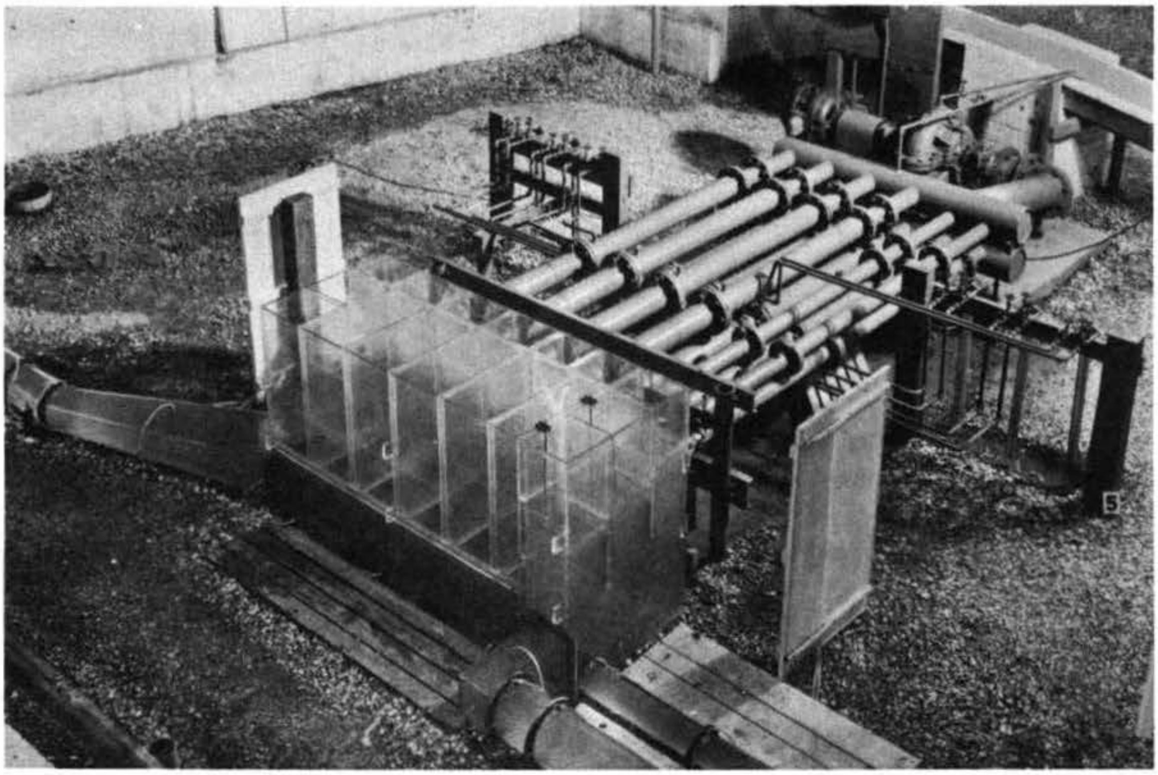

Fig. 4. Model of State Fairgrounds Pumping Plant

the pumping plant (fig. 4 and plate 3). The sewers and pumping plant were constructed of plastic. Pump action was produced in the pumping plant model by six separate outflow and inflow lines, the pumping force being supplied through common headers by means of a 3-cfs centrifugal pump. Each pair of lines was equipped with a circular orifice plate and valve so that any desired combination of discharges could be set up in the system. River stages were simulated initially by means of a valve installed in the discharge line. This model was later revised so as to reproduce the extension of the proposed and existing sewers to the river, and stages were maintained by means of a tailbay into which the sewers emptied (fig. 5).

Appurtenances

11. The inflow into both models was measured by means of venturi

meters. An overflow weir was provided in the headbay of the surge model to carry off the excess water upon sudden stoppage of flow.

12. Static pressures were measured by means of water piezometers and water-surface elevations were read by means of staff gages. Velocities in the sewers were measured by means of a pitot tube mounted in a special collar which permitted attachment of the tube to the pipe at any desired location. Pressures due to water hammer were measured by means of pressure cells connected to an oscillograph recorder (fig. 6). The pressure cells were of the electromagnetic type developed at the Waterways Experiment Station for the measurement of fluid pressures. A continuous photographic record of the pressure fluctuations was obtained; plate 4 shows typical oscillograph records.

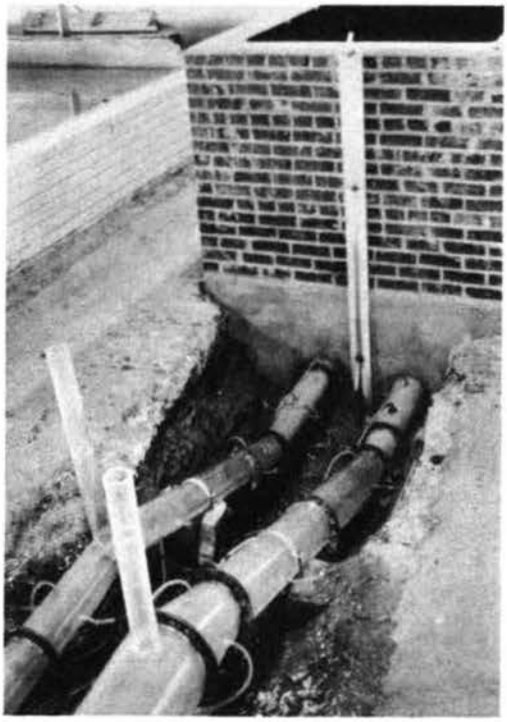

Fig. 5. Tailbay wherein water level could be controlled to simulate river stages

Fig. 6. Recording apparatus

13. Surges were measured by two methods. Initially they were measured by means of resistance gages installed in risers and in the sump and were recorded by means of an oscillograph. In later tests the maximum height and time of rise of the surges were obtained by visual observation and a stop watch, respectively.

PART III: NARRATIVE OF TESTS -- SURGE MODEL

Preliminary Tests of Model Construction Material

14. A preliminary series of tests was conducted to verify the assumptions made during the design of the model to satisfy the requirements of Cauchy's number. These tests involved: (a) determination of the speed of propagation of the pressure wave along the plastic tube with the entire length of sewer constructed of 6-in.-diameter plastic tubing; (b) determination of the coefficient of roughness of the plastic tubing.

Pressure wave propagation

15. The modulus of elasticity of the 1/8-in.-thick plastic material (Lucite) used for the model sewer was determined to be 392,000 lb per sq in. A check of this value after completion of the tests indicated that the modulus remained unchanged. The velocity of propagation of the pressure wave in the prototype was computed to be about 3760 ft per sec; thus the corresponding theoretical pressure wave in the model was approximately 660 ft per sec. The 6-in.-diameter, 1/8-in.-thick Lucite tubing was estimated to be capable of accommodating a velocity of pressure wave propagation of 766 ft per sec, assuming the tubing to be continuous with no joints. However, the model sewer was constructed of 4-ft sections of tubing with a 1-1/2-in. plastic flange on the end of each section. If the entire model had been constructed of plastic with a wall thickness of 1-1/2 in., the pressure wave velocity would have been about 2330 ft per sec. The introduction of a correction factor based upon the number of flanges on the tubing as a proportion of the total length of the model sewer permitted an upward revision of the expected velocity of pressure wave propagation to about 925 ft per sec. This velocity agreed closely with the measured velocity of 1000 ft per sec. Thus the actual value of the velocity of pressure wave propagation (1000 ft per sec) was somewhat in excess of the velocity (660 ft per sec) required for satisfying exact similitude requirements.

16. The model was initially constructed with short stub pipes at each of the riser locations. These stubs were capped, thus providing a

series of branching dead-end pipes. The first tests indicated that the sudden closure of the valve at the pumping plant resulted in a constantly increasing pressure from the valve toward the headbay. This condition was caused by the reflection of pressure waves from the dead ends of the small branching pipes which in turn increased the pressure in the main sewer. Plugs inserted flush with the main sewer in these branch pipes equalized shock pressures along the sewer line.

Coefficient of roughness

17. The coefficient of roughness of the plastic model sewer was determined from Manning's formula and is presented on plate 5. The computed average velocities based on the flow through the sewer were used in the computation of roughness values. The slope of the hydraulic gradient was determined from piezometers located along the model sewer.

Water-hammer Pressures in Sewer

18. Initial tests to determine water-hammer pressures were made with the entire length of sewer constructed of 6-in. plastic tubing. No laterals or risers were installed and all short stub connecting pipes were plugged as described in paragraph 16. The quick-acting closing valve was located at the end of the sewer section with no intervening sump chamber. Pressures were recorded at six locations along the sewer (plate 2) for discharges of 1400, 1800, and 2000 cfs prior to stoppage of the flow. Results of these tests (plates 6-8) indicated that a maximum pressure of approximately 450 lb per sq in. existed at all points for conditions of maximum discharge. Pressures decreased as the discharge prior to valve closure was decreased.

19. The lower 50 ft of the 6-in. tubing was replaced by 4-in. tubing in succeeding tests and the magnitude of the pressure wave was again measured for flows of 1400, 1800, and 2000 cfs with instantaneous valve closure. The maximum pressures in the 4-in. line were approximately 2200 lb per sq in., whereas the maximum pressures in the 6-in. line upstream were only about 750 lb per sq in. (plates 9-11). A comparison of the pressures obtained at the three discharges is shown in table 1. Variation in the

length of time of closure of the valve to determine the effect on water-hammer pressures in the line revealed that for a closure time in excess of 40 sec the resulting water-hammer pressures were negligible.

Surges in Sump of Pumping Station and Sewer

Effect of sump installation

20. A chamber reproducing to scale the proper dimensions and area of the prototype sump of the pumping station was installed in the model. The surge relief flap openings, located near the top of the sump, were not reproduced initially and the chamber was constructed to sufficient height to prevent overtopping (fig. 7). Since the actual time of stoppage of the prototype pumps was unknown, it was assumed that the time of stoppage would probably not exceed 2 minutes. A series of tests with discharges of 1400, 1800, and 2000 cfs was made and the time of stoppage varied from 1 to 156 seconds (table 2). It may be noted from this table that the maximum height of rise of the water surface in the sump chamber varied only slightly regardless of the length of time required to close the valve (pump stoppage). Coincidental with these tests, measurements of the water-hammer pressures existing upline from the sump were also made. These measurements indicated the high pressures noted previously within the closed sewer system upstream were completely relieved by the sump chamber and were in the same range as the height of rise of water in the sump.

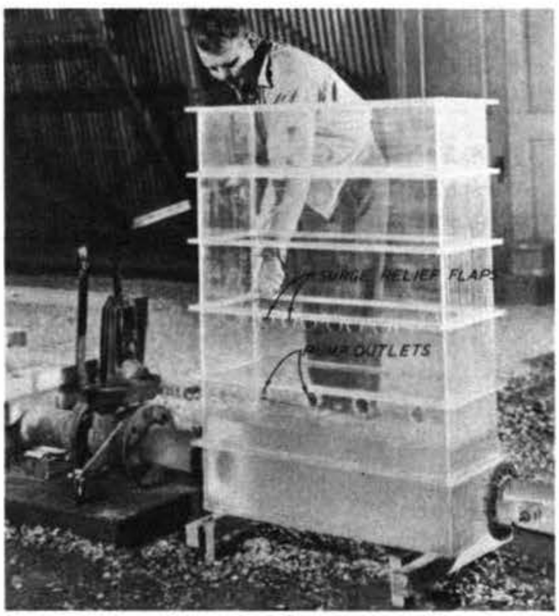

Fig. 7. Sump chamber with surge relief valves and pump outlets

Effect of relief openings and pump outlets

21. The height of rise of water in the sump chamber, as described

in the preceding paragraph, exceeded the prototype elevation of the top of the sump (elev 462). Therefore, it was desired that the effect of the surge relief openings and the pump outlets on the water-surface elevation be studied. Accordingly, the 60-in. relief flap gates were reproduced, seven with a center-line elevation of 452.0 and one with a center-line elevation of 455.5. Tests were made for varying times of stoppage for flows of 1400, 1800, and 2000 cfs. The height of rise of the water level in the sump chamber was greatly reduced from heights previously measured (table 3). Some relief also will probably result from flow through the pump inlets and into the outlet channel even though the pumps are stopped. As the pump impellers and fittings will offer considerable resistance to this flow, the assumption was made that each pump outlet would have an effective area of 50 per cent of its actual area. Openings of the desired size were made in the sump walls at the proper elevations. The previous tests were repeated and, as expected, the height of rise of the water level in the sump chamber was reduced still further (table 4).

Effect of risers (manholes)

22. The five principal manholes in the section of sewer included in the model limits were simulated by the addition of a vertical riser installed in the correct location. The relief openings and pump outlets in the sump chamber were closed and the number of risers was increased one at a time in succeeding tests. The maximum height of water level in the sump and individual risers was measured for flows of 1400, 1800, and 2000 cfs (tables 5-6). Complete hydrographs of the surge in the sump and risers were obtained (plate 12). To study the effect of the initial sump stage on the height of rise, tests were also conducted with the initial sump stage varied over a range of approximately 25 ft. These tests revealed that water-surface elevations in the sump and risers were parallel and varied as the difference in elevation between initial sump stages. Thus the surge rise in feet was the same, regardless of the sump stage at which flow stoppage was accomplished. The stoppage of only one or more of the large pumping units also could cause a surge in the sump chamber and risers. Accordingly, tests were made simulating the closure of individual and combinations of pumping units. The maximum heights of these

surges are listed in table 7. As to be expected, an increase in the amount of flow stoppage increased the amount of surge in the sump chamber and in the sewer upstream.

Variation in sump area

23. Tests were made with sump areas of 700, 1400, and 2100 sq ft to study the effect that variation of the area of the sump would have on the height of rise of water level in the sewer line caused by a stoppage of the pumping units. Results of these tests (tables 8-10) indicated that the height of rise was greatly increased by decreasing the area of the sump.

Submergence of sump outlets

24. The tests previously described were made with the pump outlets and relief flap valves open to atmospheric pressure. In the prototype, however, the head against which the pumps will operate could be above the pump outlet. Therefore, in an attempt to simulate the effect of an increase of the river stage on the surge relief through the pump outlet lines, the pump openings in the sump chamber were raised to the same elevation as the flap relief valves (elev 452.0). This increase in elevation had the same effect as increasing the river stage. The area of pump openings was decreased 50 per cent as in previous tests. Tests were made for flows of 1100, 1400, 1600 cfs as requested by representatives of the Louisville District. Results are listed in table 11.

Dispersion of inlet flow

25. For all previously described tests, the total inflow was introduced into the upper end of the model. To investigate the effect of dispersion of the inlet flow, a total flow of 1800 cfs was distributed between the five main laterals included in the model. The sudden stoppage of flow at the pumping plant resulted in an increase in water level in the sump and risers (table 12) which agreed closely with the results obtained when the total flow was introduced at the upper end of the model.

Storage of surge flow

26. All previously described tests with the risers installed were conducted with the surge flow contained within the risers. To study the effect of permitting the risers to overflow and containing the water in

storage reservoirs, the riser nearest the sump chamber (point 21) was cut off at elevations of 440, 438, and 436 ft in successive tests. For each elevation of top of the riser and the complete stoppage of a flow of 1800 cfs, the maximum elevation attained by the surge in each of the other risers was measured. These tests indicated that lowering the top of this riser from elev 440 to 436 had no effect on the elevation of the surge in the other risers (table 13). However, the reduction in the elevation of the riser from that required to prevent overflow to 440 resulted in a considerable reduction in the maximum elevation of the surge in the risers upline.

27. Tests were also made for the three riser elevations with the flow only partially stopped. These tests simulated the stoppage of one of the 1500-hp pumping units. Under these conditions, only a slight rise in water level was caused in the risers of the sewer line. The riser at point 21 was overtopped very slightly when it was cut off at 436.0 (table 14).

Area of risers

28. In tests to determine the effect of area of risers on the surge the area of each of the five risers or manholes reproduced on the model was increased from 50.68 sq ft (prototype) to 201 sq ft. The height of surge was then measured in each riser for flows of 1400, 1800, and 2000 cfs. The maximum height of surge attained was approximately the same as that obtained with the smaller riser. However, the time required for the surge to reach this elevation was increased (table 15).

Time of Concentration

29. It was realized that the time of concentration of flows could not be simulated for the entire Louisville sewer system, since the model reproduced only a small part of the over-all sewer system. However, it was considered probable that some indications of the relative characteristics of the times of concentration for partially full and for surcharged sewers could be demonstrated by the model. The tests to investigate the time of concentration were made in two phases.

30. The first-phase tests were conducted with the sump and all risers installed. An initial discharge of 1100 cfs was used and the sewer was flowing partially full. A sudden increase in flow was introduced at the upper end of the sewer and the time that elapsed before the discharge increased at the lower end was noted. This test was repeated with the sewer initially surcharged (flowing full) by a flow of 1800 cfs. The elapsed time before increased flow was noted at the pumping station in the partially full sewer was 119 sec (prototype), whereas in the surcharged sewer it was only 28 sec.

31. The second phase of the time-of-concentration tests was made with the risers removed and all openings sealed, thus providing a closed conduit from the upper end of the model to the sump. An initial flow of 750 cfs was introduced into the model so that the 6-in. (model) pipe was about one-third full and the 4-in. (model) pipe was about one-half full. The flow of water was suddenly increased as before and the time for increased flow at the pumping plant was measured. The sewer was then surcharged as before and the time of travel of the additional water again measured. The time of travel for the low flow was 351 sec (prototype) and for the high flow it was less than 6 sec. Thus, it would appear that for conditions of a surcharged sewer system, increases in flow at widely separated areas would be reflected almost instantaneously at the pumping plant. The time of concentration would be affected slightly by the number of manhole openings but not to the degree expected. This factor is important in the determination of the design of pumping capacity for the plant.

PART IV: NARRATIVE OF TESTS -- PUMPING PLANT MODEL

Test Conditions

32. Tests on the 1:16-scale model of the pumping plant (fig. 8) were made for conditions of both gravity and pump flow. During initial tests the outflow was controlled by a valve located in the discharge

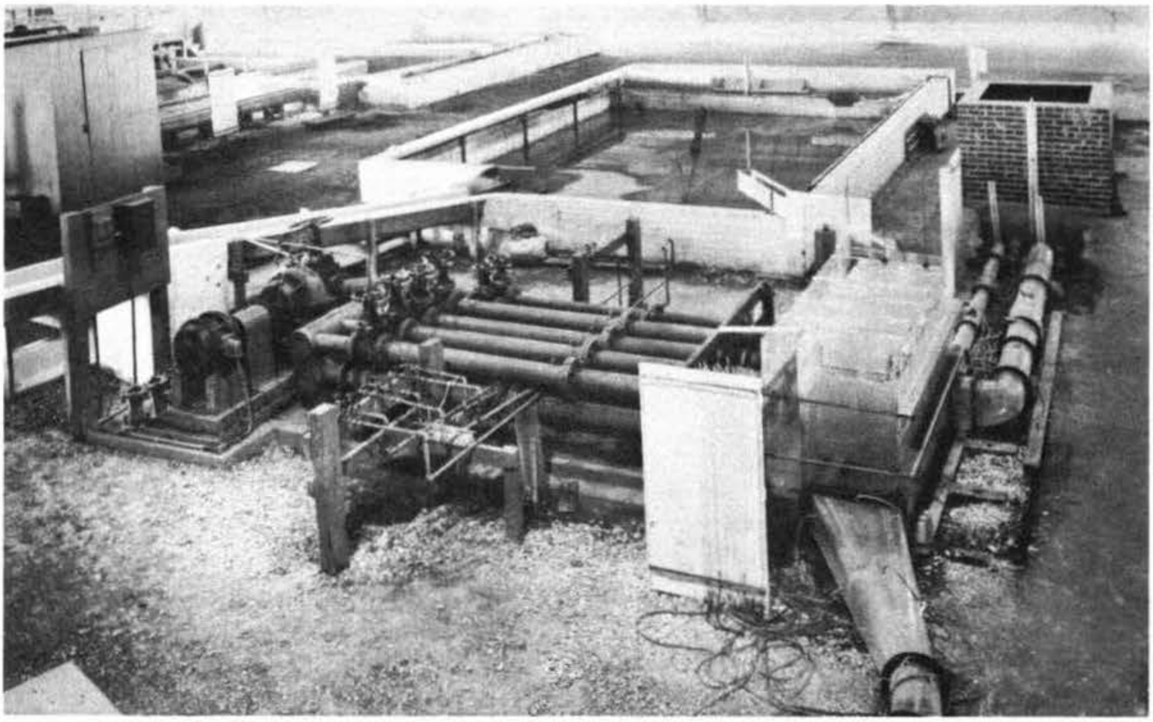

Fig. 8. Details of pumping plant model

sewers downstream from the pumping plant. River stages were set by piezometers located in the sewer lines at the same distance below the pumping plant as are the sewer outlets in the prototype. These sewer lines were later extended and tests made with the river stages reproduced in a tailbay at the end of the sewer discharge lines.

Original Design

33. Details of the original design of the pumping plant are shown on plate 3. Tests revealed the distribution of velocities in the intake

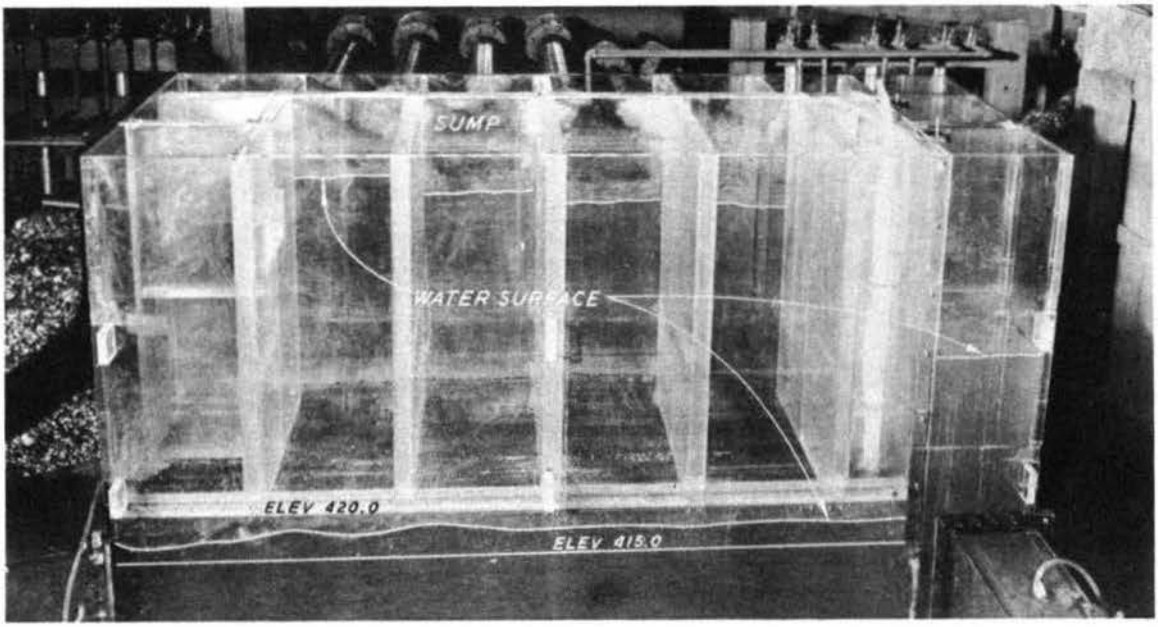

Fig. 9. View of pumping plant from north showing sump chamber in foreground and pump discharge channel in background. Water surface indicated is for conditions of pump flow

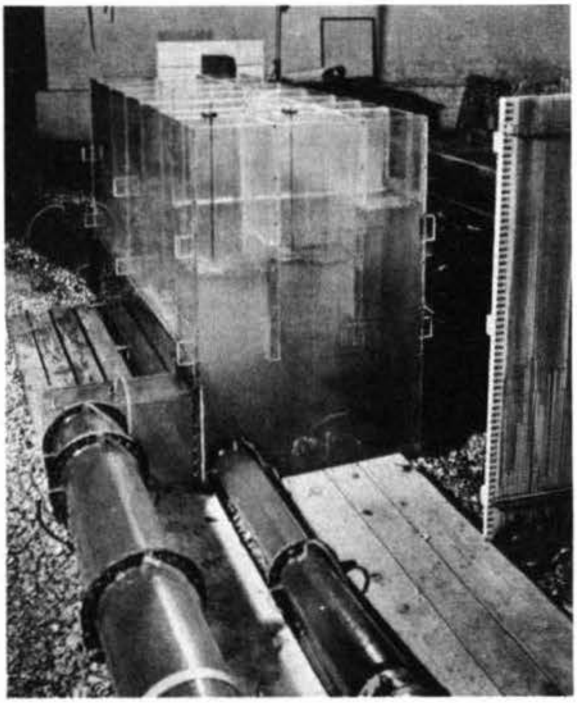

Fig. 10. Pumping plant from west showing pump discharge channel and sewer lines on riverward side of plant

transition to be uneven and losses in the outlet transition to be high for conditions of both pump and gravity flow. Plates 13 and 14 show the velocity distribution in the intake transition and the outfall lines for gravity and pump flow, respectively. Plate 15 indicates the velocity ranges at which the measurements were made. Losses in the discharge channel for the pumps also were very high because of the impediment to flow afforded by the cellular-type partition walls in the channel (figs. 9 and 10).

34. Pressure measurements throughout the intake and outlet

transitions for gravity and pump conditions are listed in tables 16-18 while table 19 lists the elevation of the water surface in the various compartments of the sump chamber and pump discharge channel.

Development of Revisions to Pumping Plant

Revised intake and outlet transitions

35. A revised transition (plate 16) was installed at the intake and the distribution of flow was improved slightly, especially for pumping conditions, as evidenced by the velocity patterns (plates 17-18). The outlet transition to the existing sewer was replaced by an elbow section (plate 16), which also improved flow conditions in the outlet area. Tables 16-19 show a comparison of the elevation of the hydraulic grade lines obtained in the sewer lines and in the pumping plant for both gravity and pump flow with the intake transition revised, and with both intake and outlet transitions revised. Plate 19 shows the locations of the piezometers. The gradient at the upper end of the intake transition was lowered a maximum of 3 ft by the revision of the intake and outlet transitions. Although the revised intake transition provided more even flow throughout, velocities were still higher in the right portion of the transition. However, the eddy which was present in the left portion of the intake transition for the original condition (plate 14) was eliminated. The tests also indicated that the upper part of the entrance to the outlet transition should be rounded and a bellmouthed entrance provided for the 72-in. sewer, as turbulent flow still existed in these areas.

Determination of sump operating range

36. It was desired to maintain the elevation of the hydraulic grade line at the upper end of the intake transition as near elev 416.4 as possible. During pumping operations this gradient was controlled by the water-surface elevation in the sump. Therefore, it was necessary to determine the maximum permissible water-surface elevation in the sump as well as the minimum elevation required to maintain full flow in the intake transition. This safe-operating range was determined to be between

elev 415.5 and 419.5 ft msl (plate 20). Maintenance of a sump elevation of 415.5 eliminated air from the intake transition. A sump elevation higher than 419.5 raised the elevation of the hydraulic grade line at the upstream end of the transition above the desired elevation of 416.4. Therefore, in order to maintain the 5-ft operating range as required for efficient operation of the sump pumps (paragraph 5), it will be necessary to lower the elevation of the sump and transition 1 ft, unless a different method of pump control can be used.

Tests of pump intakes

37. Tests were made to study the relative merits of the draft-tube type pump intake (fig. 11) and the open-pit type intake with separator walls (fig. 12 and plate 21). Visual observation made of the flow conditions existing in the sump chamber and intakes for both types indicated that flow in the open-pit intake was more turbulent than that in the draft-tube intake. However, the relative efficiency curves (plate 22) of the two intakes indicated the open-pit setting type of intake to be slightly more efficient than the draft-tube intake. It is believed that additional tests of the intakes should be made on a larger scale model with the prototype impeller and pump characteristics reproduced, as the scale of this model and the absence of individual pumps would tend to disparage the results obtained.

Operation of pumps

38. A study of the effects of operating various combinations of pumps was made with both the draft-tube and open-pit type intakes installed. In each series of tests the sump level was maintained between elev 417 and 422 and the outfall sewer was allowed to flow free (no river control). The first test consisted of starting the pumps in succession, beginning with the riverside pump and progressing toward the landside pump. The discharge of those pumps already operating did not change as additional pumps were started. No turbulence was observed in the sump until the next to last pump on the landside was started, at which time a marked increase in turbulence was noted in the sump. This turbulence was greatly increased when the most landward pump was started.

39. When the order of starting the pumps was reversed, i.e., the

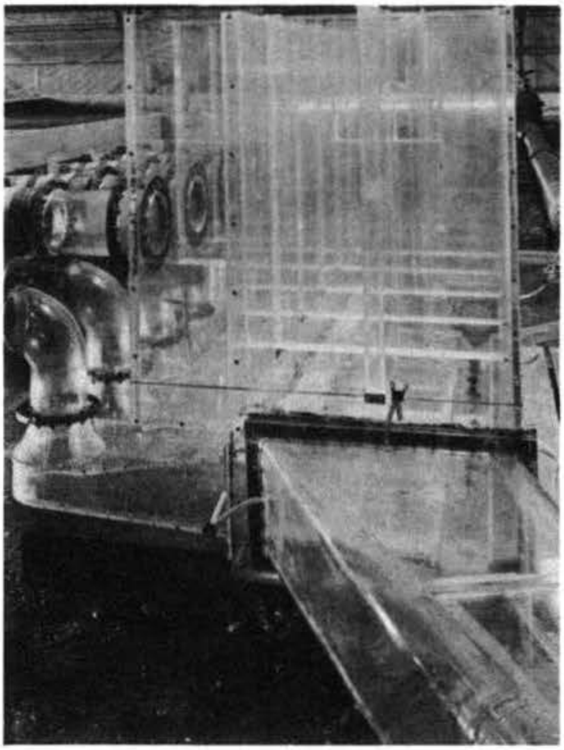

Fig. 11. Draft-tube type
pump intakes

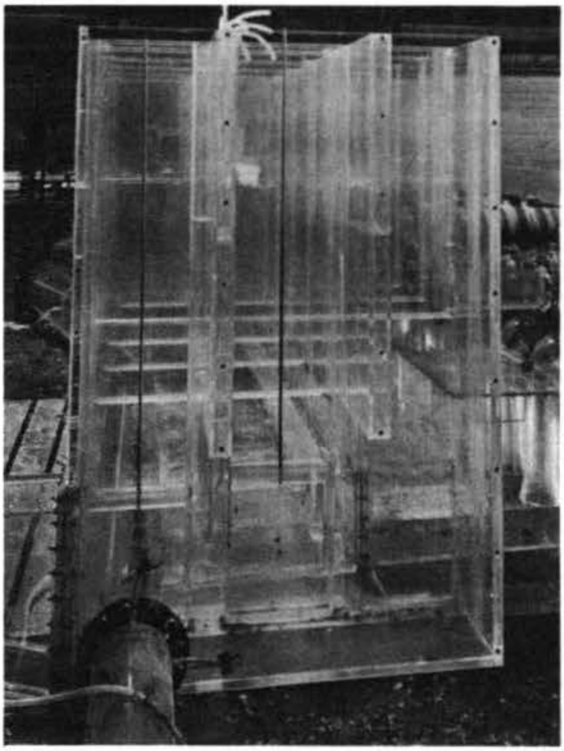

Fig. 12. Open-pit type
pump intakes

pumps were turned on from the landside to the riverside, considerable turbulence was noted when the first pump was started and existed throughout the rest of the test. Numerous other combinations of pumps also were investigated and the effects noted. The starting of additional pumps had no effect on the capacity of the pumps already operating, regardless of the pumping combination used. The only effect noted for any test was the turbulence as discussed above. This turbulent condition is not believed sufficiently great to warrant revision of the design.

Enlargement of sluice gate

40. As the losses through the pumping plant for gravity flow were still slightly higher than considered desirable, one of the sluice gates between the lower end of the sump and the exit channel was enlarged (plate 23). The enlargement of the gate reduced the elevation of the water surface in the sump which in turn reduced the elevation of the hydraulic grade line at the upper end of the intake transition to elev 416.9 (table 20). The water-surface elevations in the sump and discharge

channel were also equalized (compartments 6, 14, and 15, table 21). No further revisions were made to lower the gradient for gravity flow, as representatives of the Louisville District stated that the gradient was satisfactory and any additional lowering desired could be obtained by streamlining the entrances to the outfall sewers.

Revisions to pump discharge channel

41. Several revisions were made to the pump discharge channel in order to increase its capacity. The first of these revisions consisted of removing the two baffle walls in the west exit channel and putting an ogee crest just below the step-down at the lower end of the south discharge channel (fig. 13). A 45-degree fillet, 2 ft on a side, was also added to the southwest corner of the pumping plant (plate 23). These revisions increased the maximum discharge capacity at high river stages from 575 cfs to 740 cfs and reduced the turbulence in the west discharge channel. Later tests indicated that the ogee section below the south channel was not required and it was removed from the model. The capacity of the discharge channel for high river stages was further increased to 920 cfs by cutting a 10-ft by 10-ft relief opening in the riverside of the southwest corner (fig. 14). The hydraulic grade line in the sewer below the pumping plant and the water-surface elevations in the discharge channel obtained during these tests are indicated in tables 22 and 23, respectively.

42. The required pump discharge of 1200 cfs with a river stage of 458.3 ft was not obtained by the preceding revision. The baffle walls in the south discharge channel still caused excessive losses. Therefore, that portion of the south wall baffle walls above elev 445.0 was removed. Although this revision increased the capacity of the discharge channel to the required amount, flow in this channel was still turbulent. Accordingly, all of the baffle walls were removed from the discharge channel and tests were made for various pump flows (table 24). Since supporting members were required in the discharge channel, however, twelve 2-ft-sq horizontal struts were installed. Tests made to determine the effect of these struts on flow conditions (table 25) revealed no obstruction to flow.

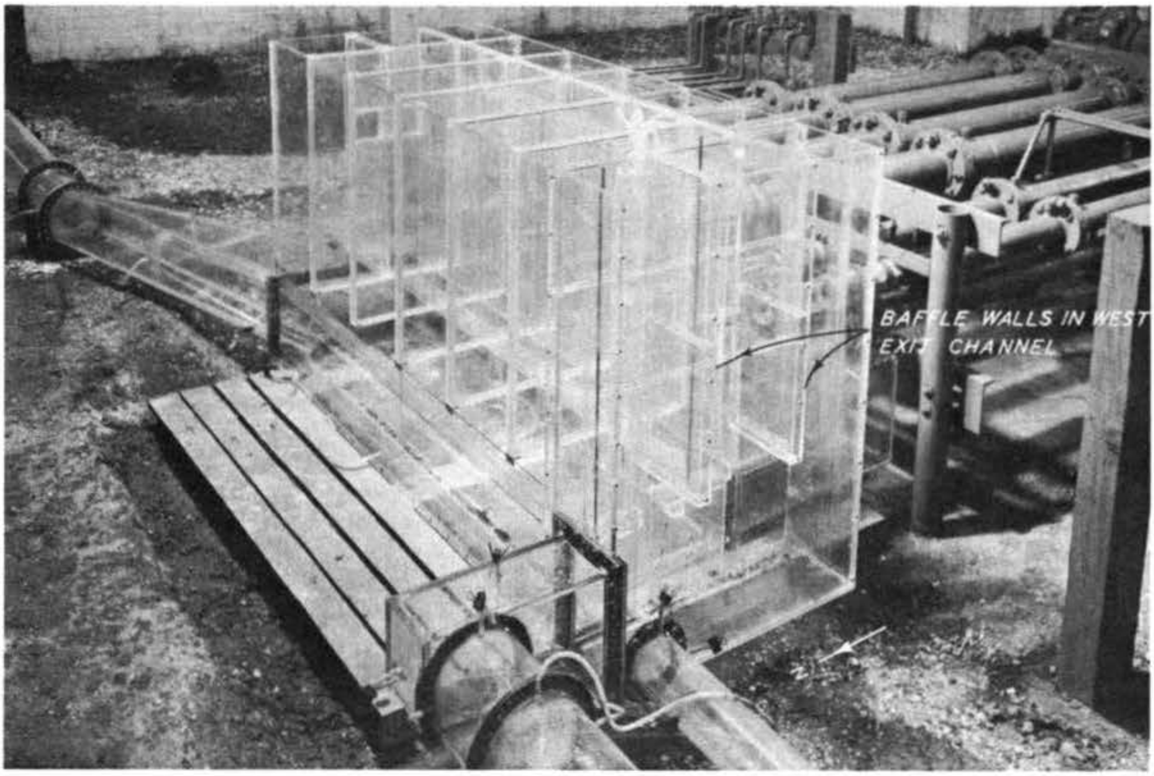

Fig. 13. View of baffle walls in pumping plant

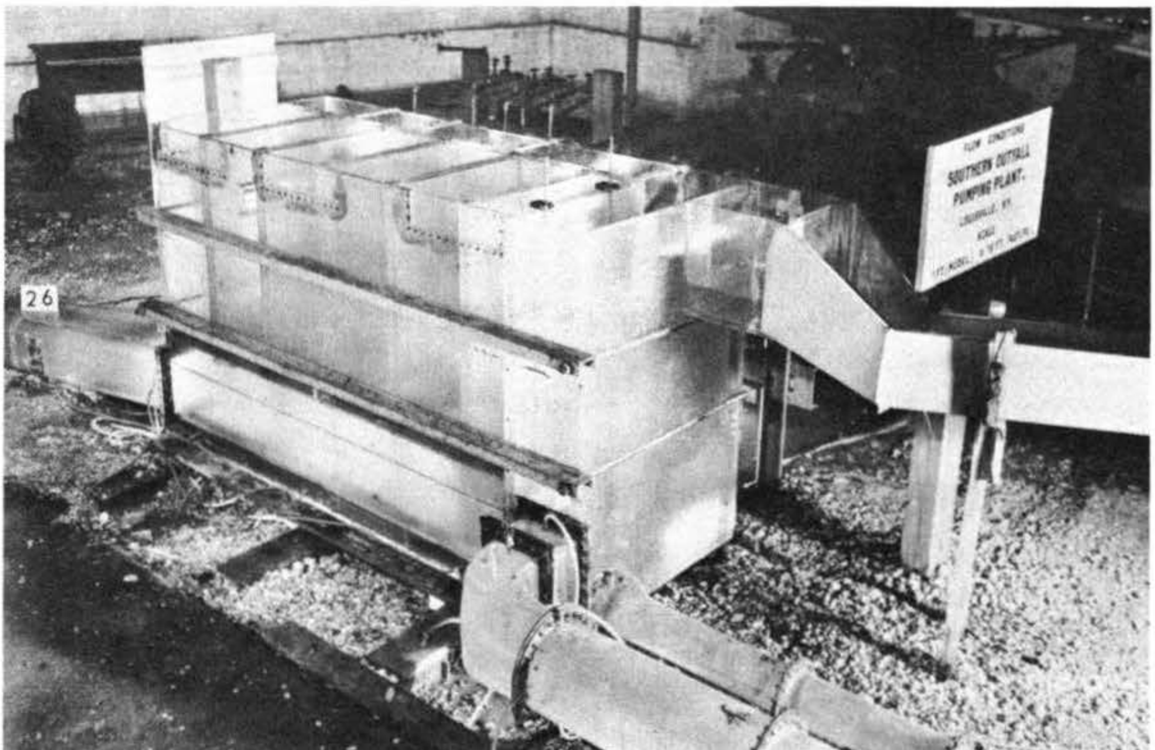

Fig. 14. Relief opening in pump discharge channel

Baffle partition walls in sump chamber

43. In an attempt to reduce surges in the sump chamber, the baffle partition walls in the sump chamber were lowered from elev 420 to elev 415. Tests were made for both pump and gravity flow. These tests revealed no apparent improvement in flow conditions in the sump and no effect on the pressure readings formerly obtained (tables 26-27). The baffle partition walls were then completely removed and no change in sump action was noticed.

Sewer Line Riverward of Pumping Plant

44. The hydraulic grade lines for all flows during initial tests were maintained by a valve at the lower end of the model (plate 3), as only 45 ft (prototype) of the outfall sewers were reproduced below the pumping plant. An analysis was made of the losses* occurring in the outfall lines for both gravity and pump flow (tables 28-29) and of the computed hydraulic grade line elevation at the upper end of the outfall sewer as compared to that used in the model (table 30). These analyses indicated that the control elevations used during the model study for gravity flows of 1800 cfs and pump flows of 1200 cfs were fairly close to the computed elevations, although the elevation of the grade line used for pump flow of 1800 cfs was too low.

45. Accordingly, the outfall sewers were extended in accordance with plans furnished by the Louisville District (fig. 5) and a series of tests was made to determine the hydraulic grade lines for various operating conditions. The following data were obtained with the model conditions as indicated:

a. Hydraulic grade line existing in the 10-ft-7-in. by 10-ft-1/2-in. sewer without the pumping plant installed for a discharge of 1800 cfs and river stage elevations of 382.5 and 427.2 (plate 24).

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* Losses in miter bends determined from curve contained in "Energy Loss in Smooth- and Rough-Surfaced Bends and Curves in Pipe Lines," by Werner Schubait.

b. Hydraulic grade lines existing in the sewer and through the pumping plant for gravity flow of 1800 cfs and river stages of 382.5, 412.0, and 427.2 (plate 25).

c. Hydraulic grade lines in sewer for pump flows of 1800 and 1200 cfs at river stage elevations of 430 and 458.3, respectively (plate 26).

Velocities were measured for all the above flow conditions and the distribution of flow between the existing horseshoe-shaped sewer and the proposed 72-in. circular sewer was computed (table 31).

46. Negative pressures were measured in the sewer lines during some of the above tests, which indicated that the sewer lines had developed siphonic action. The model was revised by the addition of vents (fig. 5) and tests were made as indicated below. Hydraulic grade lines were determined for:

a. Existing conditions without the pumping plant, a flow of 1800 cfs and a river stage of 382.5. Vent installed in existing sewer at sta 0+00 (plates 24 and 27).*

b. Pumping plant installed with 1800-cfs gravity flow and river stage of 382.5. Vent installed in existing sewer at sta 0+00 and in 72-in. proposed sewer at sta 5+60 (plate 27).

c. Pumping plant installed with 1800-cfs gravity flow and a river stage of 382.5. Vents installed in existing sewer at sta 0+00, 5+40, and in 72-in. sewer at sta 5+60 (plate 28). Hydraulic grade lines were not obtained for river stages of 412.0 and 427.2 as the addition of air vents did not change the grade lines previously obtained without vents.

The above tests indicated that the outfall sewers should be vented to obtain the desired capacity for gravity flow. Velocity cross sections were taken in the outfall lines for the various flow conditions and the flow distribution (between the two outfall lines) was improved (table 31).

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* Station 0+00 located at piezometer 1, 247.34 ft upstream from pumping plant.

PART V: DISCUSSION OF TEST RESULTS

Surge Model

47. The simplified model of the Southern Outfall Sewer was constructed for the purpose of obtaining basic data regarding surge action to be expected in a sewer system embracing a main-line sewer and several laterals and risers. Initial studies were conducted to determine the water-hammer pressures encountered in a closed line and the amount of relief afforded these pressures by the addition of an open sump and risers. Additional studies made to determine the effect of varying the time of complete stoppage of pump discharge on the height of surge in the sump and risers indicated that within certain limits this variation in time made no appreciable difference in the height of surge produced.

48. The model tests also indicated that with the Southern Outfall Sewer as originally designed the possibility existed of large and destructive surges developing in the pumping plant sump and along the lower reaches of the main sewer. High or moderately high surges could develop under certain circumstances, not only from total shutdown of the pumping plant, but also from shutdown of individual pump units. Model tests indicated that the pumping plant itself could be protected from damage due to surges in the sump by installation of surge relief valves or flap gates that would open and allow discharge direct to the river. However, these features would not protect the area upline from the sump. Some reduction in surge upline from the sump appears possible by permitting overflow of risers (manholes) into storage areas constructed for the purpose. The results of the time-of-concentration study of flow at the pumping plant from the various areas of the sewer system indicate that, if the sewer is under pressure as planned, a flow increase from any of the areas would be reflected in increased flow at the pumping plant almost immediately. The number of manholes tend to cause a slight time lag but not enough to be a serious factor in design considerations.

49. The results obtained from the model study have been compared with the results obtained from a method developed by the Ohio River

Division* for analytically determining the surge heights in a simple sewer system. The model and computed data compare favorably, thus permitting an analytical computation to be made of the surges to be expected in the Southern Outfall Sewer, rather than determining these surges from a more extensive model study.

Pumping Plant Model

50. Model tests of the State Fairgrounds Pumping Plant indicated that the original design did not produce the desired results. The intake and outlet transitions were revised in order to reduce flow losses through the pumping plant for gravity flow. The gate openings between the sump chamber and the pump discharge chamber were enlarged in order to lower the hydraulic grade line at the upper end of the intake transition to the desired elevation. The cellular construction of the pump discharge channel as originally designed restricted the flow during high river stages to such an extent that only approximately one-half the desired discharge could be passed at high river stages. The recommended design included the removal of all baffle walls in the pump discharge chamber to accommodate the desired discharge. In addition, a relief opening was recommended for direct passage of flow from the plant to the river during high river stages.

51. A study of the relative merits of the draft-tube and open-pit setting types of pump intakes indicated that the open-pit setting was slightly more desirable, although additional model tests are recommended before accepting these results as conclusive. Tests also indicated that the pumps could be operated in any desired combination without affecting the discharge of any other pumps in the plant. A series of tests made on the outfall lines indicated the desirability of venting these lines and of possibly increasing the size of the proposed 72-in. line to improve distribution of flow further.

52. During the course of the tests, consideration was given to the

* Appendices A and B.

location of the pumping plant immediately over the existing sewer alignment. This location should simplify intake transition problems; however, no tests were made with the plant in this position.

Table 1

WATER-HAMMER PRESSURES IN SEWER

Pumping Plant Omitted

Location	Prototype Pressure - Pounds Per Square Inch					
	Discharge = 1400 cfs		Discharge = 1800 cfs		Discharge = 2000 cfs	
	6-in. Tubing	Lower Section 4-in. Tubing	6-in. Tubing	Lower Section 4-in. Tubing	6-in. Tubing	Lower Section 4-in. Tubing
Protection levee	356	1542	519	2000	565	2200
*Pt 21 (4-in. Sect)	---	611	---	----	---	912
Pt 21	307	512	393	685	456	736
Pt 20	282	486	374	627	457	714
Pt 19	288	544	390	695	465	771
Pt 18	303	512	403	646	486	727
Pt 17	290	551	437	690	454	742

Notes: Time of valve closure = 0.944 seconds.
No laterals or risers.
All values are in prototype units except size of tubing.

* This pressure cell added when lower end was changed to 4-in. tubing.

Table 2

EFFECT OF TIME OF VALVE CLOSURE ON WATER-SURFACE ELEVATIONS IN SUMP

No Surge Relief Openings

Discharge = 1400 cfs Velocity = 15.89 fps in 4-in. Line		Discharge = 1800 cfs Velocity = 20.44 fps in 4-in. Line		Discharge = 2000 cfs Velocity = 22.35 fps in 4-in. Line	
Time of Valve Closure Seconds	Max* Elev	Time of Valve Closure Seconds	Max* Elev	Time of Valve Closure Seconds	Max* Elev
0.94	468.30	0.94	485.90	0.94	495.20
3.96	468.48	6.79	485.60	5.66	497.28
10.19	468.38	19.24	485.60	33.96	496.80
39.05	468.32	47.54	485.12	57.17	496.64
78.11	467.42	81.50	484.48	93.39	495.52
109.24	467.20	96.22	484.16	156.22	494.88
135.84	466.40	136.41	483.20	------	------

Notes: All values are in prototype units except size of sewer line. Lower end of model constructed of 4-in. tubing.

*Water-surface elevation in sump at 420.0 msl prior to beginning of tests.

Table 3

EFFECT OF RELIEF OPENINGS ON WATER-SURFACE ELEVATION IN SUMP

Discharge = 1400 cfs Velocity = 15.89 fps in 4-in. Line		Discharge = 1800 cfs Velocity = 20.44 fps in 4-in. Line		Discharge = 2000 cfs Velocity = 22.35 fps in 4-in. Line	
Time of Valve Closure Seconds	Max* Elev	Time of Valve Closure Seconds	Max* Elev	Time of Valve Closure Seconds	Max* Elev
2.8	453.8	2.8	455.7	4.5	457.0
14.2	453.8	38.5	455.7	41.3	456.8
33.4	453.8	65.1	455.7	106.4	456.6
78.7	453.6	97.4	455.7	152.8	456.6
118.9	453.6	133.6	455.4	-----	-----

Notes: All values are in prototype units except size of sewer line. Lower end of model constructed of 4-in. tubing.

*Water-surface elevation in sump at 420.0 ft msl prior to beginning of tests.

Table 4

EFFECT OF RELIEF OPENINGS AND PUMP OUTLETS

ON WATER-SURFACE ELEVATION IN SUMP

Discharge = 1400 cfs Velocity = 15.89 fps in 4-in. Line		Discharge = 1800 cfs Velocity = 20.44 fps in 4-in. Line		Discharge - 2000 cfs Velocity = 22.35 fps in 4-in. Line	
Time of Valve Closure Seconds	Max* Elev	Time of Valve Closure Seconds	Max* Elev	Time of Valve Closure Seconds	Max* Elev
2.8	444.5	4.0	451.2	4.0	452.2
23.8	444.5	20.4	451.2	36.8	452.2
54.9	444.2	49.8	450.9	52.1	452.2
78.1	444.2	74.8	450.9	90.6	452.2
125.1	443.2	121.7	450.9	140.4	451.8
-----	-----	134.1	450.7	-----	-----

Notes: All values are in prototype units except size of sewer line. Lower end of model constructed of 4-in. tubing.

*Water-surface elevation in sump at 420.0 ft msl prior to beginning of tests.

Table 5

EFFECT OF RISERS ON MAXIMUM SURGES IN SUMP AND SEWER LINE

Condition and Location	Water-surface Elevation msl								
	Discharge = 1400 cfs Elev Upper Pool = 442.5			Discharge = 1800 cfs Elev Upper Pool = 455.0			Discharge = 2000 cfs Elev Upper Pool = 465.0		
	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds
Sump - No risers									
Sump	420.0	468.5	130	420.0	485.6	136	420.0	495.2	---
Sump - 1 riser									
Sump	420.0	469.0	132	420.0	485.3	136	420.0	495.4	141
Pt 21	433.9	463.5	127	441.1	479.0	136	446.1	488.7	139
Sump - 2 risers									
Sump	420.0	469.0	130	420.0	485.1	136	420.0	494.9	141
Pt 21	433.8	463.9	127	441.1	479.5	130	446.3	488.8	133
Pt 19	436.6	455.7	127	444.8	470.1	133	451.8	479.4	130
Sump - 3 risers									
Sump	420.0	468.8	133	420.0	485.3	136	420.0	495.8	140
Pt 21	433.6	463.5	133	441.3	479.3	138	446.1	488.8	140
Pt 19	436.5	456.0	133	445.8	470.3	136	451.6	479.7	140
Pt 17	439.6	448.5	133	450.3	462.0	136	457.5	470.6	141

Notes: All values are in prototype units.
Area of sump = 2100 sq ft.
Area of risers = 50.68 sq ft each.
Time of valve closure approximately 3 seconds.

Table 6

MAXIMUM SURGES IN SUMP AND SEWER LINE

WITH 5 RISERS INSTALLED

	Water-surface Elevation msl								
	Discharge = 1400 cfs Elev Upper Pool = 436.5			Discharge = 1800 cfs Elev Upper Pool = 450.0			Discharge = 2000 cfs Elev Upper Pool = 458.5		
Location	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds
Sump	415.0	465.0	141	415.0	480.8	141.4	415.0	490.5	135.8
Pt 21	429.0	459.5	147	435.8	474.0	135.8	441.3	483.5	135.8
Pt 20	429.8	456.2	146.8	437.2	470.5	135.8	443.3	479.8	135.8
Pt 19	432.1	452.5	146.9	440.8	466.0	135.8	447.0	475.3	135.8
Pt 18	434.1	477.8	147	444.0	460.8	135.8	450.9	469.5	135.8
Pt 17	435.5	445.3	147	445.8	458.3	135.8	453.5	467.0	135.8

Notes: All values are in prototype units.
Area of sump = 2100 sq ft.
Area of each riser = 50.68 sq ft.
Time of valve closure approximately 1 second.
Time of rise measured from the time valve is completely closed.
Risers installed at points 17, 18, 19, 20, and 21.

Table 7

MAXIMUM SURGES IN SUMP AND SEWER LINE

FOR PARTIAL REDUCTIONS OF FLOW

Initial Discharge 1800 cfs

		Maximum Water-surface Elevations Units Stopped as Indicated					
		One 1500-hp Pump		Two 1500-hp and One 500-hp Pumps		Three 1500-hp and One 500-hp Pumps	
Location	Normal Elev	Max Elev	Time of Rise Seconds	Max Elev	Time of Rise Seconds	Max Elev	Time of Rise Seconds
Sump	420.0	432.8	141	441.1	130	461.6	130
Point 21	441.1	446.2	62	453.9	96	466.1	130
Point 19	445.4	448.1	62	450.4	96	456.8	130
Point 17	450.2	451.7	96	453.5	130	457.7	130

Notes: All values are in prototype units.
Sump and risers at points 17, 19, and 21 installed in model.
Area of sump = 2100 sq ft.
Area of risers = 50.68 sq ft each.
Discharge = 1800 cfs; elev upper pool = 455.0 ft msl.
Time of valve closure approximately 3 seconds.
Capacity of 1500-hp pump was 386 cfs.
Capacity of 500-hp pump was 128 cfs.

Table 8

MAXIMUM SURGES IN SUMP AND SEWER LINE

Sump Area = 700 sq ft

Location	Water-surface Elevation msl								
	Discharge = 1400 cfs Elev Upper Pool = 436.5			Discharge = 1800 cfs Elev Upper Pool = 450.0			Discharge = 2000 cfs Elev Upper Pool = 458.5		
	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds
Sump	415.0	490.0	74	415.0	514.5	76	415.0	527.5	79
Pt 21	429.0	478.0	79	436.0	498.9	82	441.3	512.0	82
Pt 20	429.5	473.5	85	438.2	493.5	85	443.0	505.0	88
Pt 19	432.0	464.5	85	441.0	483.5	85	447.0	494.3	88
Pt 18	434.0	456.2	88	444.0	472.6	90	450.3	482.7	89
Pt 17	435.0	451.5	91	445.9	467.5	94	453.0	476.7	90

Notes: All values are in prototype units.
Area of each riser = 50.68 sq ft.
Time of valve closure approximately 1 second.
Time of rise measured from the time valve is completely closed.
Risers installed at points 17, 18, 19, 20, and 21.

Table 9

MAXIMUM SURGES IN SUMP AND SEWER LINE

Sump Area = 1400 Sq Ft

	Water-surface Elevation msl								
	Discharge = 1400 cfs Elev Upper Pool = 436.5			Discharge = 1800 cfs Elev Upper Pool = 450.0			Discharge = 2000 cfs Elev Upper Pool = 458.5		
Location	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds
Sump	415.0	472.0	107	415.0	490.0	105	415.0	502.9	130
Pt 21	429.6	462.8	96	436.0	480.2	96	441.3	492.2	130
Pt 20	430.0	460.2	85	436.5	476.2	91	443.1	486.8	130
Pt 19	432.0	455.5	88	441.0	470.5	88	447.0	480.5	130
Pt 18	434.0	448.0	85	444.0	463.1	85	451.2	472.8	130
Pt 17	435.0	446.0	85	445.8	460.0	85	453.6	469.5	130

Notes: All values are in prototype units.
Area of each riser = 50.68 sq ft.
Time of valve closure approximately 1 second.
Time of rise measured from the time valve is completely closed.
Risers installed at points 17, 18, 19, 20, and 21.

Table 10

MAXIMUM SURGES IN SUMP AND SEWER LINE

Sump Area = 2100 sq ft

	Water-surface Elevation msl								
	Discharge = 1400 cfs Elev Upper Pool = 436.5			Discharge = 1800 cfs Elev Upper Pool = 455.0			Discharge = 2000 cfs Elev Upper Pool = 458.5		
Location	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds
Sump	415.0	464.0	141	415.0	480.0	141	415.0	490.5	136
Pt 21	429.0	459.0	147	435.8	474.0	136	441.3	483.5	136
Pt 20	430.0	456.0	147	437.2	470.5	136	443.3	479.8	136
Pt 19	432.0	452.0	147	440.8	466.0	136	447.0	475.3	136
Pt 18	434.0	447.0	147	444.0	460.8	136	450.9	469.5	136
Pt 17	435.0	445.0	147	445.8	458.3	136	453.5	467.0	136

Notes: All values are in prototype units.
Area of each riser = 50.68 sq ft.
Time of valve closure approximately 1 second.
Time of rise measured from the time valve is completely closed.
Risers installed at points 17, 18, 19, 20, and 21.

Table 11

MAXIMUM SURGES IN SUMP AND SEWER LINE

Pump Outlets and 60-in. Relief Openings Installed at Same Elevation

	Water-surface Elevation msl								
	Discharge = 1100 cfs Elev Upper Pool = 432.0			Discharge = 1400 cfs Elev Upper Pool = 436.5			Discharge = 1600 cfs Elev Upper Pool = 442.5		
Location	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds
Sump	415.0	438.0	96	415.0	452.5	91	415.0	453.0	85
Pt 21	425.5	438.0	110	428.0	452.0	91	431.5	454.2	85
Pt 20	426.0	441.0	141	429.2	450.0	91	432.5	452.8	85
Pt 19	428.0	443.5	147	431.6	448.0	91	435.7	450.7	85
Pt 18	429.0	444.0	192	433.0	445.0	124	437.8	448.0	88
Pt 17	430.0	444.0	221	434.0	444.0	124	438.5	446.0	91

Notes: All values are in prototype units.
Area of sump = 2100 sq ft.
Area of risers = 50.68 sq ft.
Time of valve closure approximately 1 second.
Time of rise measured from the time the valve is completely closed.
Risers installed at points 17, 18, 19, 20, and 21.
All openings in sump at elevation 452.0 ft msl.

Table 12

EFFECT OF DISPERSION OF INFLOW ON MAXIMUM SURGES

IN SUMP AND SEWER LINE

		Water-surface Elevation msl Total Discharge at Sump = 1800 cfs Elev Upper Pool = 450.0		
Location	Lateral Inflow cfs	Normal Elev	Max Elev	Time of Rise Seconds
Sump	---	415.0	480.0	142
Pt 21	98	435.8	473.5	142
Pt 20	24	437.5	469.5	142
Pt 19	136	440.6	464.0	142
Pt 18	64	444.0	459.0	142
Pt 17	204	445.0	458.0	142

Notes: All values are in prototype units.
Area of manhole = 50.68 sq ft.
Area of sump = 2100 sq ft.
Time of valve closure approximately 1 second.
Time of rise measured from the time the valve is completely closed.
Risers installed at points 17, 18, 19, 20, and 21.
A discharge of 1274 cfs introduced in main sewer upstream from Pt 17.

Table 13

EFFECT OF OVERTOPPING RISER AT POINT 21 ON

MAXIMUM SURGES IN SUMP AND SEWER LINE

	Water-Surface Elevation msl								
	Discharge = 1800 cfs - Elev Upper Pool = 450.0								
	Elev Top of Riser = 440.0			Elev Top of Riser = 438.0			Elev Top of Riser = 436.0		
Location	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds
Sump	415.0	462.6	113	415.0	461.8	113	415.0	461.3	116
Pt 21	435.4	-----	---	435.5	-----	---	435.5	-----	---
Pt 20	437.2	455.3	141	437.2	455.5	141	437.2	455.5	158
Pt 19	440.5	455.0	141	440.5	454.0	141	440.5	455.0	158
Pt 18	443.8	453.0	141	443.8	453.0	141	443.8	453.0	158
Pt 17	445.5	453.0	141	445.5	453.0	170	445.5	453.5	181

Notes: All values are in prototype units.
Area of sump = 2100 sq ft.
Area of risers = 50.68 sq ft.
Time of valve closure approximately 1 second.
Time of rise measured from the time the valve is completely closed.
Risers installed at points 17, 18, 19, 20, and 21.

Table 14

EFFECT OF OVERTOPPING RISER AT POINT 21 ON MAXIMUM SURGES IN SUMP AND SEWER LINE

Stoppage of One 1500-hp Pump (386 cfs)

	Water-surface Elevation msl								
	Discharge = 1800 cfs - Elev Upper Pool = 450.0								
	Elev Top of Riser = 440.0			Elev Top of Riser = 438.0			Elev Top of Riser = 436.0		
Location	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds
Sump	415.0	416.0	57	415.0	416.0	57	415.0	416.0	57
Pt 21	435.5	436.5	85	435.5	436.5	85	435.5	-----	---
Pt 20	437.2	438.2	113	437.2	438.2	113	437.2	438.0	68
Pt 19	440.5	441.5	141	440.5	441.5	141	440.5	441.0	96
Pt 18	443.8	444.5	141	443.8	444.5	141	443.8	443.8	--
Pt 17	445.5	445.5	---	445.5	445.5	---	445.5	445.5	--

Notes: All values are in prototype units.
Area of sump = 2100 sq ft
Area of risers = 50.68 sq ft
Time of partial valve closure approximately 1 second.
Time of rise measured from the time the valve movement is completely stopped.
Risers installed at points 17, 18, 19, 20, and 21.

Table 15

MAXIMUM SURGES IN SUMP AND SEWER LINE

Area of Risers = 201 sq ft

	Water-Surface Elevation msl								
	Discharge = 1400 cfs Elev Upper Pool = 436.5			Discharge = 1800 cfs Elev Upper Pool = 450.0			Discharge = 2000 cfs Elev Upper Pool = 458.5		
Location	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds	Normal Elev	Max Elev	Time of Rise Seconds
Sump	415.0	463.5	141	415.0	479.9	153	415.0	489.0	164
Pt 21	429.0	458.0	153	435.8	472.5	153	440.0	481.9	164
Pt 20	430.0	456.4	153	437.6	470.9	158	442.0	479.5	170
Pt 19	432.0	452.2	164	440.6	466.3	170	446.0	474.1	175
Pt 18	435.0	448.0	164	444.0	461.7	170	449.5	468.7	181
Pt 17	435.5	446.0	170	445.8	459.1	170	452.0	466.2	181

Notes: All values are in prototype units.
Area of sump = 2100 sq ft.
Time of valve closure approximately 1 second.
Time of rise measured from the time the valve is completely closed.
Risers installed at points 17, 18, 19, 20, and 21.

Table 16

ELEVATION OF HYDRAULIC GRADE LINE FOR GRAVITY FLOW CONDITIONS

Discharge = 1800 cfs   Outlet Sewer Uncontrolled

Piez No.	Original Conditions	Revised Intake Transition	Revised Intake and Outlet Transitions
1	422.0	421.0	419.0
2	422.0	420.8	418.6
3	421.0	421.0	419.0
4	421.0	420.5	418.4
5	421.0	420.5	418.4
6	420.6	420.0	418.2
7	421.0	420.0	418.3
8	420.0	420.0	418.0
9	421.0	418.5	417.0
10	420.0	420.0	418.0
11	420.0	420.0	418.0
12	421.0	420.2	418.5
13	421.0	419.0	418.0
14	419.0	418.5	417.0
15	420.0	419.8	418.0
16	421.0	422.0	420.0
17	421.0	422.0	420.0
18	421.0	422.0	420.0
19	421.0	422.0	420.0
20	421.0	422.0	420.0
21	422.0	422.5	420.4
22	422.0	420.5	420.5
23	422.0	422.0	420.4
24	422.0	422.5	420.4
25	423.0	422.8	421.0
34	422.0	422.5	421.0
35	422.0	422.5	421.0
49	423.0	423.0	421.4
50	407.0	407.0	408.5
51	398.6	399.5	400.5
52	398.6	399.5	400.0
53	410.0	409.0	410.0
54	409.0	409.0	409.5
55	409.0	408.0	409.0
56	409.0	409.0	409.0
57	420.0	420.5	417.0
58	417.0	418.0	415.5
59	419.0	419.5	417.5
60	416.0	416.0	417.5
61	417.0	417.0	416.5
62	411.0	411.0	412.8
63	413.0	413.0	416.8
64	414.0	414.0	415.5
65	414.0	414.0	416.8
66	414.0	414.0	415.5
67	414.0	414.0	414.0
68	415.0	415.4	415.8
69	415.0	415.2	414.5
70	415.0	415.4	415.8
71	422.0	422.4	421.0
75	422.0	422.4	420.8
76	423.0	423.0	421.4
77	423.0	423.0	421.4
78	423.0	423.0	421.4

Note: Piezometer locations are shown on plate 19.

Table 17

ELEVATION OF HYDRAULIC GRADE LINE FOR PUMP FLOW CONDITIONS

Discharge = 1800 cfs River Stage = 430.0 ft msl

Piez No.	Original Conditions	Revised Intake Transition	Revised Intake and Outlet Transitions
1	420.0	418.5	418.5
2	419.4	418.4	418.4
3	419.8	418.5	418.5
4	418.0	418.0	418.0
5	418.0	418.0	418.0
6	417.6	417.8	417.8
7	418.6	417.8	417.8
8	418.0	417.8	417.8
9	418.0	417.5	417.5
10	418.0	417.5	417.5
11	418.4	417.5	417.5
12	419.0	417.8	417.8
13	417.6	417.0	417.0
14	417.0	416.0	416.0
15	418.0	417.0	417.0
16	418.0	418.5	418.5
17	419.0	418.5	418.5
18	419.4	418.5	418.5
19	419.0	418.5	418.5
20	420.0	418.5	418.5
21	421.0	419.0	419.0
22	420.0	419.0	419.0
23	419.0	419.0	419.0
24	419.6	420.0	420.0
25	421.0	420.0	420.0
34	448.4	452.0	452.0
35	448.0	451.0	451.0
49	420.6	420.5	420.5
50	430.5	431.0	431.0
51	417.0	415.0	420.0
52	417.0	418.0	420.0
53	424.0	420.0	424.6
54	428.0	428.6	428.4
55	428.4	426.0	429.0
56	428.4	429.0	428.8
57	438.0	439.0	433.6
58	437.0	437.0	433.6
59	435.0	436.0	434.0
60	433.0	433.0	435.0
61	420.8	430.0	434.4
62	426.0	427.0	427.2
63	431.0	431.0	433.2
64	431.0	431.0	432.0
65	430.5	431.0	433.0
66	431.0	431.0	432.2
67	431.0	431.0	431.6
68	431.2	432.0	432.4
69	432.4	432.0	429.6
70	432.8	432.0	432.6
71	449.0	449.2	446.6
75	448.0	448.2	446.0
76	420.0	420.5	433.6
77	421.8	420.5	433.6
78	421.2	420.5	433.6

Note: Water-surface elevation in sump maintained at 420.0 ft msl at upper end. Piezometer locations are shown on plate 19.

Table 18

ELEVATION OF HYDRAULIC GRADE LINE FOR PUMP FLOW CONDITIONS

Discharge = 575 cfs* River Stage = 458.3 ft msl

Piez No.	Original Conditions	Revised Intake Transition	Revised Intake and Outlet Transitions
1	420.2	419.0	419.6
2	417.6	419.0	419.6
3	420.2	419.0	419.7
4	420.1	419.0	419.8
5	420.1	419.0	419.8
6	420.0	419.0	419.8
7	420.1	419.2	419.8
8	420.0	419.2	419.8
9	420.1	420.2	419.8
10	420.1	419.2	419.8
11	420.1	419.4	419.8
12	420.2	419.6	419.9
13	420.0	419.0	419.8
14	419.8	419.4	419.6
15	420.0	419.8	419.8
16	420.1	420.2	420.0
17	-----	421.8	419.8
18	420.2	420.4	420.0
19	420.2	420.2	419.9
20	420.2	420.2	420.0
21	420.2	420.8	420.0
22	420.4	420.2	420.0
23	420.4	420.2	420.0
24	420.4	420.4	420.0
25	420.4	420.4	420.0
34	460.2	460.2	460.1
35	460.1	460.1	460.0
49	420.0	421.4	419.8
50	458.2	458.0	458.5
51	456.8	456.6	457.4
52	456.8	456.6	457.3
53	457.5	457.2	458.0
54	458.0	457.6	458.3
55	457.9	457.5	455.2
56	458.0	457.6	458.4
57	459.0	458.7	459.0
58	458.8	458.5	453.0
59	458.7	458.4	459.0
60	458.0	458.2	459.1
61	457.7	457.8	457.6
62	458.3	457.6	458.3
63	458.2	458.0	458.8
64	458.2	457.9	458.7
65	458.2	458.0	458.0
66	458.1	458.0	458.6
67	458.3	458.0	458.6
68	458.2	458.0	458.6
69	458.7	458.0	457.6
70	458.3	458.0	456.6
71	460.2	459.9	460.3
75	460.0	459.8	460.0
76	-----	421.8	420.0
77	-----	422.2	420.0
78	-----	422.2	420.0

Notes: Water-surface elevation in sump maintained at 420.0 ft msl at upper end. Piezometer locations are shown on plate 19.

* Maximum quantity of water that could be pumped due to piling up of water in upper end of discharge channel.

Table 19

WATER-SURFACE ELEVATIONS IN PUMPING PLANT

Compartment No.	Pumps Operating Discharge = 1800 cfs Tailwater = 430.0			Gravity Flow Discharge = 1800 cfs			Pumps Operating Discharge = 575 cfs Tailwater = 458.3		
	Original	Revised Intake	Revised Intake and Outlet	Original	Revised Intake	Revised Intake and Outlet	Original	Revised Intake	Revised Intake and Outlet
1	420.0	Same as Original	420.0	422.0	Same as Original	420.5	420.0	Same as Original	420.0
2	420.0		420.0	422.5		420.8	420.0		420.0
3	420.0		420.5	422.5		421.0	420.0		420.0
4	421.0		421.0	422.5		421.2	420.0		420.0
5	421.0		421.0	422.6		421.4	420.0		420.0
6	421.0		421.0	422.6		421.5	420.0		420.0
7	450.6		448.0				459.8		460.0
8	450.6		448.0				459.7		460.0
9	450.0		447.0				459.6		459.9
10	448.2		445.8				459.4		459.8
11	447.0		445.0				459.2		459.5
12	446.5		444.0				459.1		459.3
13	446.0		443.5				458.9		459.2
14	439.0		436.8	422.0		421.0	458.2		458.4
15	438.4		436.0	422.0		420.0	458.0		458.4

Notes: The water-surface elevations are average for each compartment.
For compartment locations see plate 15.

Table 20

EFFECT OF ENLARGED SLUICE GATE ON HYDRAULIC GRADE LINE

FOR GRAVITY FLOW CONDITIONS

Discharge = 1800 cfs   River Stage Uncontrolled

Revised Intake and Outlet Transitions

Piez No.	Original Gate Opening	Enlarged Gate Opening
1	419.0	418.7
2	418.6	418.4
3	419.0	418.6
4	418.4	418.1
5	418.4	418.0
6	418.2	417.7
7	418.3	417.8
8	418.0	417.6
9	417.0	417.0
10	418.0	416.9
11	418.0	417.3
12	418.5	417.6
13	418.0	416.9
14	417.0	416.1
15	418.0	416.9
16	420.0	419.5
17	420.0	419.5
18	420.0	418.6
19	420.0	419.3
20	420.0	419.5
21	420.4	420.1
22	420.5	420.2
23	420.4	420.0
24	420.4	420.0
25	421.0	420.8
34	421.0	420.5
35	421.0	420.5
49	421.4	421.0
50	408.5	408.0
51	400.5	400.5
52	400.0	400.1
53	410.0	409.5
54	409.5	408.5
55	409.0	408.5
56	409.0	409.0
57	417.0	417.7
58	415.5	416.4
59	417.5	417.3
60	417.5	417.8
61	416.5	416.4
62	412.8	412.5
63	416.8	416.7
64	415.5	415.6
65	416.8	415.9
66	415.5	415.6
67	414.0	413.7
68	415.8	415.5
69	414.5	415.3
70	415.8	415.5
71	421.0	420.5

Note: Piezometer locations are shown on plate 19.

Table 21

EFFECT OF ENLARGED SLUICE GATE ON WATER-SURFACE ELEVATIONS

FOR GRAVITY FLOW CONDITIONS

Discharge = 1800 cfs River Stage Uncontrolled

Revised Intake and Outlet Transitions

Compartment No.	Original Gate Opening	Enlarged Gate Opening
1	420.5	420.0
2	420.8	420.4
3	421.0	420.6
4	421.2	420.7
5	421.4	420.9
6	421.5	421.0
14	421.0	421.0
15	420.0	420.0

Notes: The water-surface elevations are average for each compartment. For compartment location see plate 15.

Table 22

EFFECT OF REVISIONS TO PUMP DISCHARGE CHANNEL ON HYDRAULIC GRADE LINE

Revised Intake and Outlet Transitions

Piez No.	Discharge = 1800 cfs River Stage = 430.0 ft msl Modified Step West Baffles Removed	Discharge = 740 cfs River Stage = 458.3 ft msl Modified Step West Baffles Removed	Discharge = 920 cfs River Stage = 458.3 ft msl Modified Step West Baffles Removed Relief Opening Installed
50	428.0	458.0	458.4
51	421.0	456.7	457.1
52	421.0	456.8	457.0
53	428.3	457.9	458.1
54	428.5	458.0	458.1
55	428.9	457.8	458.0
56	429.5	457.9	458.1
57	436.2	459.0	459.0
58	436.1	459.0	458.9
59	434.6	458.9	458.8
60	436.0	459.1	459.0
61	434.0	458.9	459.3
62	428.2	457.8	458.1
63	433.6	458.8	458.8
64	432.8	458.5	458.6
65	433.2	458.7	458.6
66	432.9	458.6	458.6
67	431.4	458.4	458.4
68	432.8	458.5	458.6
69	432.6	458.5	458.6
70	432.8	458.5	458.6

Note: Locations of piezometers are shown on plate 19.

Table 23

EFFECT OF REVISIONS TO PUMP DISCHARGE CHANNEL ON

WATER-SURFACE ELEVATIONS IN PUMPING PLANT

Revised Intake and Outlet Transitions

Compartment No.	Discharge = 1800 cfs River Stage = 430.0 ft msl Modified Step West Baffles Removed	Discharge = 740 cfs River Stage = 458.3 ft msl Modified Step West Baffles Removed	Discharge = 920 cfs River Stage = 458.3 ft msl Modified Step West Baffles Removed Relief Opening Installed
1	420.0	420.0	420.0
2	420.0	420.0	420.0
3	420.0	420.0	420.0
4	420.0	420.0	420.0
5	420.0	420.0	420.0
6	420.0	420.0	420.0
7	442.9	460.0	460.0
8	442.6	459.8	459.9
9	441.8	459.7	459.8
10	440.3	459.4	459.4
11	439.6	459.2	459.3
12	438.7	459.0	459.1
13	438.0	458.8	458.6
14	438.0	458.2	458.6
15	437.8	458.0	458.5

Note: Locations of compartments are shown on plate 15.

Table 24

EFFECT OF REMOVAL OF BAFFLE WALLS FROM DISCHARGE CHANNEL

ON WATER-SURFACE ELEVATIONS IN PUMPING PLANT

Revised Intake and Outlet Transitions

Compart-ment No.	South Baffles Removed to Elev 445.0				South Baffles Completely Removed			
	Relief Opening Closed		Relief Opening Operating		Relief Opening Closed		Relief Opening Operating	
	Q = 1800 RS = 430.0	Q = 925 (max) RS = 458.3	Q = 1200 RS = 458.3	Q = 1350 (max) RS = 458.3	Q = 1800 RS = 430.0	Q = 1050 (max) RS = 458.3	Q = 1200 RS = 458.3	Q = 1670 (max) RS = 458.3
1	420.0	420.0	420.0	420.0	420.0	420.0	420.0	420.0
2	420.0	420.0	420.0	420.0	420.0	420.0	420.0	420.0
3	420.0	420.0	420.0	420.0	420.0	420.0	420.0	420.0
4	420.0	420.0	420.0	420.0	420.0	420.0	420.0	420.0
5	420.0	420.0	420.0	420.0	420.0	420.0	420.0	420.0
6	420.0	420.0	420.0	420.0	420.0	420.0	420.0	420.0
7	444.0	459.2	459.2	459.4	440.1	459.3	459.0	459.8
8	443.4	459.2	459.2	459.4	439.7	459.3	459.0	459.7
9	442.7	459.1	459.1	459.3	439.6	459.5	458.9	459.6
10	441.6	459.0	458.8	458.9	439.1	459.2	458.8	459.5
11	439.0	458.8	458.6	458.6	437.6	459.1	458.7	459.4
12	438.5	458.7	458.4	458.4	437.4	459.0	458.6	459.2
13	438.0	458.5	458.4	458.4	437.1	459.0	458.6	459.0
14	437.8	458.5	458.3	458.3	436.5	458.9	458.6	459.1
15	437.5	458.5	458.4	458.3	437.6	459.1	458.8	459.2

Notes: Locations of compartments are shown on plate 15.
All elevations in ft msl.

Table 25

WATER-SURFACE ELEVATIONS IN PUMPING PLANT WITH STRUTS

IN SOUTH CHANNEL REPLACING BAFFLE WALLS

Revised Intake and Outlet Transition

Compartment No.	Discharge = 1200 cfs River Stage = 458.3 ft msl	Discharge = 1800 cfs River Stage = 430.0 ft msl
1	420.0	420.0
2	420.0	420.0
3	420.0	420.0
4	420.0	420.0
5	420.0	420.0
6	420.0	420.0
7	459.2	440.7
8	459.2	440.0
9	459.1	439.7
10	459.0	439.5
11	458.9	438.0
12	458.7	437.7
13	458.5	437.3
14	458.7	436.3
15	458.7	437.9

Notes: Locations of compartments are shown on plate 15.
Spillway crest removed from south channel stepdown.
Fillet still in southwest corner.

Table 26

EFFECT OF REMOVAL OF BAFFLE WALLS FROM SUMP CHAMBER

ON WATER-SURFACE ELEVATIONS IN PUMPING PLANT FOR GRAVITY FLOW

Enlarged Gate Opening

Revised Intake and Outlet Transitions

Compartment No.	Discharge = 1800 cfs		
	Bottom of Sump Baffles at Elev 420.0	Bottom of Sump Baffles at Elev 415.0	
		East*	West*
1	420.0	420.0	420.0
2	420.5	420.0	420.5
3	420.6	420.2	420.8
4	420.8	420.5	421.0
5	420.9	420.5	420.5
6	421.0	420.4	420.4
14	420.2	420.0	420.0
15	420.2	420.0	420.0

*Denotes average water-surface elevation measured along east and west sides, respectively, of each compartment.

Note: Locations of compartments are shown on plate 15.

Table 27

EFFECT OF REMOVAL OF BAFFLE WALLS FROM SUMP CHAMBER

ON ELEVATION OF HYDRAULIC GRADE LINE FOR GRAVITY FLOW CONDITIONS

Enlarged Gate Opening

Revised Intake and Outlet Transitions

	Discharge = 1800 cfs	
Piez No.	Bottom of Sump Baffles at Elev 415.0	Bottom of Sump Baffles at Elev 420.0
1	418.5	418.6
2	418.0	418.3
3	418.1	418.6
4	417.5	418.1
5	417.3	418.0
6	417.1	417.4
7	417.3	417.9
8	-----	417.5
9	416.8	417.0
10	416.6	417.0
11	417.1	417.3
12	417.1	417.5
13	416.9	416.8
14	416.0	416.1
15	416.9	417.0
16	418.8	419.4
17	419.0	419.5
18	419.3	419.5
19	419.0	419.4
20	418.9	419.5
21	419.8	420.0
22	419.9	420.0
23	419.9	420.2
24	420.0	420.0
25	420.2	420.3
49	420.2	420.8
50	407.7	408.0
51	400.0	400.5
52	400.0	400.1
53	410.0	409.5
54	409.0	408.5
55	408.4	408.5
56	408.9	409.0
57	417.6	417.7
58	416.4	416.4
59	417.1	417.3
60	418.1	418.2
61	416.7	416.4
62	412.5	412.5
63	416.7	416.7
64	415.6	415.6
65	416.0	415.4
66	415.6	415.6
67	413.9	413.7
68	415.6	415.5
69	415.5	415.3
70	415.7	415.5

Note: Location of piezometers shown on plate 19.

Table 28

COMPUTED FLOW LOSSES IN PROTOTYPE OUTFALL SEWER LINES - SUMP TO EXIT

DISTRIBUTION OF FLOW COMPUTED FROM VELOCITY MEASUREMENTS

GRAVITY FLOW

Sewer	Discharge cfs	Section	Area in Sq ft	Head Loss in Pipe	Entrance Loss K = 0.5	Contraction Loss	Enlargement Loss	Bend Loss #1	Bend Loss #2	Exit Loss	Total Loss
Existing	1200	A	88.6	0.88	1.43						2.31
n = 0.16	Total Q =	B	49.3	3.14		0.88	0.81	0.46	0.46		5.75
	1800 cfs	C	54.3	0.98						7.62	8.60
											16.66
Proposed	600	A	28.3	2.95	3.48						6.43
72 in.	Total Q =	B	28.3	1.97				0.35	0.35		2.67
n = .013	1800 cfs	C	28.3	0.89						6.96	7.85
											16.95
Existing	356	A	88.6	0.08	0.08						0.16
n = .016	Total Q =	B	49.3	0.28		0.06	0.07	0.04	0.04		0.49
	1200 cfs	C	54.3	0.09						0.67	0.76
	RS = 458.3										1.41
Proposed	179	A	28.3	0.26	0.21						0.47
72 in.	Total Q =	B	28.3	0.18				0.06			0.24
n = .013	1200 cfs	C	28.3	0.08						0.62	0.70
	RS = 458.3										1.41

Notes: Distribution for flow of 1200 cfs determined as follows: Total flow (1200) less flow through relief opening (665) = flow through sewers (535). This flow was distributed between existing sewer and proposed sewer in same ratio as for gravity flow at 1800 cfs.

The 72-in. sewer assumed to have same slope as existing sewer and a constant area.

Pipe assumed flowing full.

Table 29

COMPUTED FLOW LOSSES IN PROTOTYPE OUTFALL SEWER LINES - SUMP TO EXIT

DISTRIBUTION OF FLOW COMPUTED FROM VELOCITY MEASUREMENTS

PUMP FLOW

Sewer	Discharge cfs	Section	Area in Sq ft	Head Loss in Pipe	Entrance Loss K = 0.5	Contraction Loss	Enlargement Loss	Bend Loss #1	Bend Loss #2	Exit Loss	Total Loss
Existing	1360	A	88.6	1.13	1.29						2.42
n = .016	Total Q =	B	49.3	4.04		1.73	1.05	0.59	0.59		8.00
	1800 cfs	C	54.3	1.25						9.76	11.01
											21.43
Proposed	440	A	28.3	2.42	1.32						3.74
72 in.	Total Q =	B	28.3	1.61				0.19	0.19		1.99
n = .013	1800 cfs	C	28.3	0.73						3.76	4.49
											10.22
Existing	405	A	88.6	0.10	0.11						0.21
n = .016	Total Q =	B	49.3	0.36		0.12	0.10	0.055	0.055		0.69
	1200 cfs	C	54.3	0.11						0.87	0.98
											1.88
Proposed	130	A	28.3	0.21	0.11						0.32
72 in.	Total Q =	B	28.3	0.14				0.015	0.015		0.17
n = .013	1200 cfs	C	28.3	0.06						0.33	0.39
											0.88

Notes: Distribution for flow of 1200 cfs determined as follows: Total flow (1200) less flow through relief opening (665) = flow through sewers (535). This flow was distributed between existing sewer and 72-in. sewer in same ratio as for pump flow at 1800 cfs.

The 72-in. sewer assumed to have same slope as existing sewer and a constant area.

Pipe assumed flowing full.

Table 30

ELEVATION OF HYDRAULIC GRADE LINE AT

UPPER END OF OUTFALL LINES

Location	Model Elev msl	Computed Elev msl Dist A	Computed Elev msl Dist B
	Gravity Flow - 1800 cfs		
Exist. Sewer Piez. 70	415.5	413.2	-----
72-in. Sewer Piez. 56	409.0	408.6	-----
	Pump Flow - 1800 cfs		
Exist. Sewer Piez. 70	432.3	442.1	446.2
72-in. Sewer Piez 56	428.2	436.0	434.8
	Pump Flow - 1200 cfs		
Exist. Sewer Piez. 70	458.5	459.2	459.7
72-in. Sewer Piez. 56	458.0	458.9	458.7

Note: Elev at piezometer 70 is average of piezometers 68 and 70.
Elev at piezometer 56 is average of piezometers 54, 55, and 56.
Distribution A is the distribution of flow as indicated in table 28.
Distribution B is the distribution of flow as indicated in table 29.
All measurements are in prototype units.

Table 31

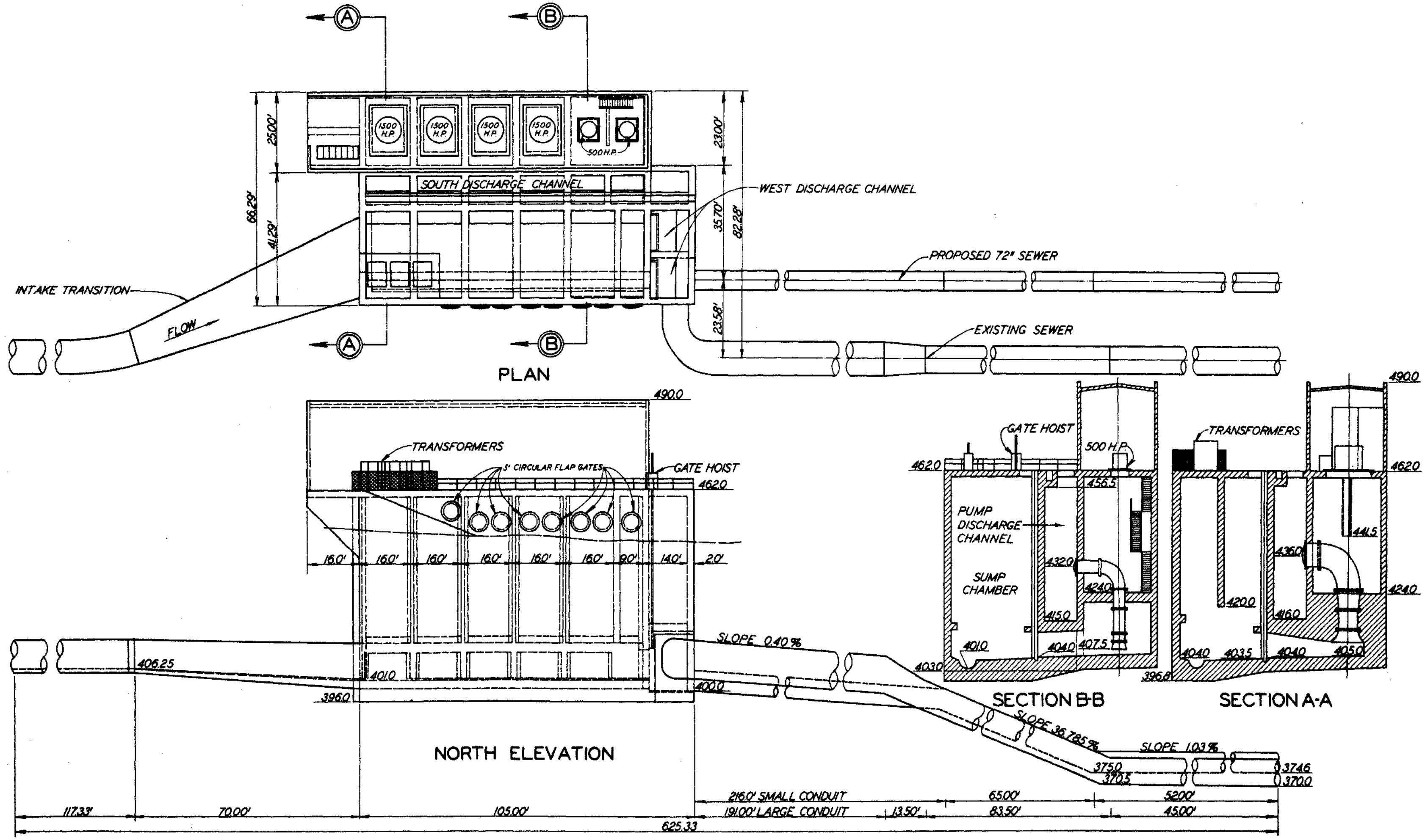
DISTRIBUTION OF FLOW BETWEEN EXISTING SEWER

AND PROPOSED 72-IN. SEWER

Test Condition	Discharge in cfs			
	Not Vented		Vented	
	Large Sewer	72-in. Sewer	Large Sewer	72-in. Sewer
Gravity Flow Total Q = 1800 RS = 382.5	975	825	1170	630
Gravity Flow Total Q = 1800 RS = 412.0	1220	580	1220	580
Gravity Flow Total Q = 1800 RS = 427.2	1250	550	1230	570
Pump Flow Total Q = 1800 RS = 430.0	1360	440	1260	540
Pump Flow Total Q = 1200* RS = 458.3	310	110	310	110

* Excess flow carried by overflow relief opening.

Notes: All discharges in prototype cfs.
All elevations in ft msl.

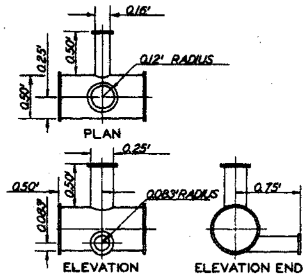


DETAILS OF STATE FAIRGROUNDS PUMPING PLANT

SCALES

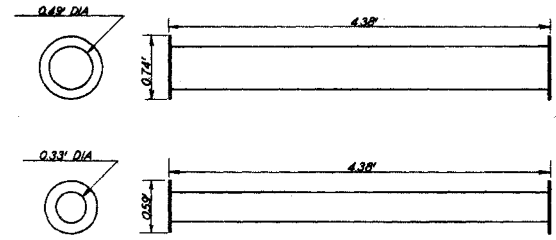
NOTE : ALL UNITS ARE IN THEIR PROTOTYPE EQUIVALENTS

PLATE 1

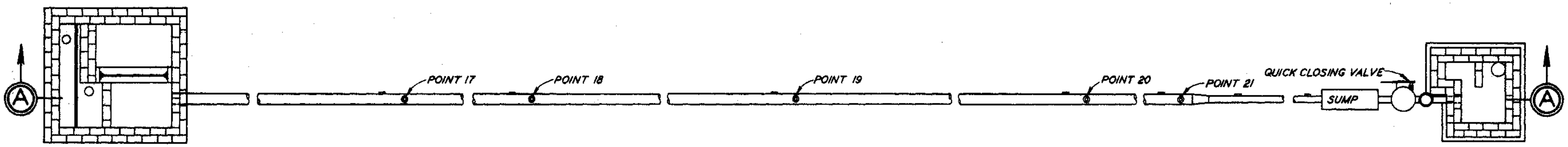


RISER AND LATERAL

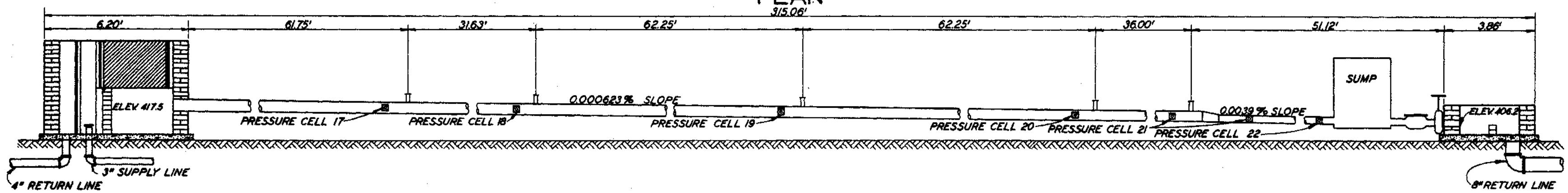
SUMMARY OF LATERALS AND RISERS				
LOCATION POINT	LATERAL NUMBER	LATERAL SIZE	RISER NUMBER	RISER SIZE
17	2	0.1667'	1	0.25'
18	1	0.1667'	1	0.25'
19	1	0.2083'	1	0.25'
20	1	0.0866'	1	0.25'
21	1	0.1667'	1	0.25'



TUBING DETAILS



PLAN



SECTION A-A

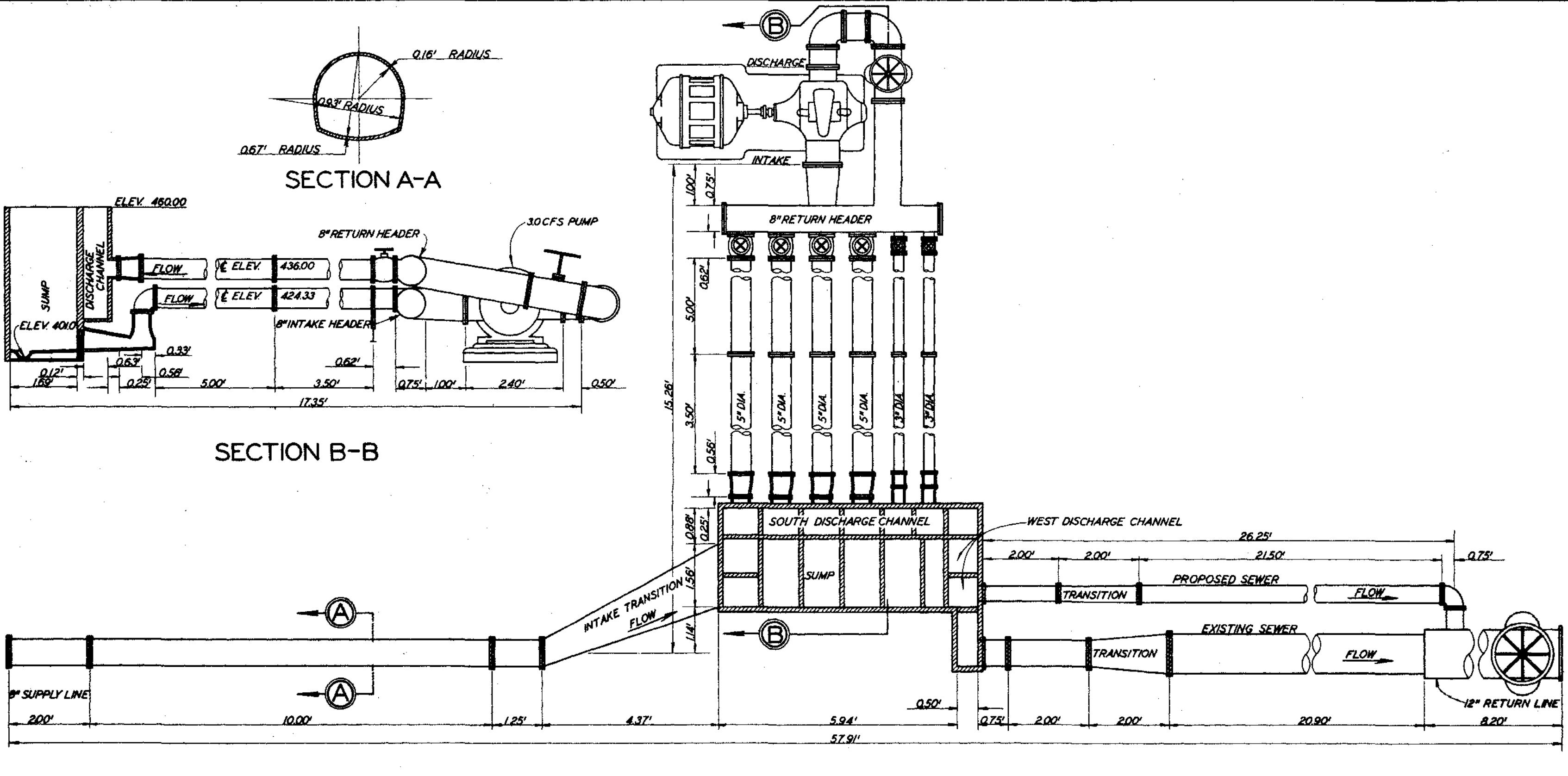
NOTE: MODEL CONSTRUCTED OF PLASTIC TUBING AS INDICATED.
MODEL INITIALLY CONSTRUCTED OF 0.5' TUBING THROUGHOUT LOWER SECTION LATER CHANGED TO 0.33' TUBING AS SHOWN.
LATERAL AND RISER WERE ADDED AFTER INITIAL TESTS.
ALL DIMENSIONS ARE GIVEN IN MODEL UNITS.

MODEL LAYOUT
SURGE STUDY
SOUTHERN OUTFALL SEWER

SCALES

PROTOTYPE 32 0 32 96 160 224 288 FT

MODEL 0 1 2 3 4 5 6 7 8 9 FT



PLAN

NOTE: ALL DIMENSIONS ARE GIVEN IN MODEL UNITS.

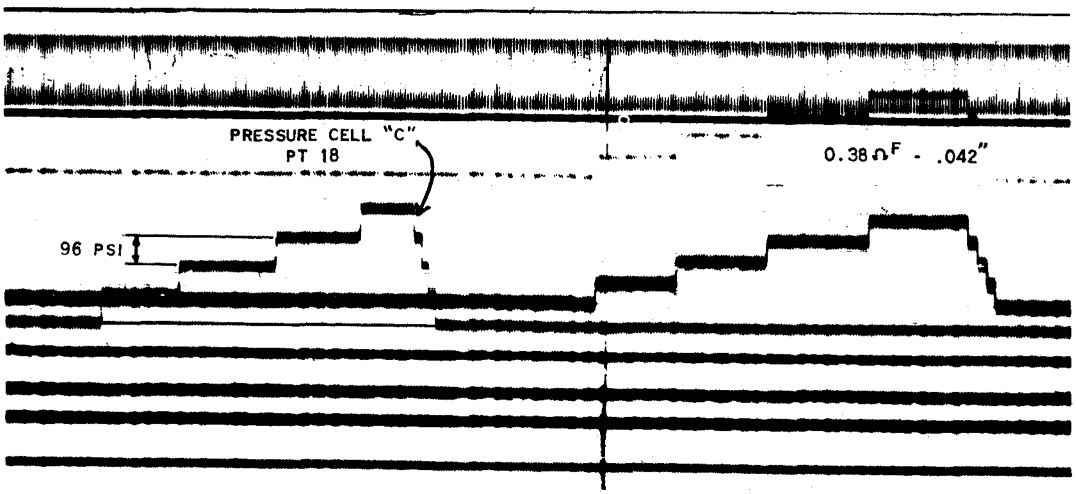
MODEL LAYOUT
STATE FAIRGROUNDS PUMPING PLANT
LOUISVILLE, KENTUCKY
INITIAL CONDITION

SCALES

PROTOTYPE 16 0 16 32 48 64 80 FT

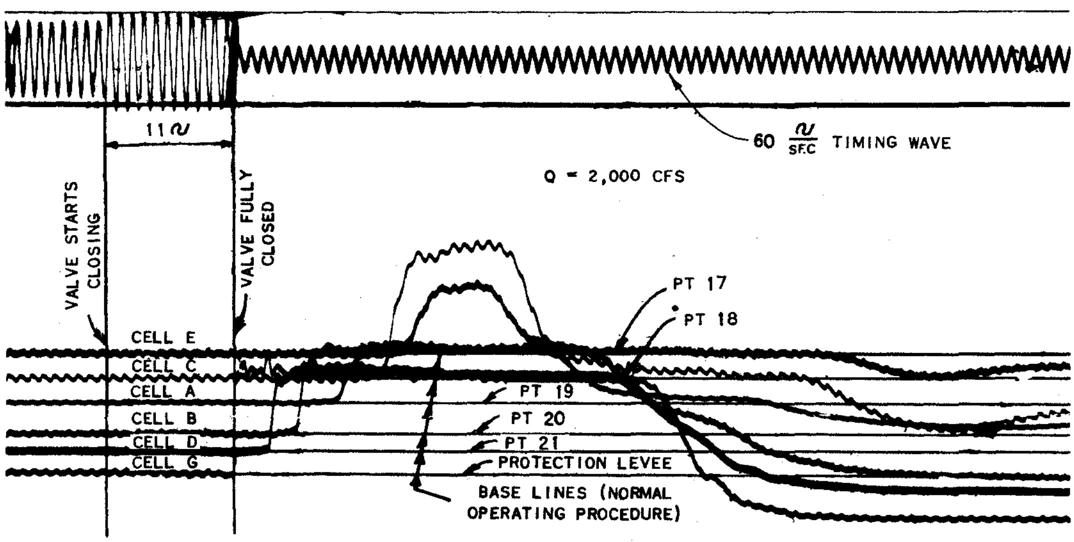
MODEL 1 0 1 2 3 4 5 FT

PLATE 3



NOTE: THE ABOVE STEPPED LINES REPRESENT 0.1 OHM RESISTANCE CHANGE IN THE GAGE CIRCUIT. CORRESPONDING HEAD DIFFERENTIAL IS INDICATED.

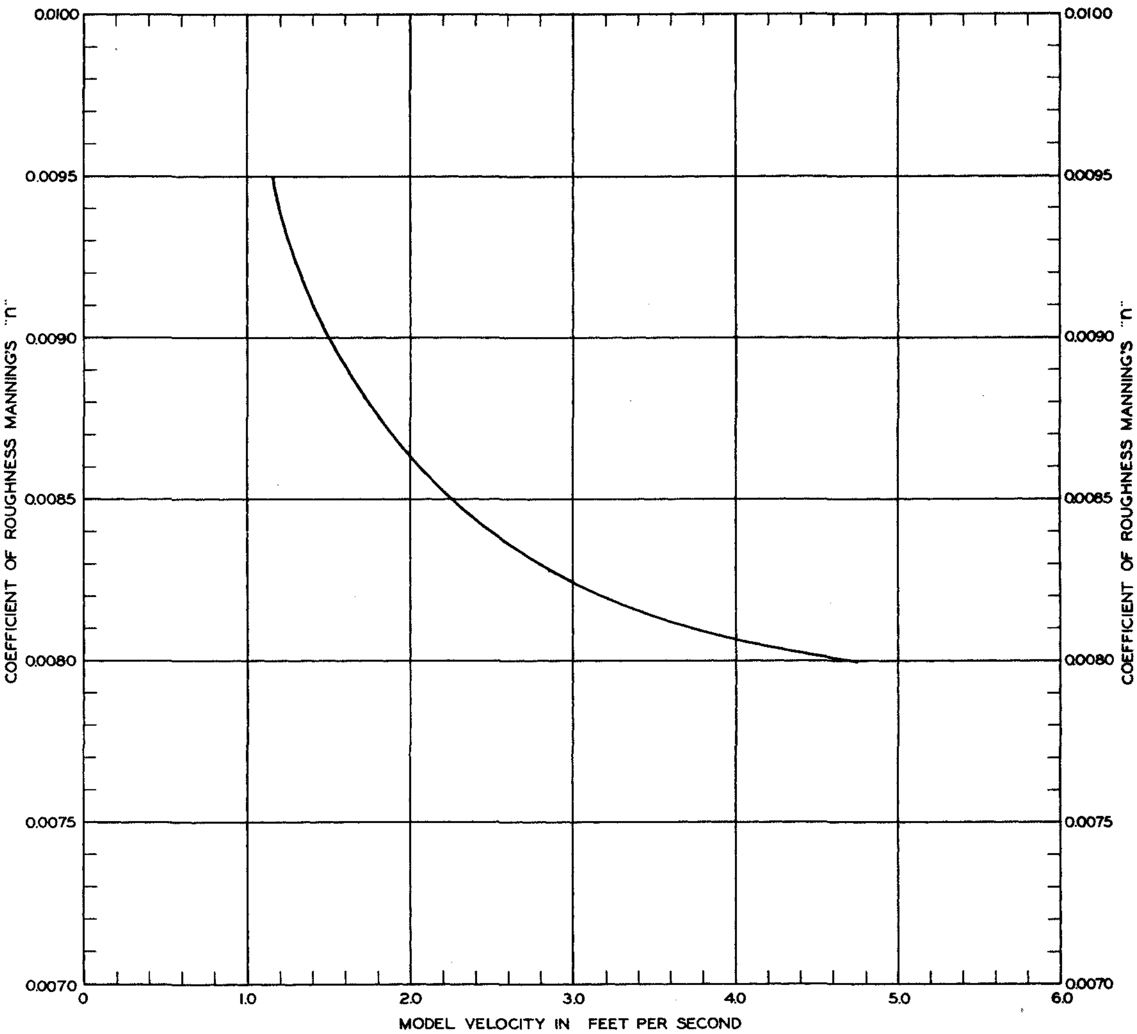
RECORDING OF CALIBRATION OF PRESSURE CELL



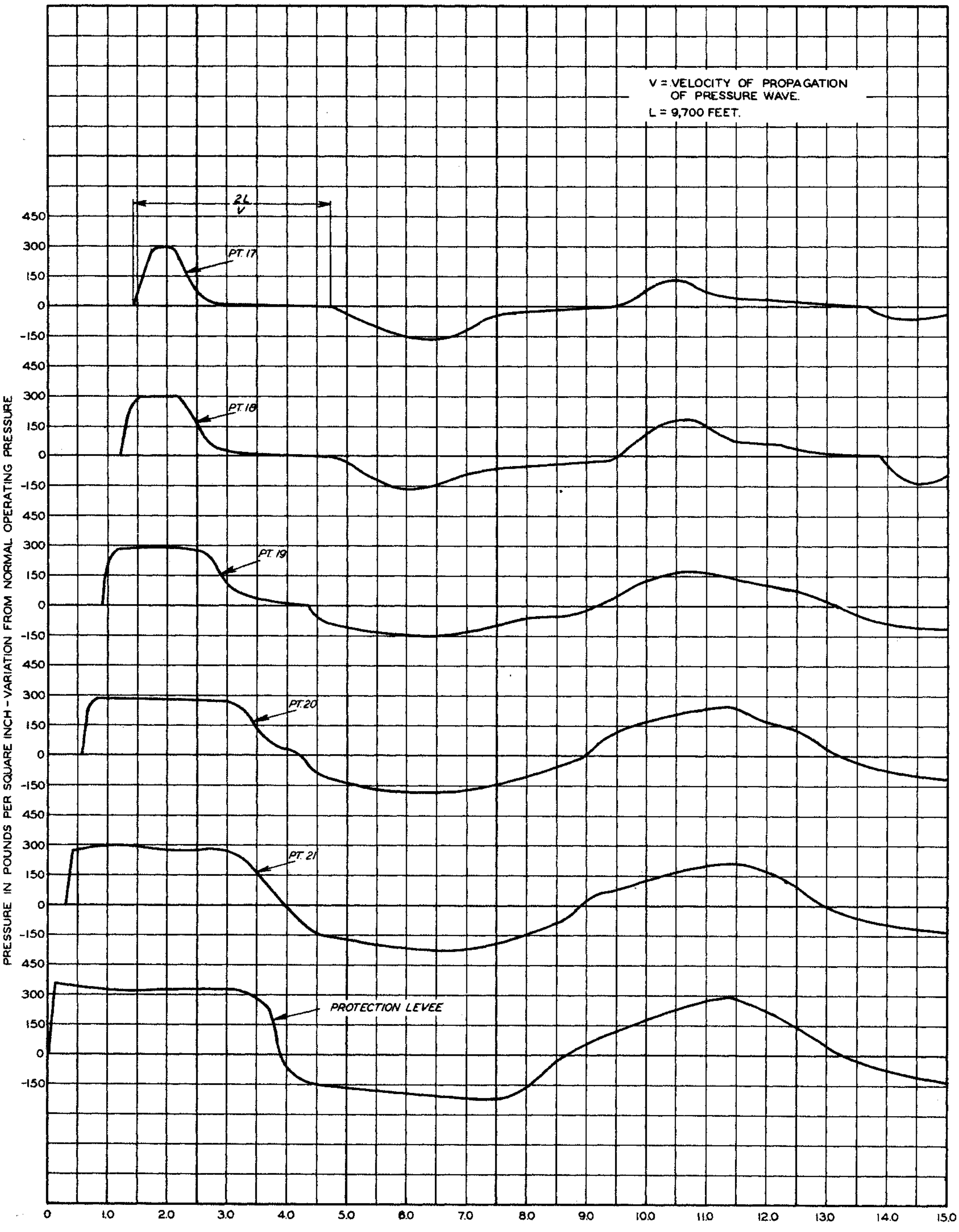
NOTE: LINES ABOVE BASELINE INDICATE NORMAL OPERATING PRESSURE. LINES BELOW BASELINE INDICATE BELOW NORMAL OPERATING PRESSURES.

OSCILLOGRAPH RECORDING FOR CLOSURE OF VALVE

TYPICAL OSCILLOGRAPHS FOR MEASUREMENT OF PRESSURES



COEFFICIENT OF ROUGHNESS
FOR
PLASTIC TUBING



NOTE:

VELOCITY 6.96 FT PER SEC.

TIME OF VALVE CLOSURE 0.944 SEC.

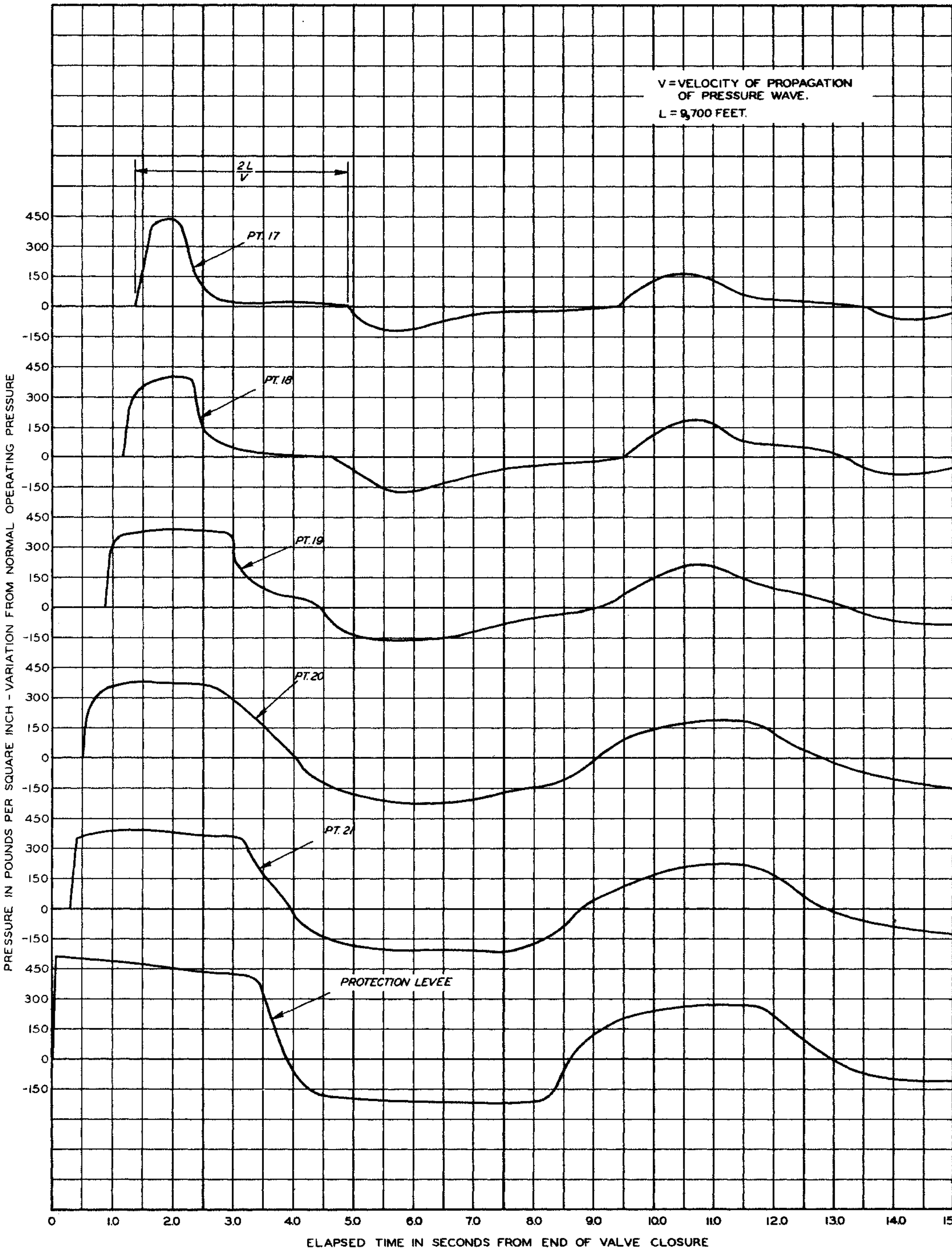
MODEL CONSTRUCTED OF 6-IN. TUBING THROUGHOUT.

NO LATERALS OR RISERS.

ALL VALUES GIVEN IN PROTOTYPE UNITS.

PRESSURE WAVES FOR SUDDEN CLOSURE OF VALVE
6-INCH TUBING
DISCHARGE 1400 CFS

PLATE 6



NOTE:

VELOCITY 8.93 FT PER SEC.

TIME OF VALVE CLOSURE 0.944 SEC.

MODEL CONSTRUCTED OF 6-IN. TUBING THROUGHOUT.

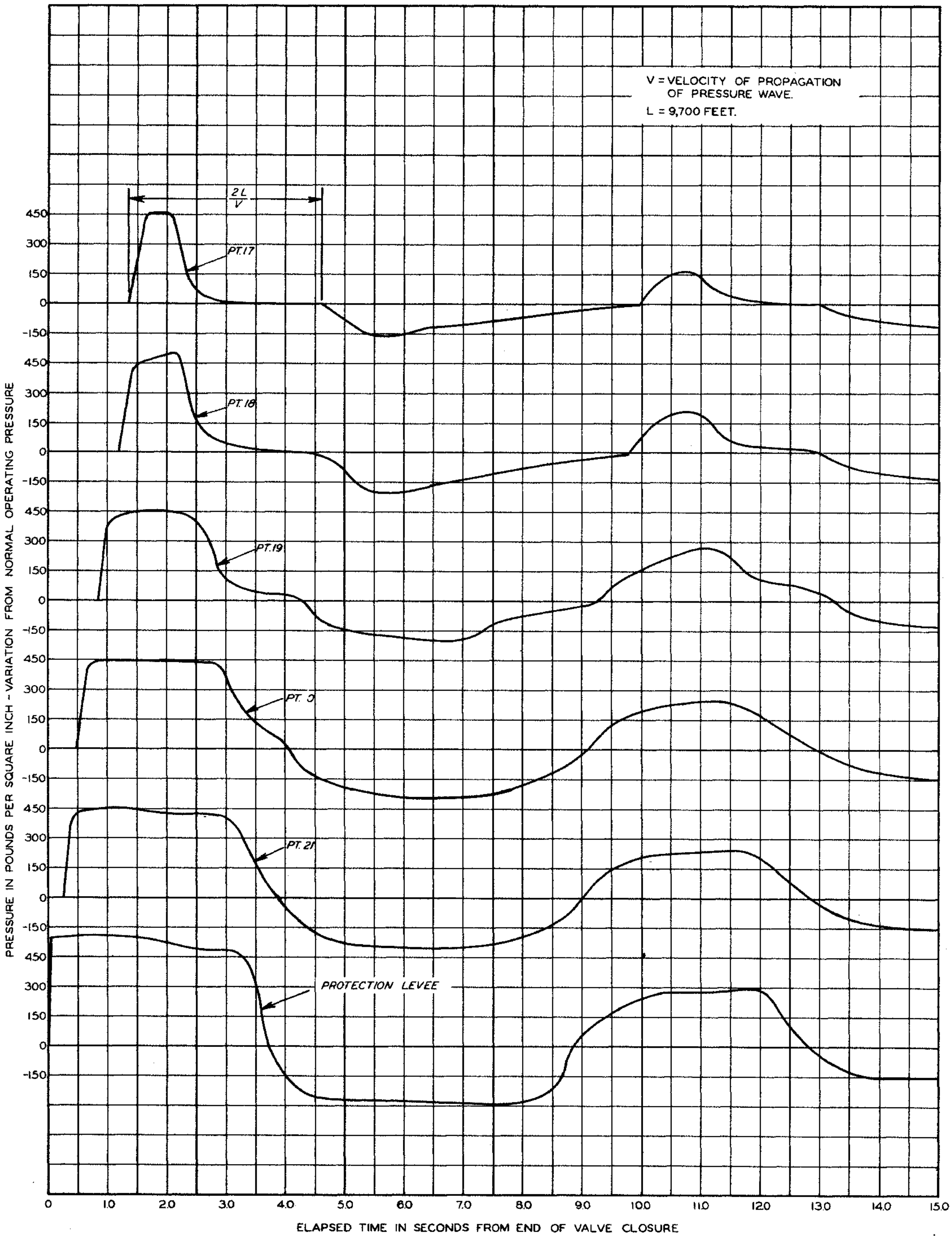
NO LATERALS OR RISERS.

ALL VALUES GIVEN IN PROTOTYPE UNITS.

PRESSURE WAVES FOR SUDDEN CLOSURE OF VALVE

6-INCH TUBING

DISCHARGE 1800 CFS



NOTE:

VELOCITY 9.93 FT PER SEC.

TIME OF VALVE CLOSURE 0.944 SEC.

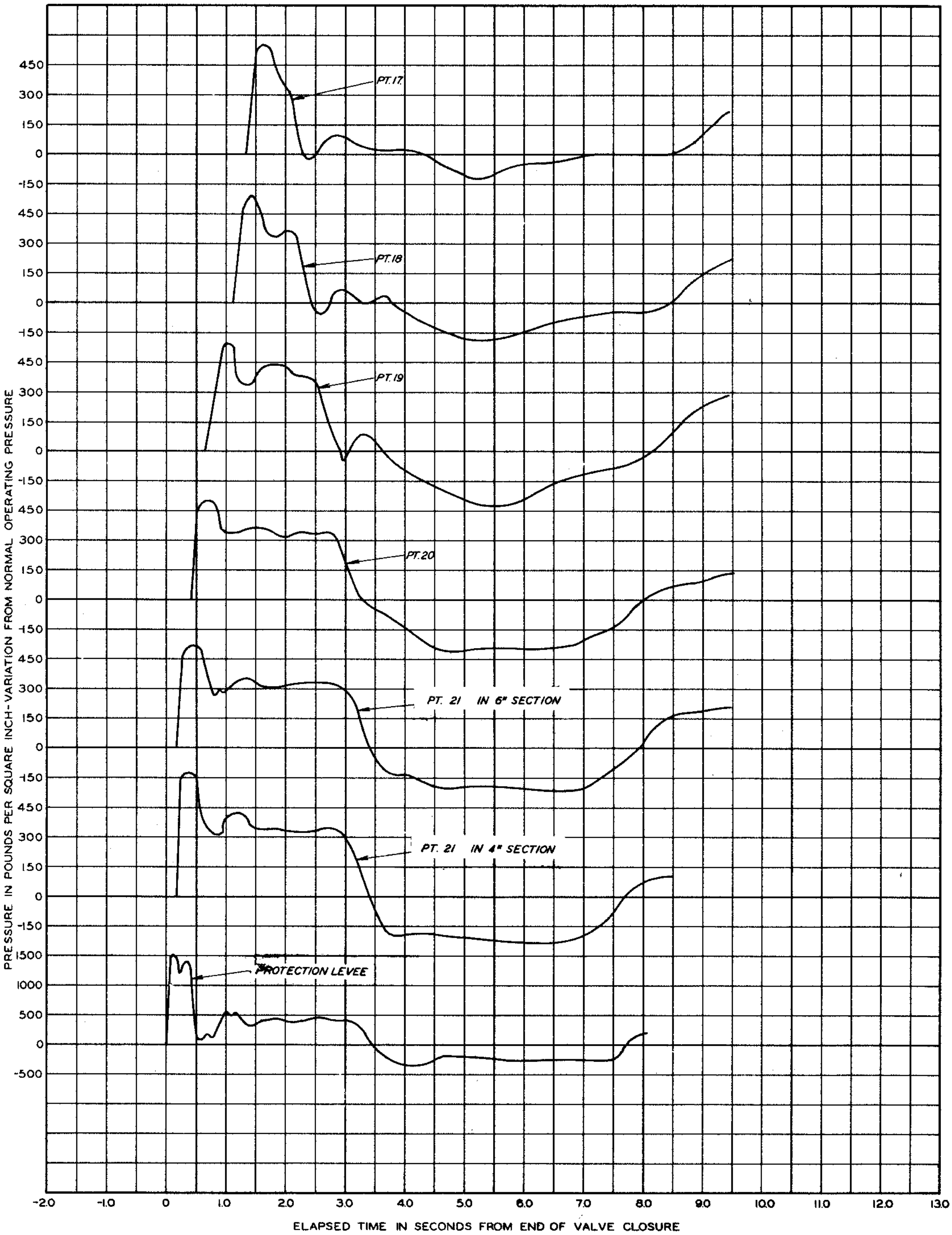
MODEL CONSTRUCTED OF 6-IN. TUBING THROUGHOUT.

NO LATERALS OR RISERS.

ALL VALUES GIVEN IN PROTOTYPE UNITS.

PRESSURE WAVES FOR SUDDEN CLOSURE OF VALVE
6-INCH TUBING
DISCHARGE 2000 CFS

PLATE 8

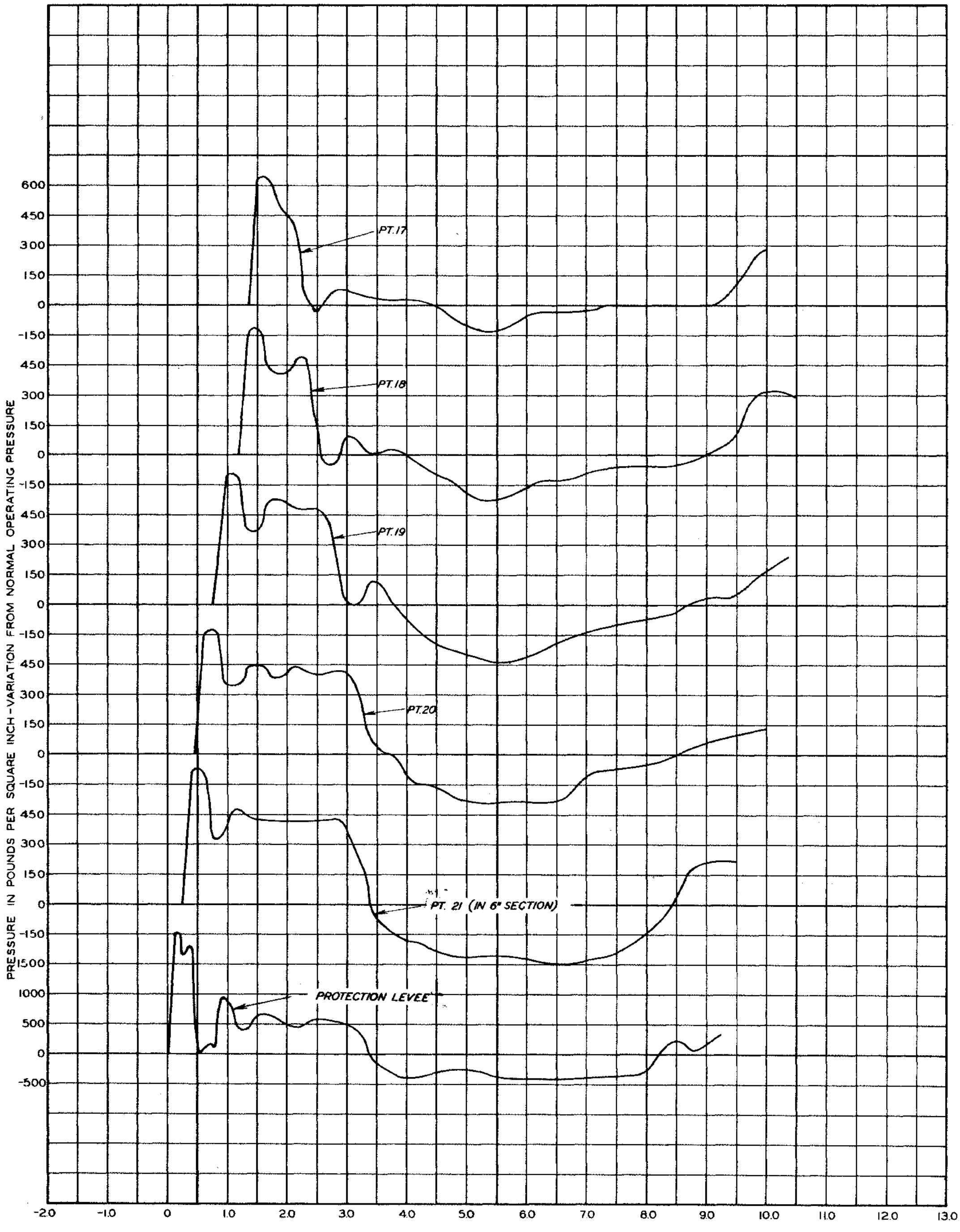


NOTE:

VELOCITY IN 6-IN. SECTION 6.96 FT PER SEC.
VELOCITY IN 4-IN. SECTION 15.67 FT PER SEC.
TIME OF VALVE CLOSURE 0.944 SEC.
MODEL CONSTRUCTED OF 4-IN. AND 6-IN. TUBING.
NO LATERALS OR RISERS.
ALL VALUES GIVEN IN PROTOTYPE UNITS.

PRESSURE WAVES FOR SUDDEN CLOSURE OF VALVE

LOWER SECTION OF 4-INCH TUBING
DISCHARGE 1400 CFS



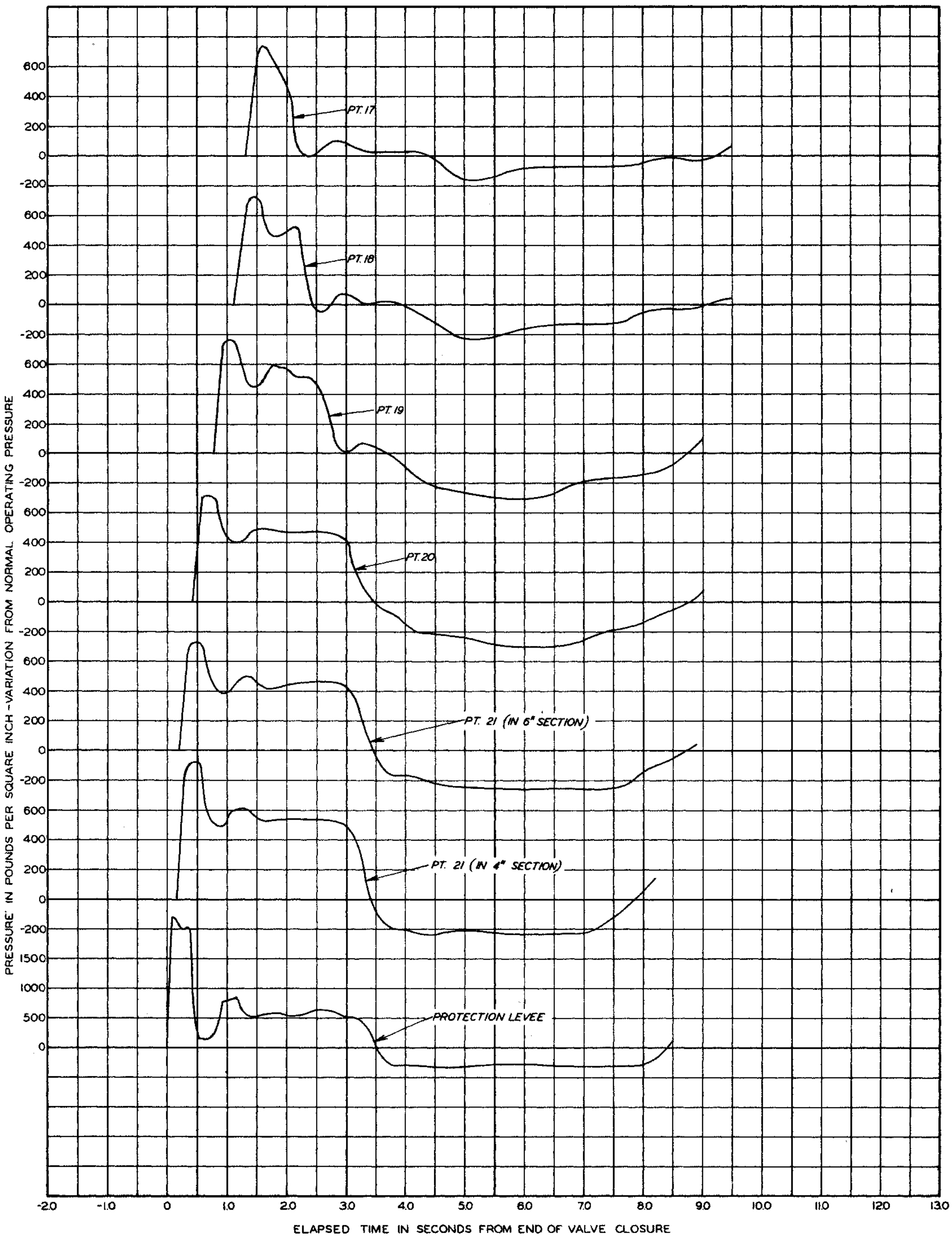
NOTE:

VELOCITY IN 6-IN. SECTION 8.93 FT PER SEC.
VELOCITY IN 4-IN. SECTION 20.09 FT PER SEC.
TIME OF VALVE CLOSURE 0.944 SEC.
MODEL CONSTRUCTED OF 4-IN. AND 6-IN. TUBING.
NO LATERALS OR RISERS.
ALL VALUES GIVEN IN PROTOTYPE UNITS.

PRESSURE WAVES FOR SUDDEN CLOSURE OF VALVE

LOWER SECTION OF 4-INCH TUBING DISCHARGE 1800 CFS

PLATE 10

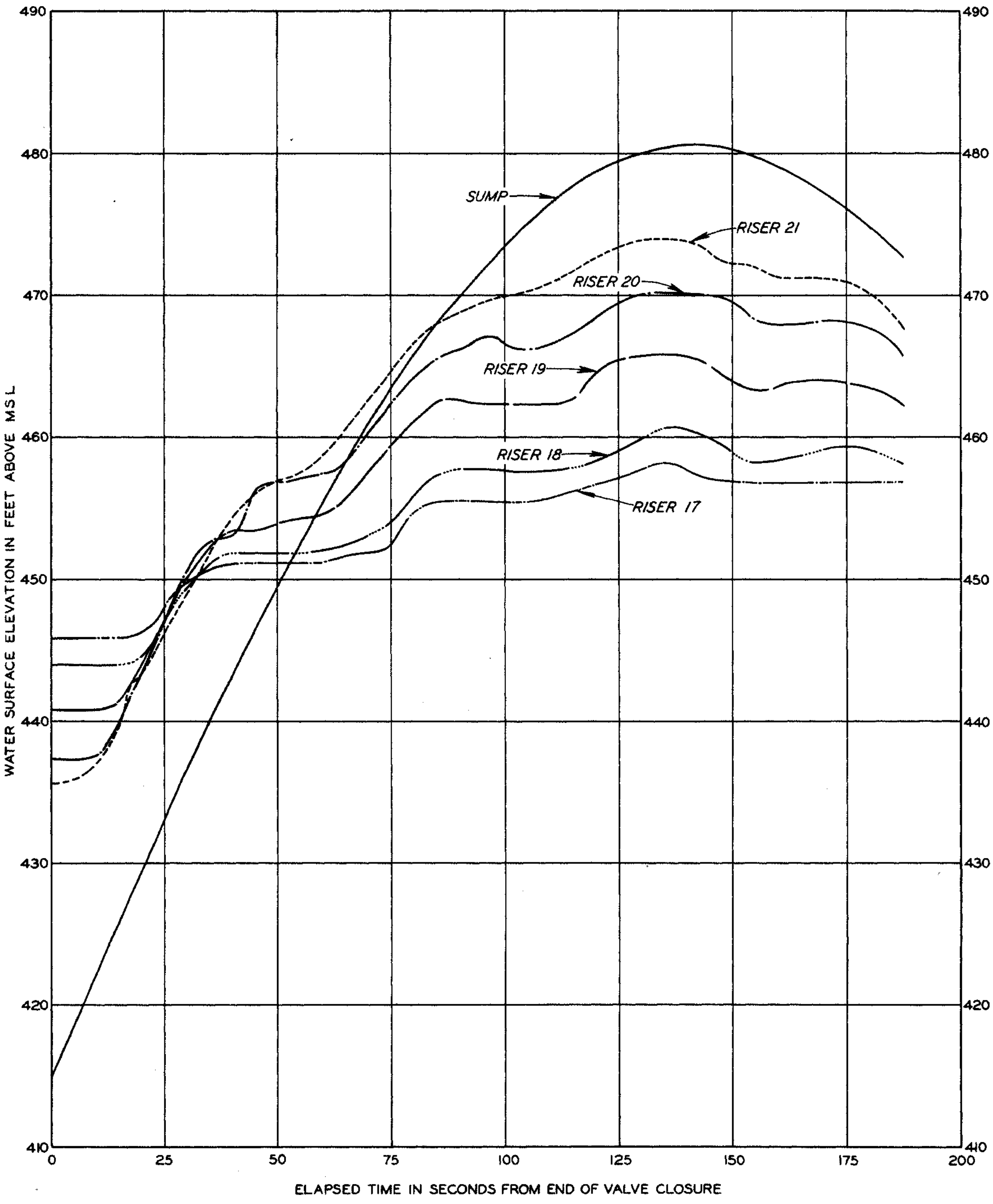


NOTE:

VELOCITY IN 6-IN. SECTION 9.93 FT PER SEC.
VELOCITY IN 4-IN. SECTION 22.35 FT PER SEC.
TIME OF VALVE CLOSURE 0.944 SEC.
MODEL CONSTRUCTED OF 4-IN. AND 6-IN. TUBING.
NO LATERALS OR RISERS.
ALL VALUES GIVEN IN PROTOTYPE UNITS.

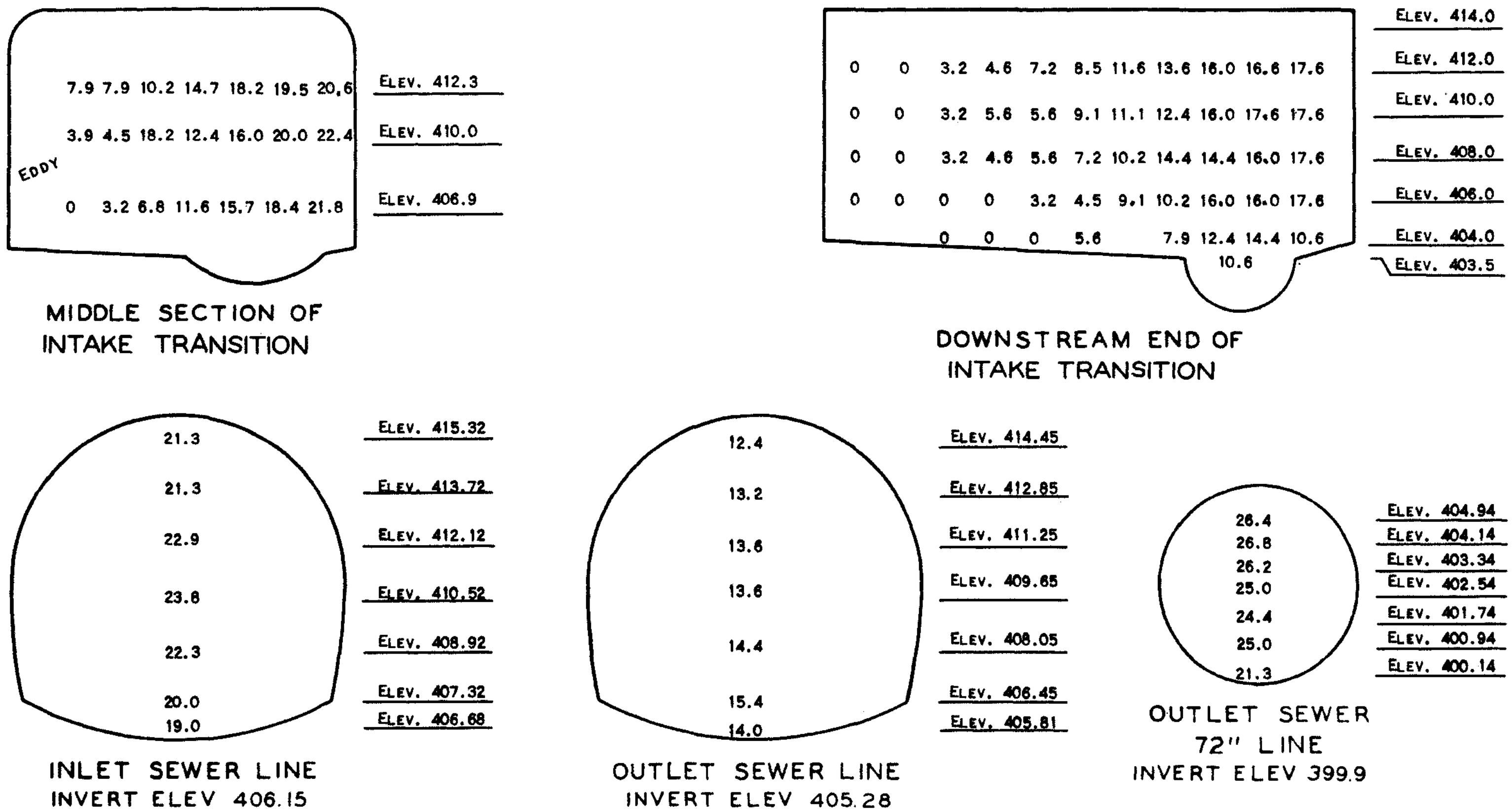
PRESSURE WAVES FOR SUDDEN CLOSURE OF VALVE

LOWER SECTION OF 4-INCH TUBING
DISCHARGE 2000 CFS



NOTE: DISCHARGE 1 800 C F S
SUMP AREA 2 100 SQUARE FEET.
ALL VALUES GIVEN IN PROTOTYPE UNITS.
THE SUMP WATER SURFACE ELEVATION WAS 415.0
WHEN VALVE WAS CLOSED.

SURGES IN SEWER LINE



NOTE: ALL MEASUREMENTS ARE IN THEIR PROTOTYPE EQUIVALENTS.

TEST CONDITIONS

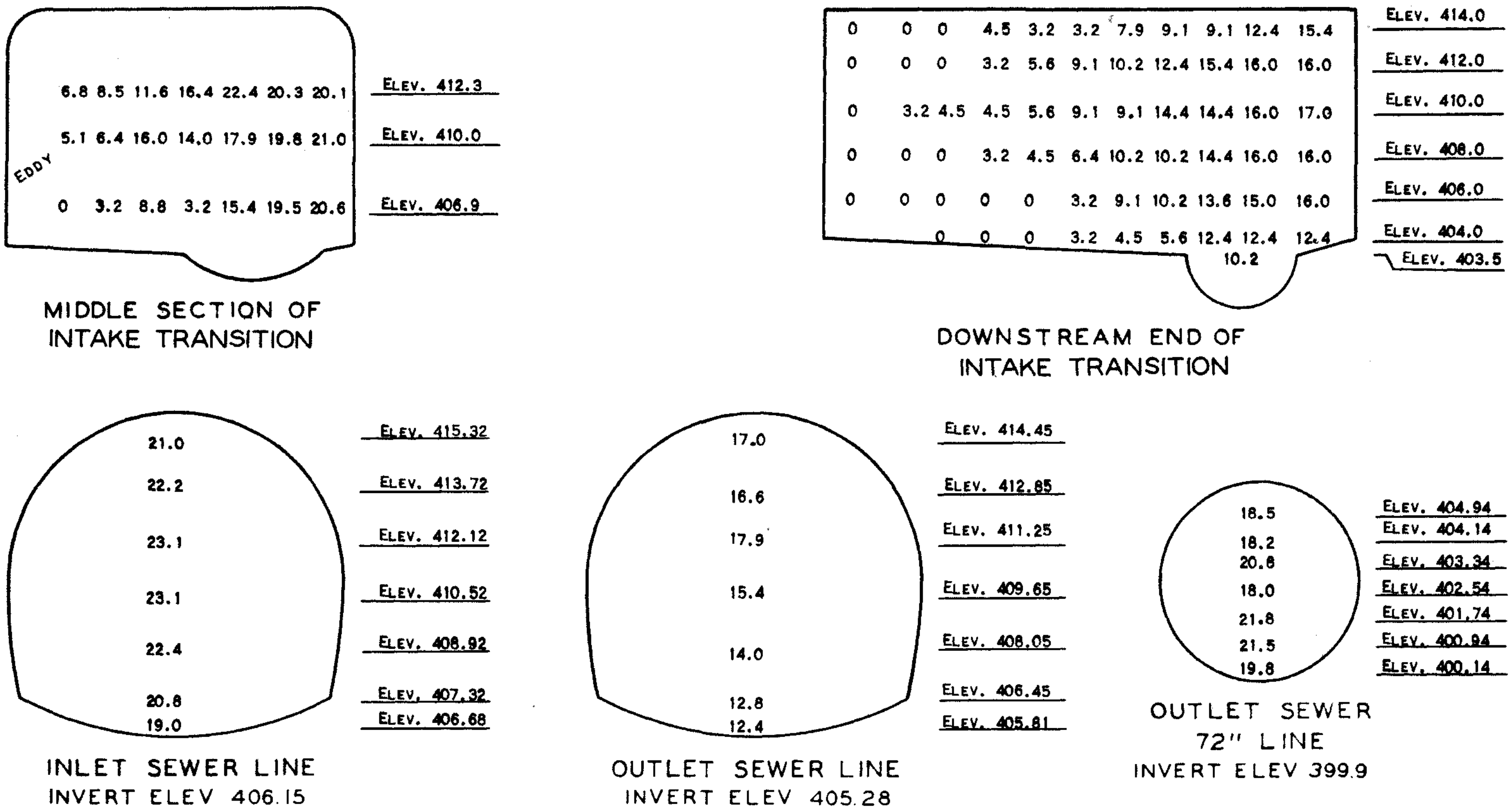
DISCHARGE	1800 CFS
SUMP ELEV	422.0

VELOCITY CROSS SECTIONS

ORIGINAL CONDITIONS

GRAVITY FLOW

PLATE 14



NOTE: ALL MEASUREMENTS ARE IN THEIR PROTOTYPE EQUIVALENTS.

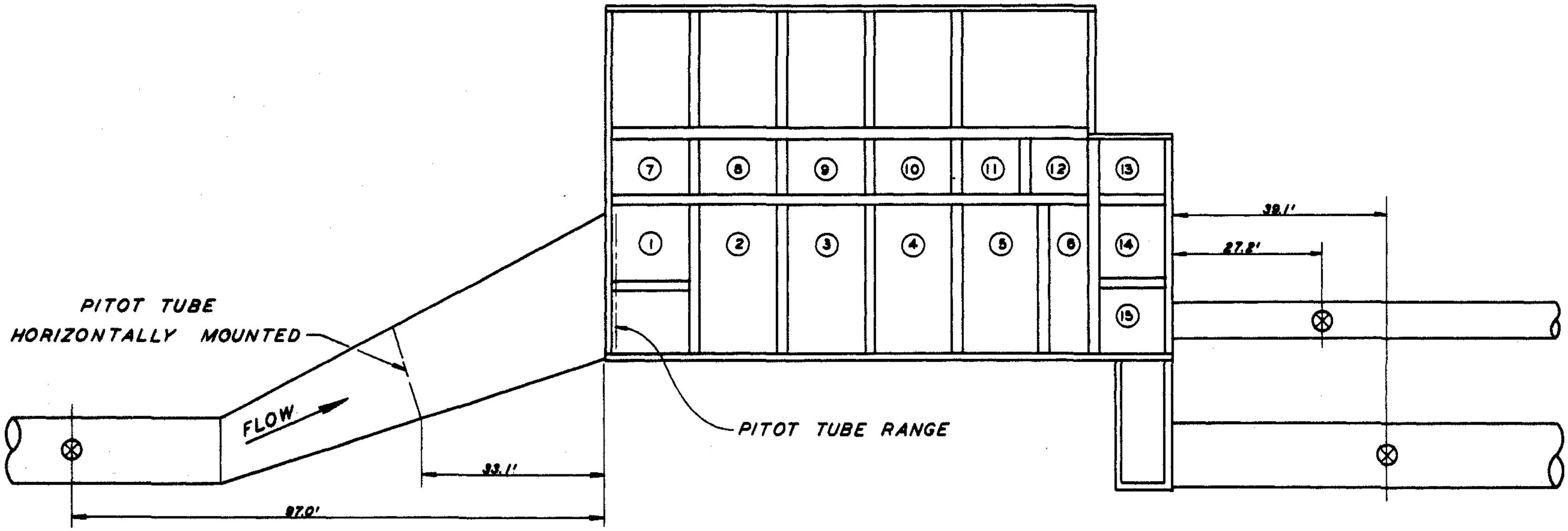
TEST CONDITIONS

DISCHARGE	1800 CFS
SUMP ELEV	420.0
RIVER STAGE	430.0

VELOCITY CROSS SECTIONS

ORIGINAL CONDITIONS

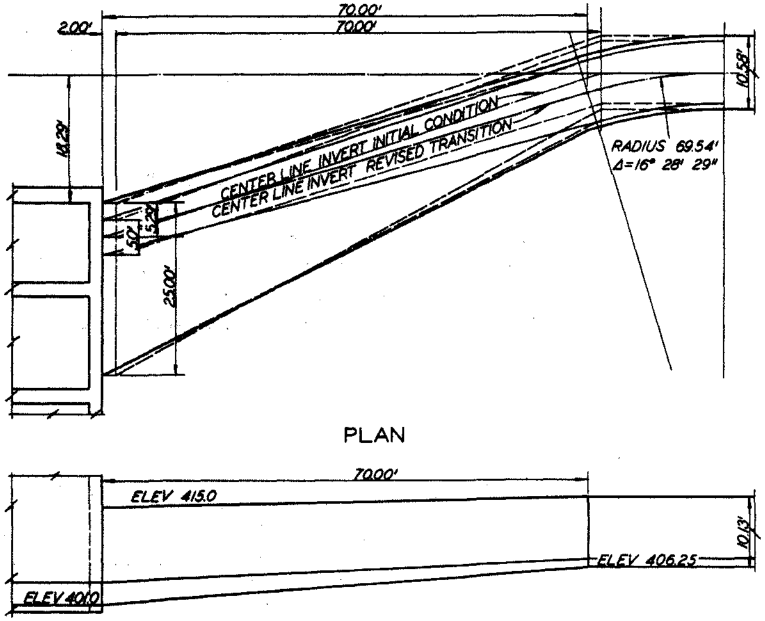
PUMP FLOW



NOTE: COMPARTMENT NUMBERS ARE FOR ORIGINAL AND REVISED CONDITIONS
PITOT TUBE LOCATIONS ARE FOR ORIGINAL AND REVISED INTAKE AND OUTLET TRANSITIONS
ALL VALUES ARE IN PROTOTYPE UNITS
⊗ PITOT TUBE VERTICALLY MOUNTED

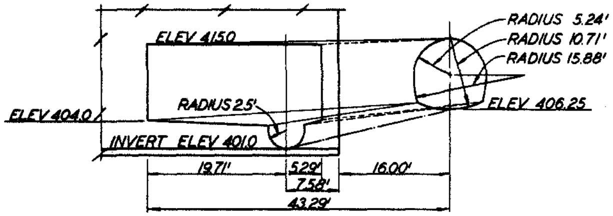
COMPARTMENT NUMBERS
AND
PITOT TUBE LOCATIONS

PLATE 16



PLAN

ELEVATION

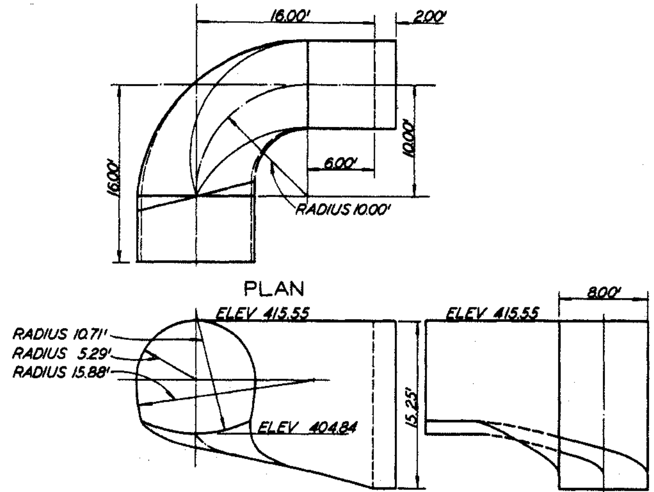


END VIEW OF INTAKE

REVISED INTAKE TRANSITION

SCALE

10 0 10 20 30 FT



PLAN

ELEVATION

END VIEW

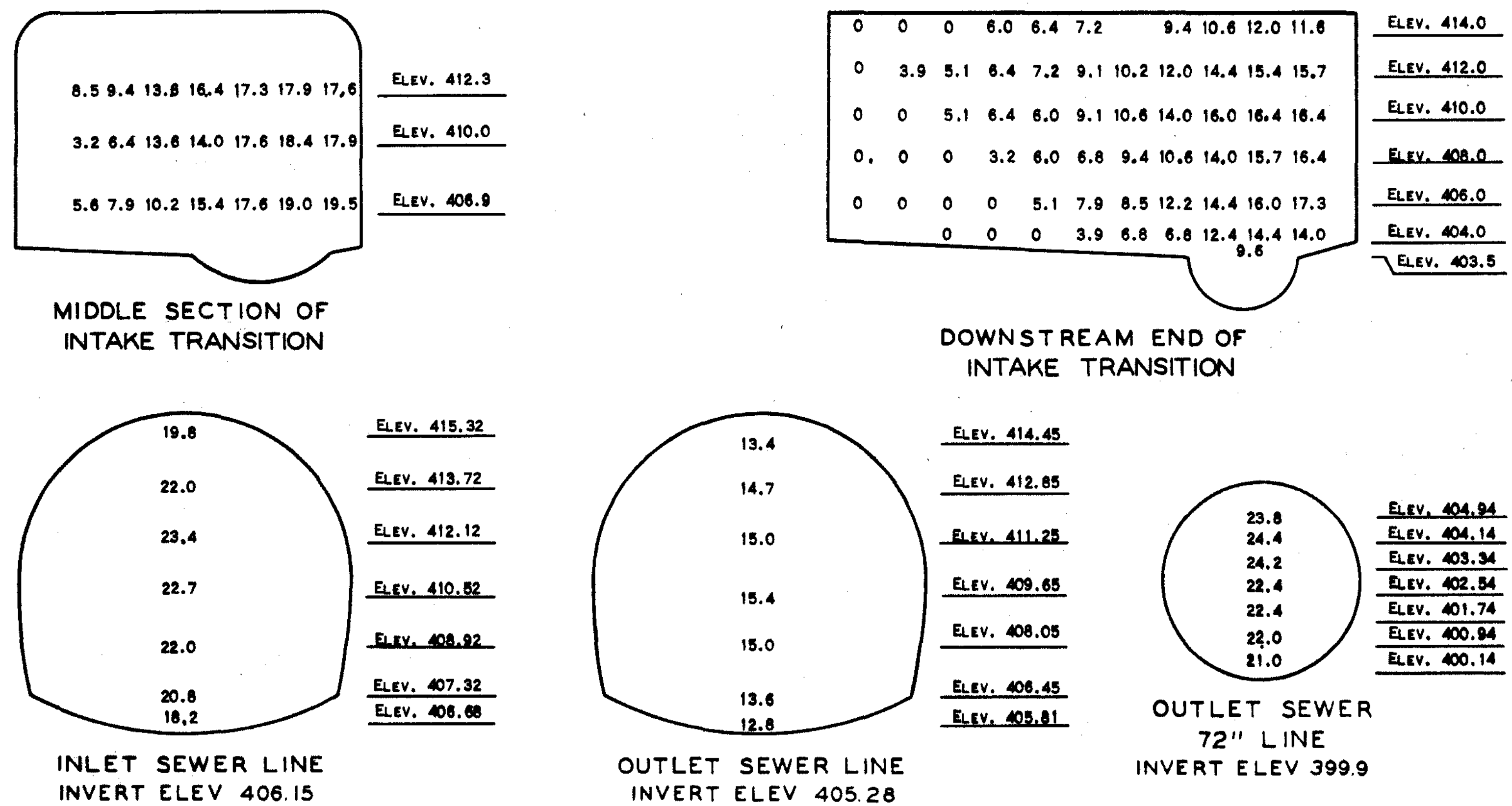
REVISED OUTLET TRANSITION

SCALE

5 0 5 10 15 FT

NOTE: ALL UNITS ARE IN THEIR PROTOTYPE EQUIVALENTS.

REVISED TRANSITIONS INTAKE AND OUTLET

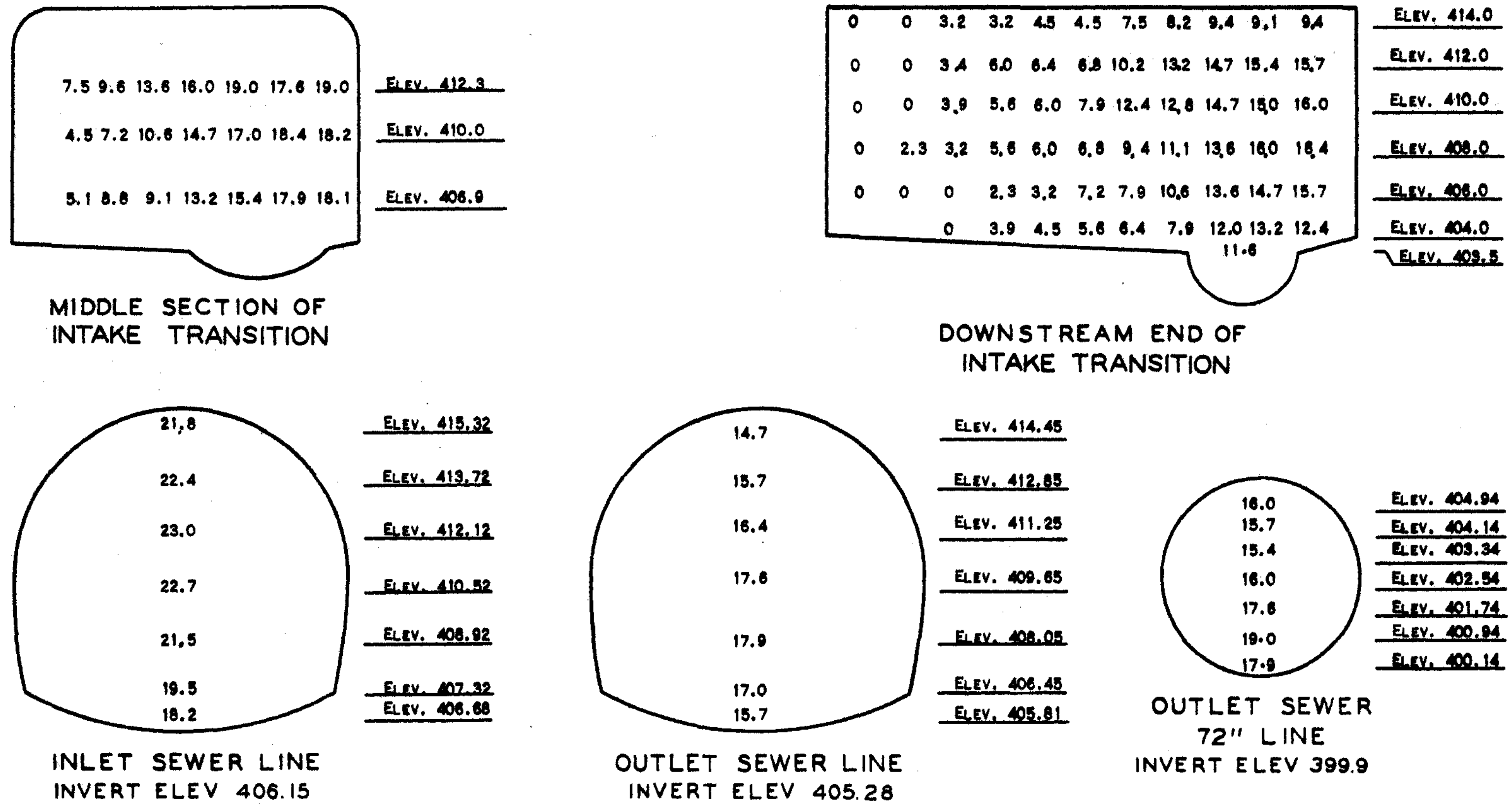


NOTE: ALL MEASUREMENTS ARE IN THEIR PROTOTYPE EQUIVALENTS.

TEST CONDITIONS

DISCHARGE 1800 CFS
SUMP ELEV 420.0

VELOCITY CROSS SECTIONS
REVISED INTAKE AND OUTLET
GRAVITY FLOW

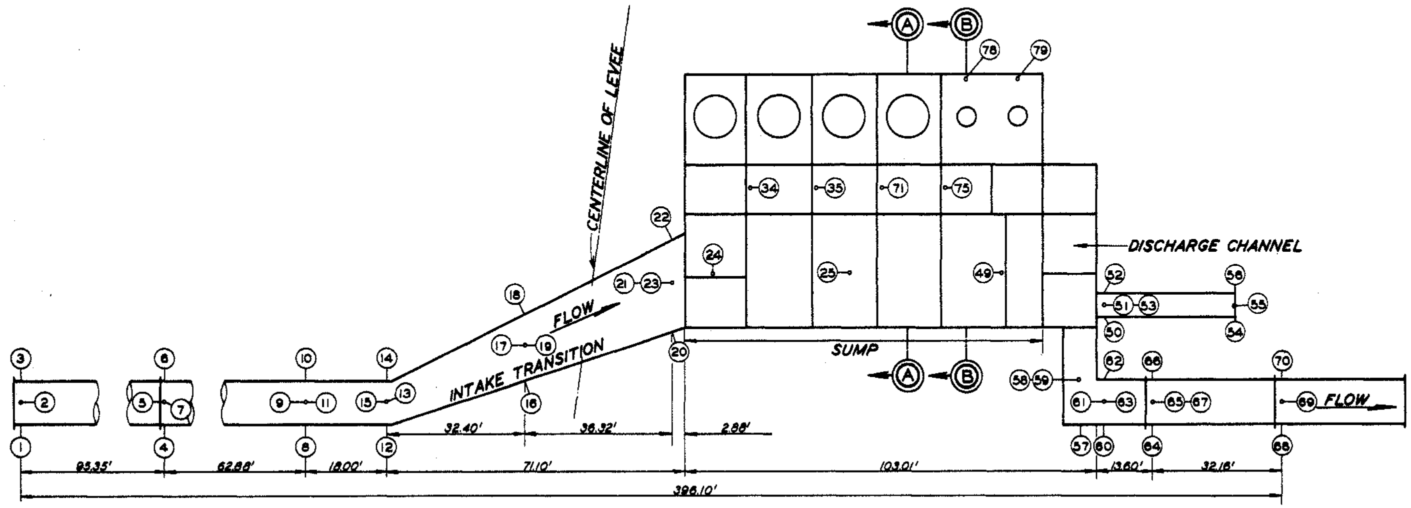


NOTE: ALL MEASUREMENTS ARE IN THEIR PROTOTYPE EQUIVALENTS.

TEST CONDITIONS

DISCHARGE	1800 CFS
SUMP ELEV	420.0
RIVER STAGE	430.0

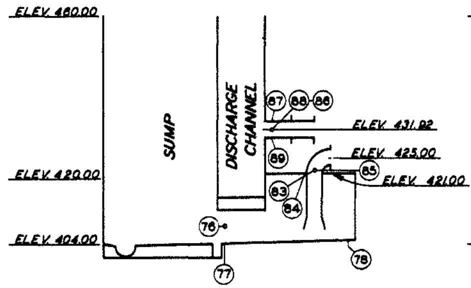
VELOCITY CROSS SECTIONS
REVISED INTAKE AND OUTLET
PUMP FLOW



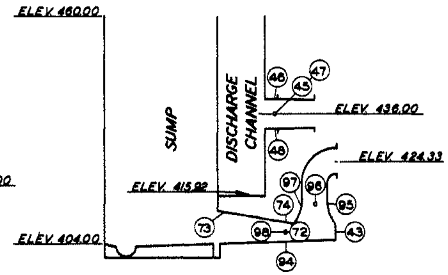
PLAN

PIEZOMETER NUMBER	PIEZOMETER ZERO
1	411.42
2	416.78
3	411.42
4	410 90
5	416.71
6	410.90
7	406.58
8	408.16
9	416.19
10	406.16
11	406.33
12	412.78
13	416.19
14	412.78
15	406.25
16	410.69
17	415.80
18	410.69
19	404.90
20	409.74
21	415.31
22	409.74
23	403.78
24	403.98
25	403.74
34	416.00
35	415.80
43	406.75
45	436.08
46	439.68
47	436.08
48	432.48
49	403.70
50	403.22

PIEZOMETER NUMBER	PIEZOMETER ZERO
51	406.18
52	403.22
53	400.18
54	402.48
55	407.26
56	402.91
57	411.26
58	415.55
59	403.90
60	410.60
61	415.81
62	410.50
63	405.17
64	410.22
65	415.76
66	410.29
67	405.13
68	407.36
69	415.81
70	407.52
71	415.65
72	406.75
73	411.76
74	408.88
75	415.38
76	407.68
77	404.00
78	405.06
79	405.06
83	422.37
84	422.37
85	422.37
86	431.92
87	433.92
88	431.92
89	431.92
94	404.25
95	415.33
96	415.33
97	415.33
98	406.75



SECTION B-B

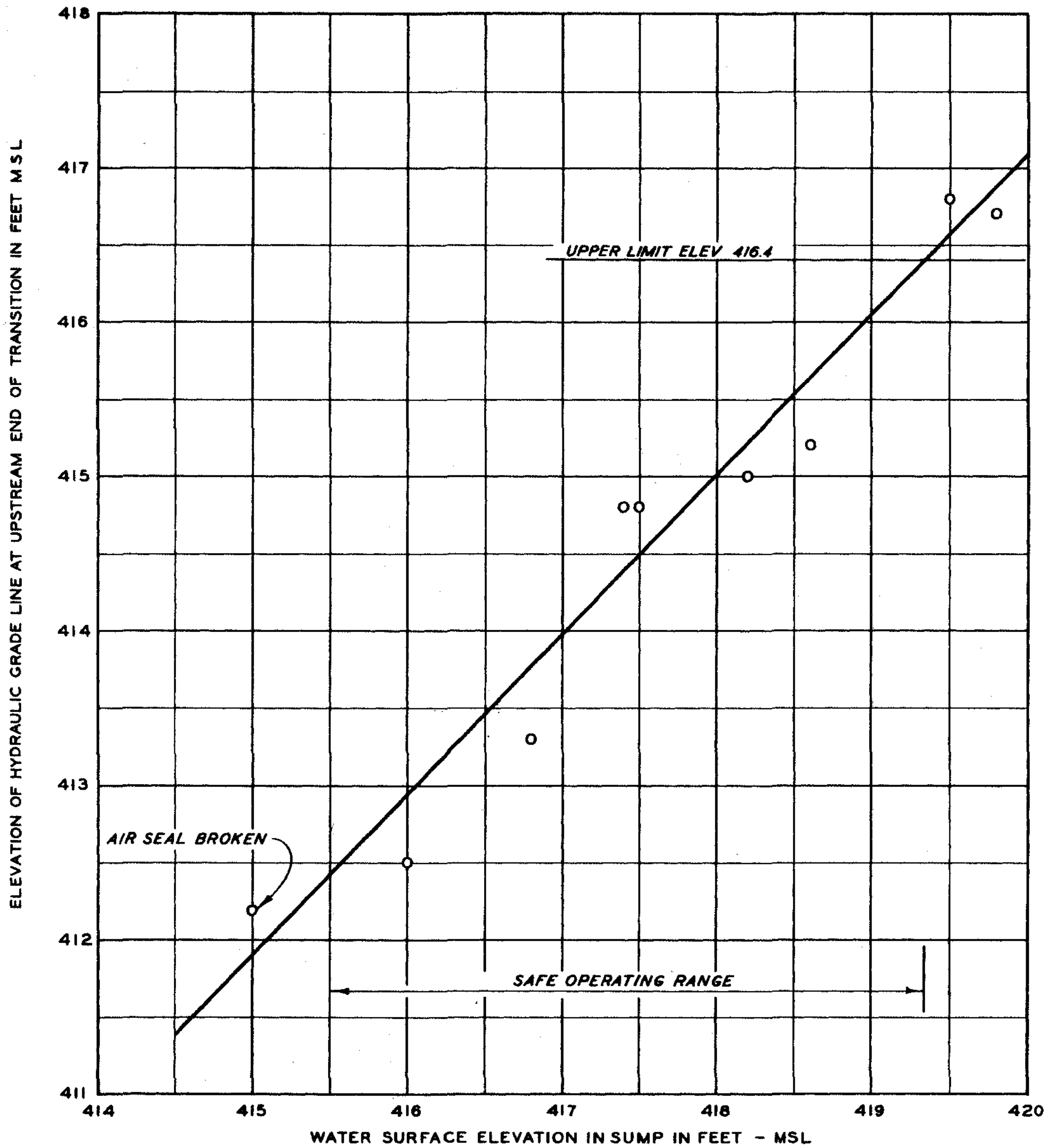


SECTION A-A

NOTE: ALL VALUES ARE IN PROTOTYPE UNITS.
PIEZOMETERS SHOWN THUS ①→②
NUMBER ON LEFT INDICATES NEAR PIEZOMETER.

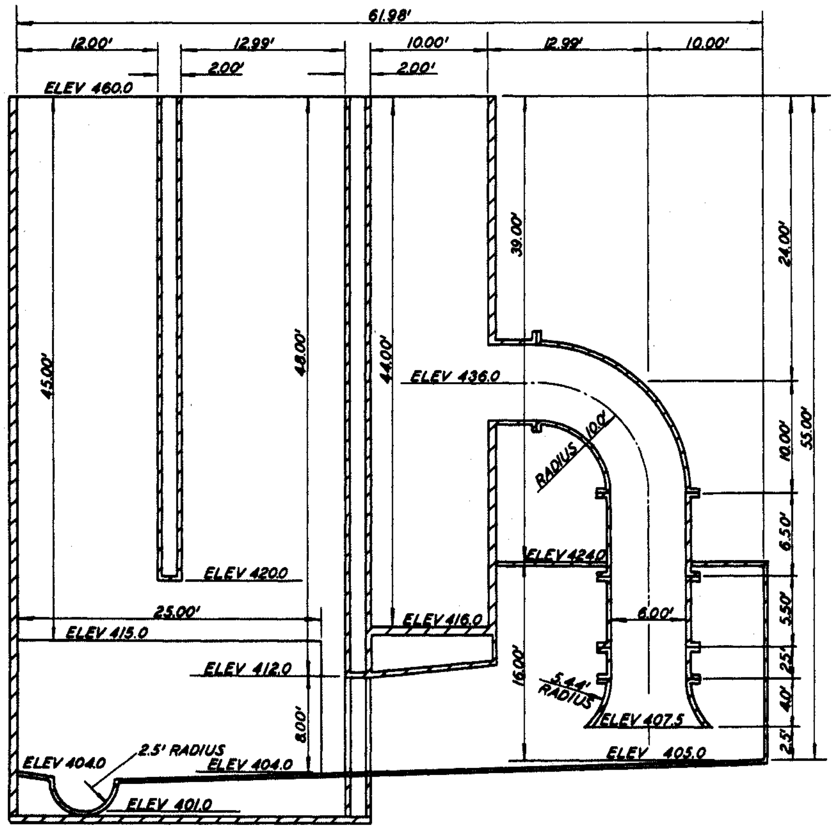
PIEZOMETER LOCATIONS
ORIGINAL CONDITIONS

SCALES

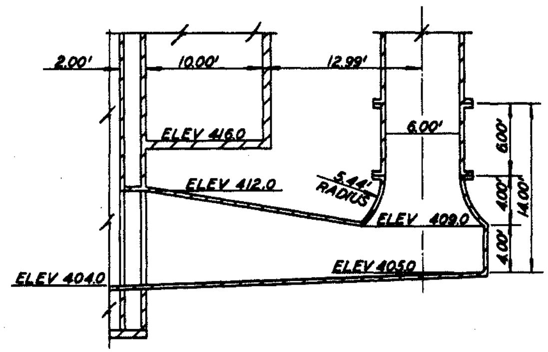


NOTE: DISCHARGE 1800 CFS.
ALL PUMPS OPERATING.

EFFECT OF SUMP ELEVATION ON HYDRAULIC GRADE LINE AT ENTRANCE TO PUMPING PLANT



OPEN PIT



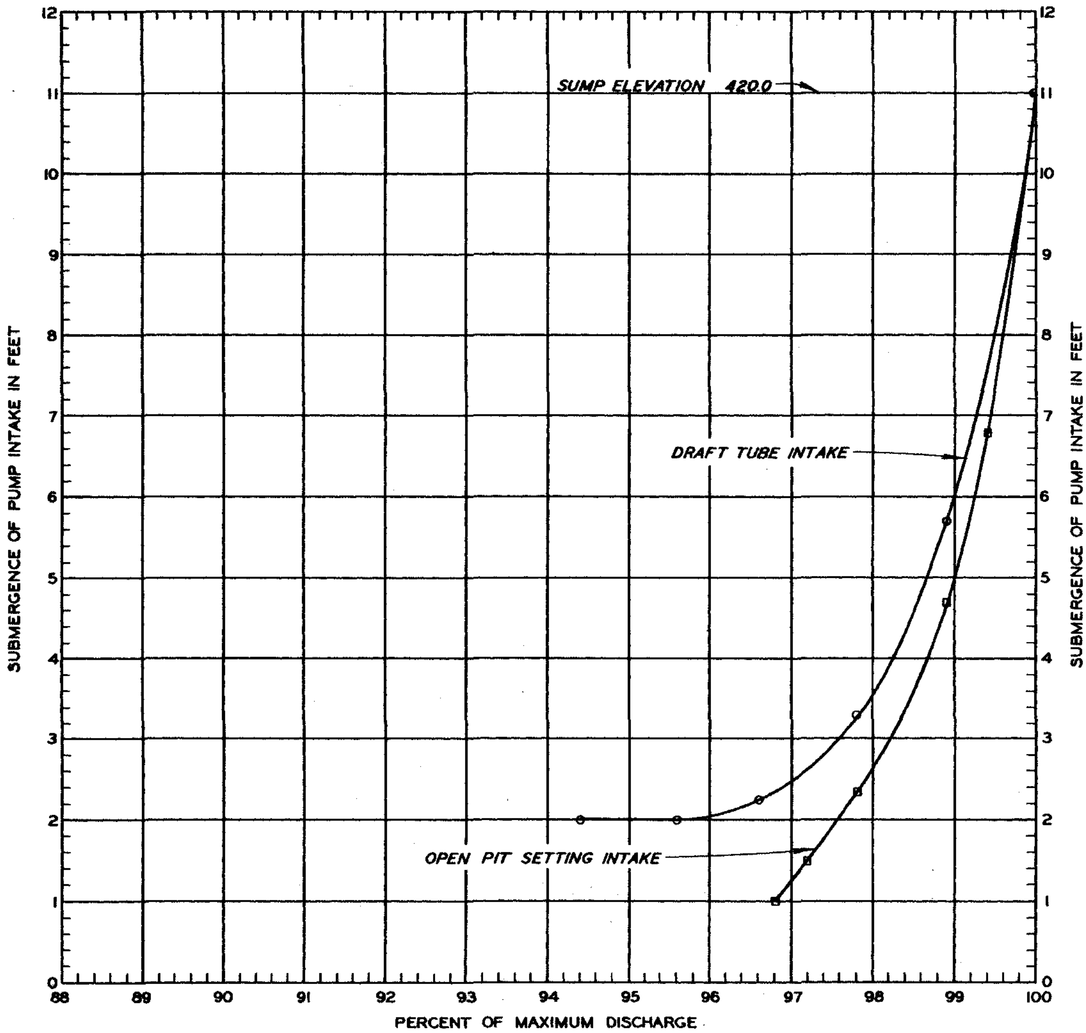
DRAFT TUBE

NOTE: ALL UNITS ARE IN THEIR PROTOTYPE EQUIVALENTS.

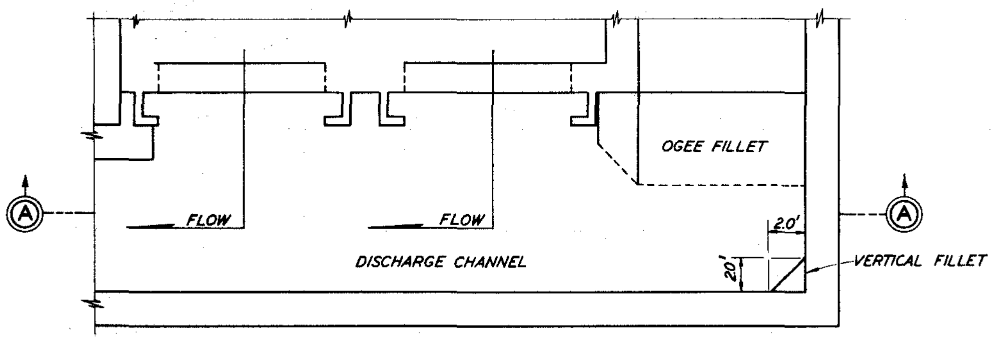
DETAILS OF PUMP INTAKES

SCALE

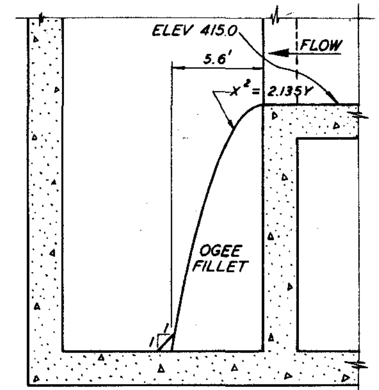
10 0 10 20 30 FT



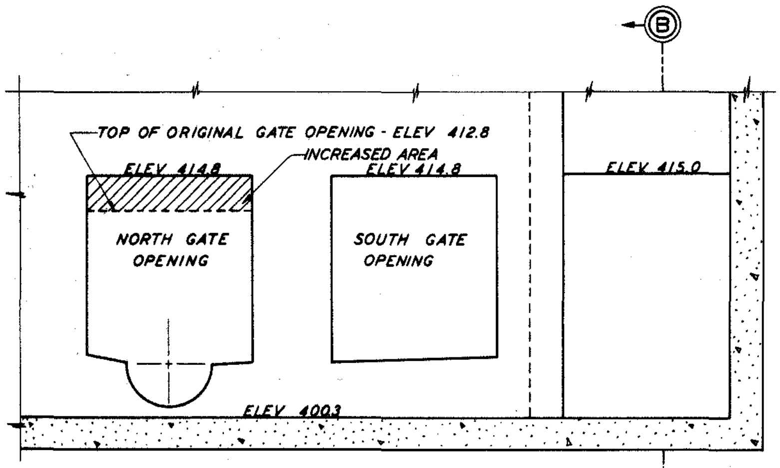
EFFECT OF TYPE PUMP INTAKE
ON CAPACITY



PLAN



SECTION B-B



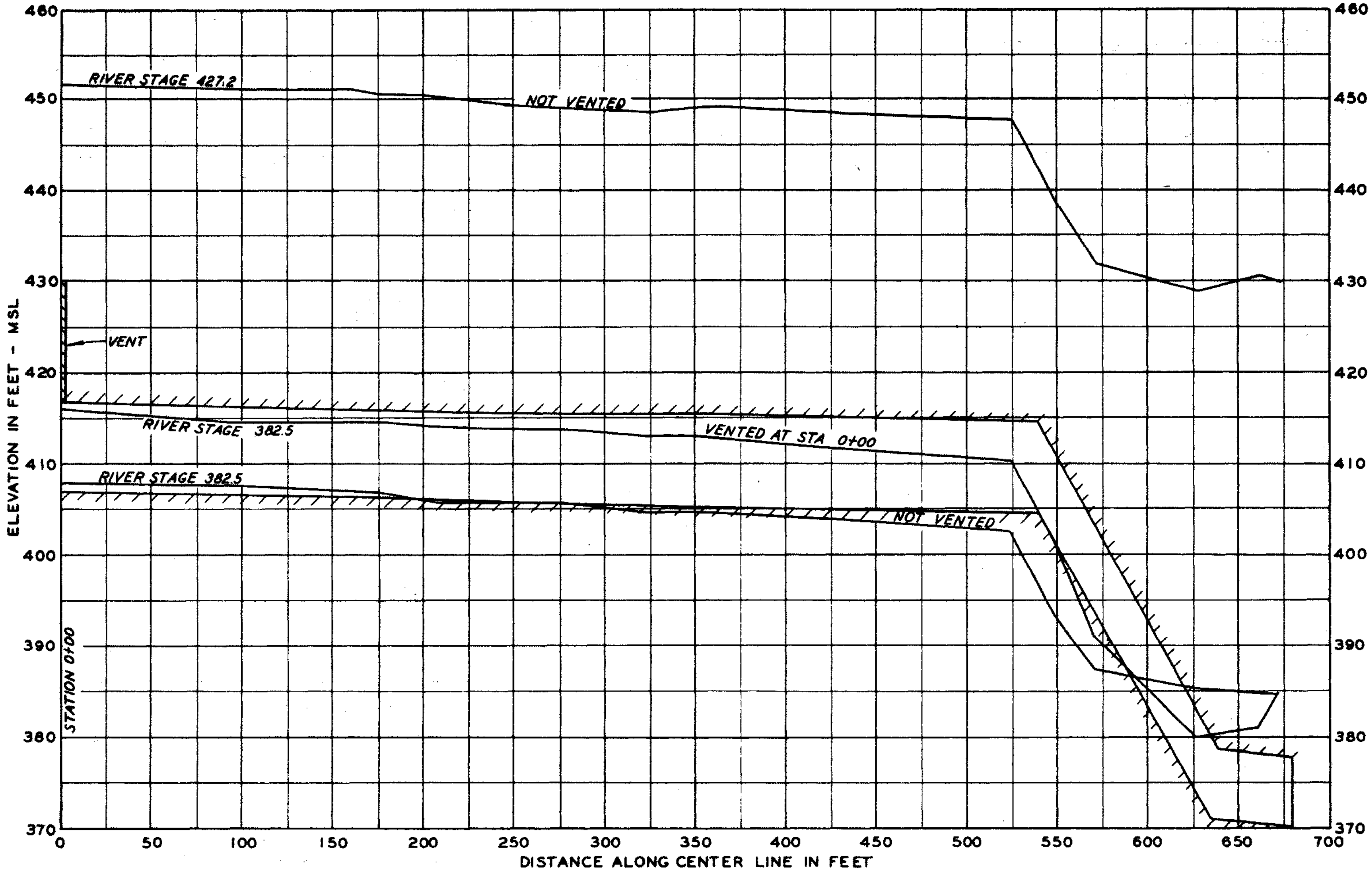
SECTION A-A

DETAILS OF SLUICE GATES AND REVISIONS TO DISCHARGE CHANNEL

SCALE

20 0 20FT

PLATE 23

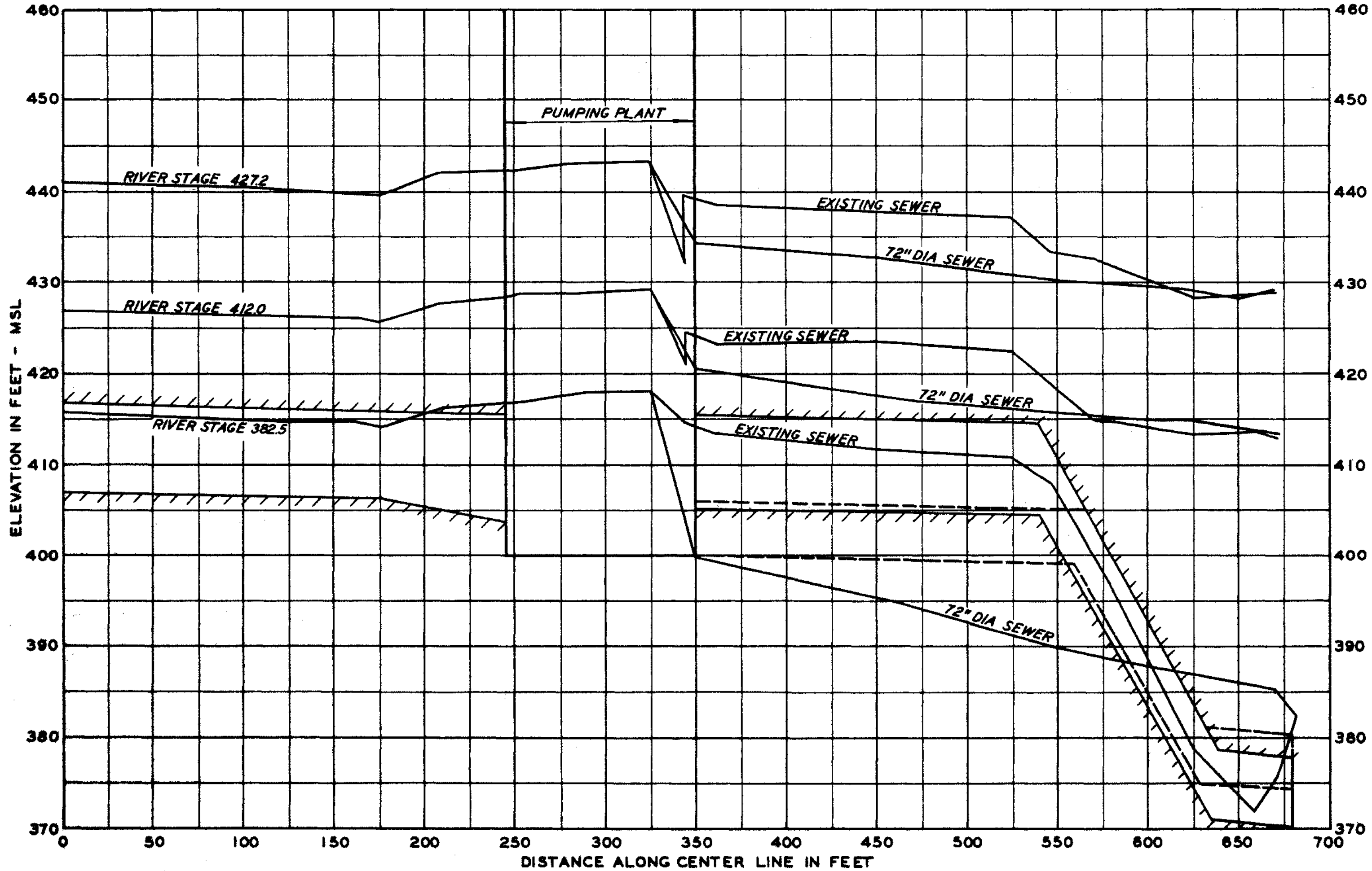


TEST CONDITIONS

DISCHARGE 1800 CFS

NOTE: STATION 0+00 LOCATED 247.34 FT UPSTREAM FROM PROPOSED PUMPING PLANT.
EXISTING CONDITIONS DENOTE PUMPING PLANT NOT INSTALLED.

HYDRAULIC GRADE LINES
EXISTING CONDITIONS
GRAVITY FLOW



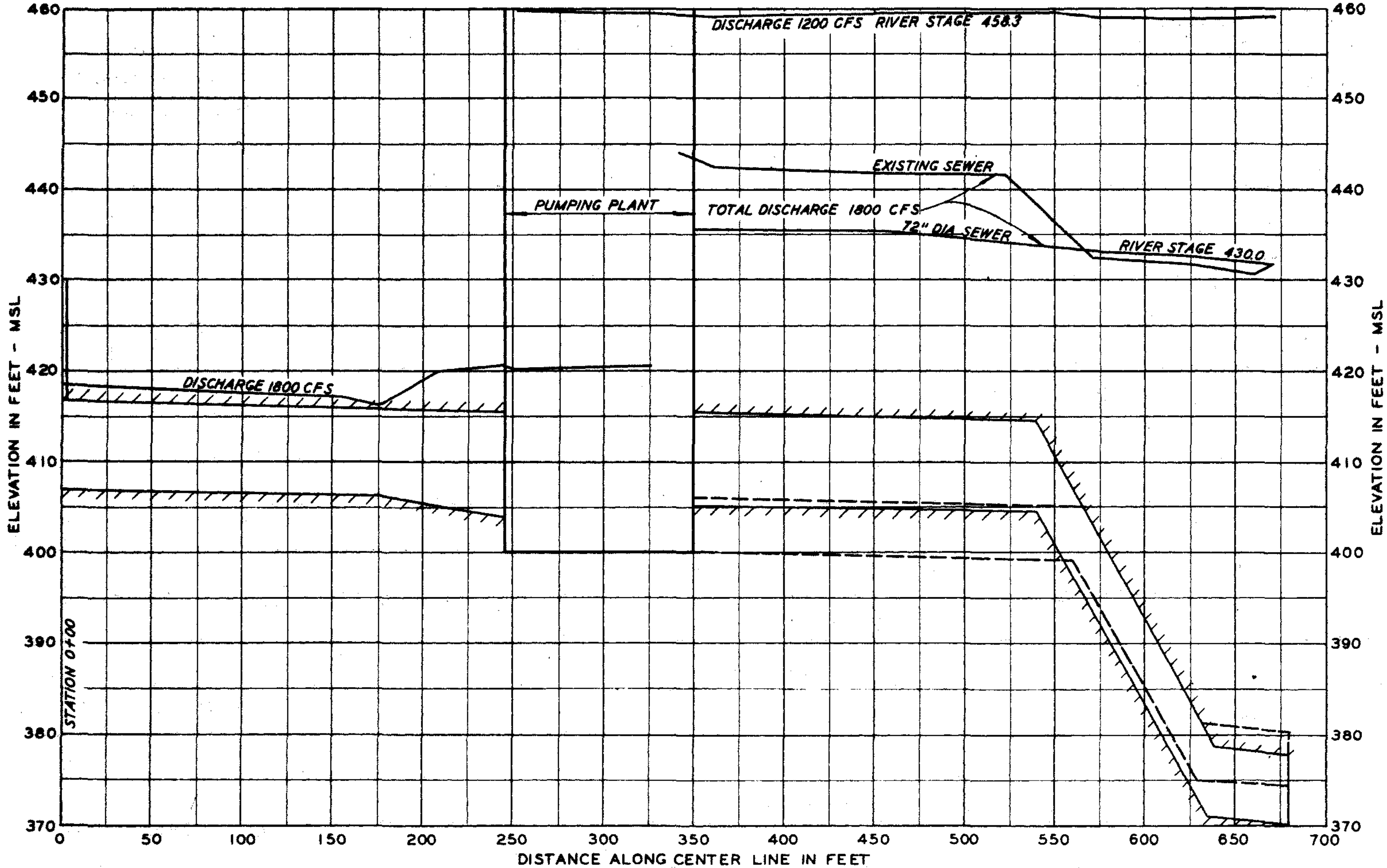
TEST CONDITIONS

DISCHARGE 1800 CFS
NOT VENTED

NOTE: INITIAL CONDITIONS DENOTE PUMPING PLANT INSTALLED.

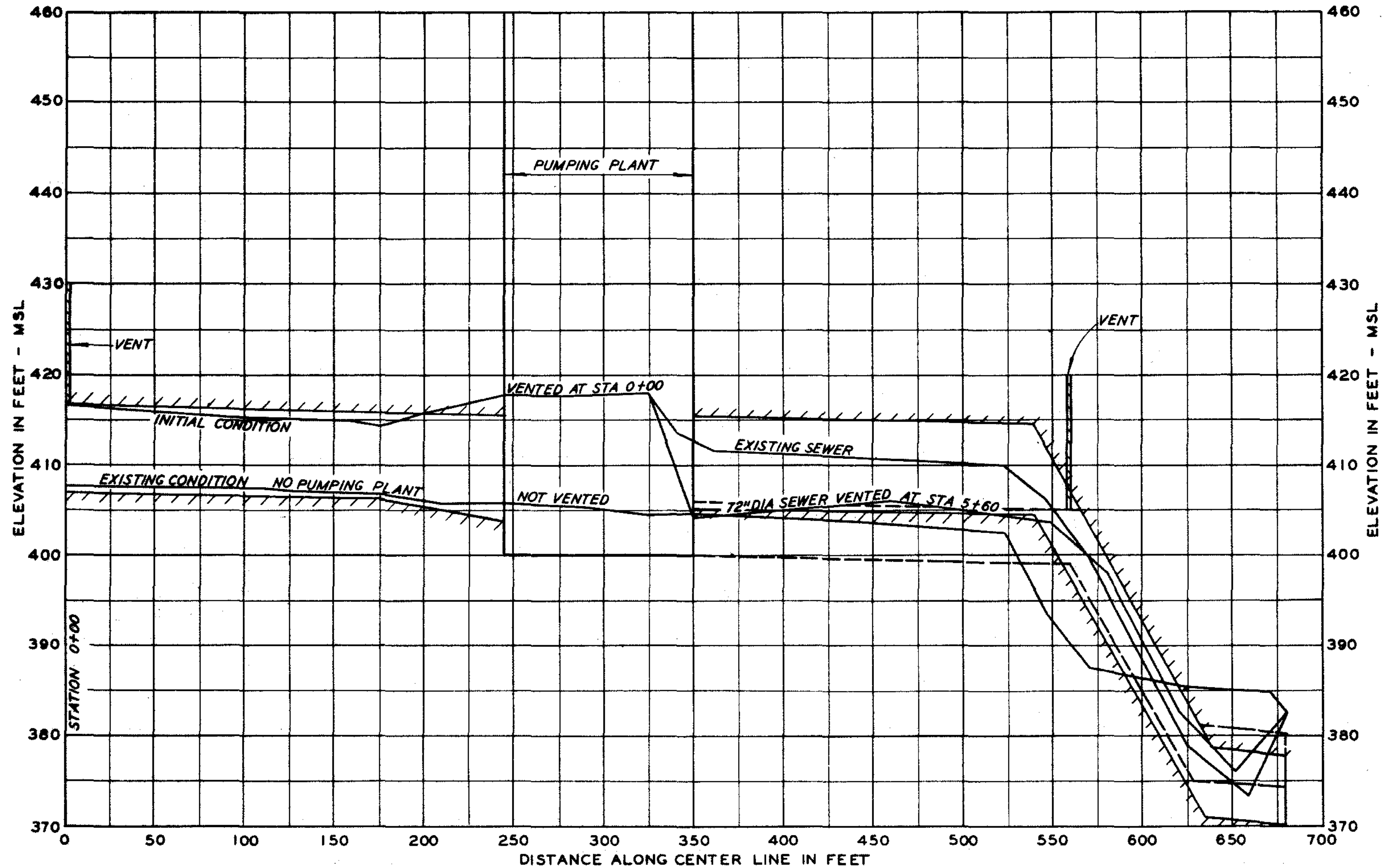
HYDRAULIC GRADE LINES
INITIAL CONDITIONS
GRAVITY FLOW

PLATE 26



NOTE: STATION 0+00 LOCATED 247.34 FT UPSTREAM FROM PROPOSED PUMPING PLANT.

HYDRAULIC GRADE LINES
INITIAL CONDITIONS
PUMP FLOW

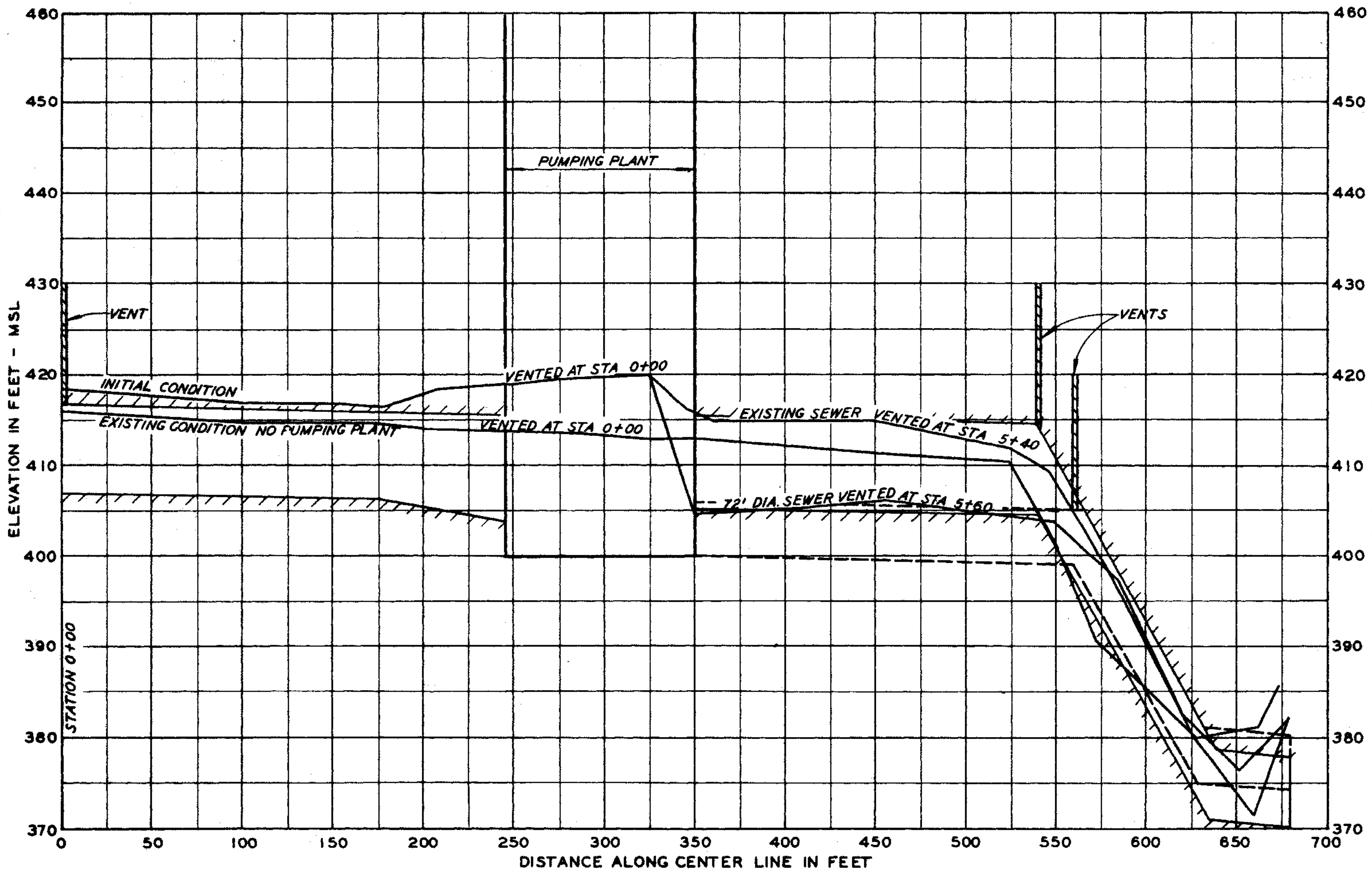


TEST CONDITIONS

DISCHARGE 1800 CFS
RIVER STAGE 382.5

NOTE: STATION 0+00 LOCATED 247.34 FT UPSTREAM FROM PROPOSED PUMPING PLANT.

HYDRAULIC GRADE LINES
INITIAL CONDITION VS NOT VENTED EXISTING CONDITION
GRAVITY FLOW



TEST CONDITIONS

DISCHARGE	1800 CFS
RIVER STAGE	382.5

NOTE: STATION 0+00 LOCATED 247.34 FT UPSTREAM FROM PROPOSED PUMPING PLANT.

HYDRAULIC GRADE LINES
INITIAL CONDITION VS VENTED EXISTING CONDITION
GRAVITY FLOW

APPENDIX A

DEVELOPMENT OF ANALYTICAL METHOD FOR DETERMINATION OF SURGES IN SEWERS

by R. L. Irwin*

1. A search of the available files pertaining to engineering literature, published both in the United States and in foreign countries, indicated that a satisfactory method for the solution of the surge problem in the Southern Outfall Sewer was not to be found. Numerous articles and papers pertaining to surges in penstocks in power systems were available; however, for the condition of multiple risers and laterals, apparently the only solution depended upon the use of model studies. Accordingly, the simplified undistorted 1:32-scale model, described in the basic report, was authorized. To reduce costs to a minimum only about 10,000 ft of the prototype sewer was reproduced. Risers were added at intervals in the main sewer line to simulate the effect of manholes.

2. Simultaneously, a study was made of the possibility of developing a procedure for computing the height of surge rise to be expected at various locations. One of these computations related to the features in the model, and the results compared very favorably with results of the model tests conducted at the Waterways Experiment Station. The method later was described to personnel of the Louisville District Office, who continued the computations and took into consideration the characteristics of flow found in the prototype sewer rather than those used in the model. An explanation of the computations made in the Louisville District is found in appendix B of this report.

3. The computational procedure consists in the derivation and solution of a set of simultaneous equations. In deriving a set of equations for solution, the number of unknown quantities are dependent upon the number of sections of pipe between the pools and surge tanks. It is

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* Formerly of the Ohio River Division and now with the Washington District.

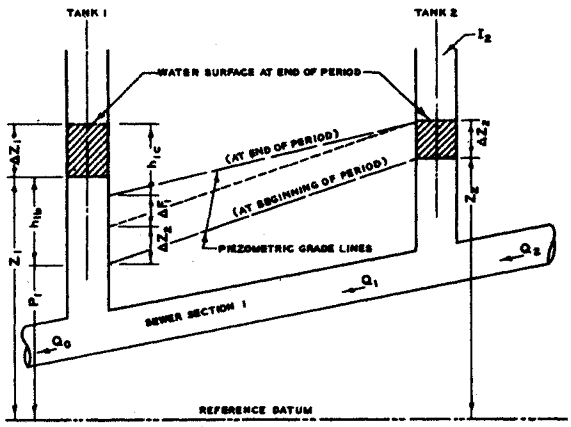
necessary to obtain the results for small successive finite increments of time, assuming the discharge, friction head, velocity head, elevation, and retarding head to vary as a straight line during the interval of time involved. The variable factors are adjusted after each step before proceeding with the computation for determining the next series of incremental changes. On plate A1 is shown the derivation of the basic equations used for determining the set of simultaneous equations applying to a continuous sewer with numerous risers or surge tanks. It is necessary that either the elevation of the uppermost pool remain constant, or that flow into the pool and the dimension of the pool be given or assumed. The derivation may be expanded to include as many laterals as are desirable, provided that each junction of the lateral with the main sewer or another lateral is made at a riser or surge tank. The uppermost point in each lateral system also must meet the conditions stated for the uppermost point on the main sewer. The finite increment of time should not be taken too large or the computation must be carried to too many places to obtain a check. The results obtained through use of time increments that are quite large often lead to serious errors because of the approximations used in deriving the basic equation.

4. If it should be desirable that the relationship for friction head and velocity head follow approximately a curve line, it then becomes necessary to solve a new set of simultaneous equations for each step. A change in the area of the riser for change in elevation also would require a separate solution of equations for each step. In many instances the same solution of equations can be used for several steps, as demonstrated by the computations on plate A2.

5. The computations shown on plate A2 were made for conditions similar to those used in the model tests, and hydrographs which are plotted from the results of the computation are shown on plate A3. The results of the model tests plotted to the same scale as used to plot the computed results are shown on plate A4.

6. From a comparison of the model and computational hydrographs the proposed computational procedure appears to be quite satisfactory. It, therefore, was considered expedient and economical to terminate the

model studies and complete the study through the computational procedure outlined above.



SCHEMATIC DIAGRAM OF SURGE ACTION

(Diagram represents surges during a short time interval in two downstream tanks of a multiple tank system with constant outflow (Q_o) from system and no direct inflow into downstream tank. Time interval represented is a short time after beginning of surge action as Z_1 would be equal to P_1 at instant surge action begins.)

NOMENCLATURE

- Q - Discharge in section of sewer upstream from tank at beginning of time period considered (cfs).
- I - Uniform tributary inflow considered to enter surge tank directly (cfs).
- q - Retardation (or acceleration) in discharge in section of sewer during time period considered (cfs).
- A - Cross-sectional area of surge tank (sq ft).
- a - Cross-sectional area of sewer (sq ft).
- L - Length of sewer section (sq ft).
- Z - Elevation of water surface in surge tank at beginning of time period considered (ft).
- ΔZ - Rise (or fall) in water surface in tank during time period considered (ft).
- P - Piezometric level of tank at beginning of time period considered (ft). (Defined as the elevation of the hydraulic grade line at upstream tank minus hydraulic losses in reach under steady flow conditions.)
- F - Hydraulic losses in section of sewer for initial conditions (ft).
- ΔF - Change in hydraulic losses in reach during time period considered (ft).
- C - Hydraulic loss factor such that $Cq = \Delta F$. (To simplify computations it has been assumed that hydraulic losses are linear functions of sewer discharge.)
- h - Retarding (or accelerating) head acting to cause retardation (or acceleration) in discharge (ft).
- t - Time period (sec).
- g - Acceleration due to gravity (32.2 ft/sec^2).

Subscripts:

- 0, 1, 2, ... n - Tank number or sewer section.
- b - Beginning of time period.
- a - Average during time period.
- e - End of time period.

DERIVATION OF BASIC EQUATIONS

From the laws of motion it can be shown that, if an unbalanced head "h" is applied to a cross-sectional area "a" of a mass of water which has a length, "L," the resulting rate of change of velocity of the water will be $\frac{gh}{L}$ and the rate of change of discharge past a given point will be $\frac{agh}{L}$. Using the adopted nomenclature this can be expressed as:

$$\frac{q_1}{t} = \frac{gh_{1a}\,a_1}{L_1}, \quad \text{(where } \frac{q_1}{t} \text{ is average rate of change of discharge in the first reach of sewer during the time period "t.")} \qquad (1)$$

Or assuming that $h_{1a} = \frac{h_{1b} + h_{1e}}{2}$,

$$\frac{q_1}{t} = \frac{ga_1}{2L_1}(h_{1b} + h_{1e}) \qquad (2)$$

From the above diagram it can be seen that,

$$h_{1b} = Z_1 - P_1, \text{ and} \qquad (3)$$

$$h_{1e} = h_{1b} + \Delta Z_1 - \Delta Z_2 - \Delta F_1 \qquad (4)$$

From consideration of volumes of water entering the two tanks during the period, it can be seen that

$$\Delta Z_1 = \frac{t}{A_1}\left(Q_1 - \frac{q_1}{2} - Q_o\right), \text{ and} \qquad (5)$$

$$\Delta Z_2 = \frac{t}{A_2}\left[\left(Q_2 - \frac{q_2}{2}\right) - \left(Q_1 - I_2 - \frac{q_1}{2}\right)\right] \qquad (6)$$

Substituting from (5) and (6) in (4) gives

$$h_{1e} = h_{1b} + \frac{t}{A_1}\left(Q_1 - Q_o - \frac{q_1}{2}\right) - \frac{t}{A_2}\left[\left(Q_2 - \frac{q_2}{2}\right) - \left(Q_1 - I_2 - \frac{q_1}{2}\right)\right] - \Delta F_1 \qquad (7)$$

Substituting (7) in (2) with ΔF_1 replaced by C_1q_1 and rearranging terms gives

$$\frac{t}{A_1}\left(Q_1 - Q_o - \frac{q_1}{2}\right) + 2h_{1b} - C_1q_1 + \left[\left(Q_1 - I_2 - \frac{q_1}{2}\right) - \left(Q_2 - \frac{q_2}{2}\right)\right]\frac{t}{A_2} = \frac{2q_1L_1}{tga_1} \qquad (8)$$

If surge tank No. 3 is assumed in the system, a similar derivation gives

$$\left[\left(Q_2 - \frac{q_2}{2}\right) - \left(Q_1 - I_2 - \frac{q_1}{2}\right)\right]\frac{t}{A_2} + 2h_{2b} - C_2q_2 + \left[\left(Q_2 - I_3 - \frac{q_2}{2}\right) - \left(Q_3 - \frac{q_3}{2}\right)\right]\frac{t}{A_3} = \frac{2q_2L_2}{tga_2} \qquad (9)$$

It has been assumed that the upstream surge tank in the system (the n^{th} tank) is connected by sewer to a constant-level headwater reservoir. By analysis similar to that above the following equation for this reach of sewer is derived:

$$\left[\left(Q_n - \frac{q_n}{2}\right) - \left(Q_{(n-1)} - I_n - \frac{q_{(n-1)}}{2}\right)\right]\frac{t}{A_n} + 2h_{nb} - C_nq_n = \frac{2q_nL_n}{tga_n} \qquad (10)$$

It will be noted that (8), (9), and (10) are simultaneous equations expressing $q_1, q_2, \ldots\ldots q_n$ (the retardations in discharge in the respective lengths of sewer during a time period) in terms of the characteristics of the sewer system and the heads, discharges, etc., for the system at the beginning of the time period. Since a similar equation can be derived for each reach of sewer between surge tanks, a direct solution for the respective retardations in discharge during each successive time period can be obtained.

DERIVATION OF BASIC EQUATIONS

PLATE A2

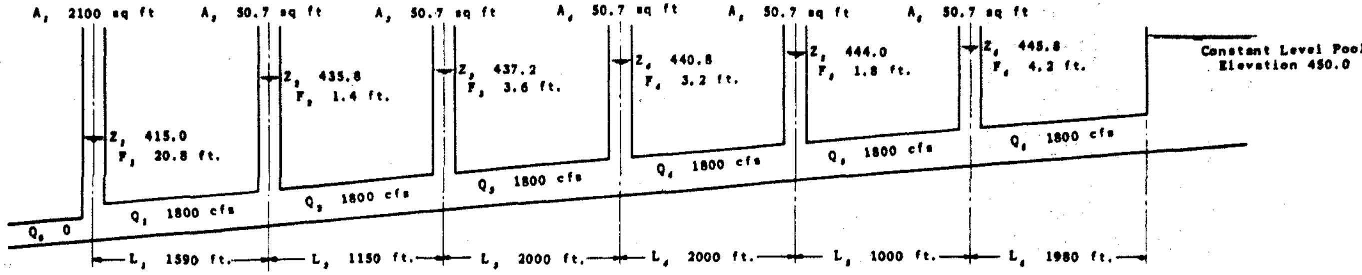
LOUISVILLE, KY. FLOOD PROTECTION PROJECT

SURGE ANALYSIS

COMPUTATION OF SURGES FOR SEWER MODEL
CONSTRUCTED AND TESTED BY U.S.W.E.S.

MODEL OF SOUTHERN OUTFALL SEWER

INITIAL DISCHARGE OF 1800 CFS
WITH INSTANT SHUTDOWN



Basic Equations

(1) $\frac{t}{A_1}(Q_1 - Q_0 - \frac{q_1}{2}) + 2h_{1b} - C_1q_1 + \left[(Q_1 - \frac{q_1}{2}) - (Q_2 - \frac{q_2}{2})\right]\frac{t}{A_2} = \frac{2q_1L_1}{tga_1}$

(2) $\left[(Q_2 - \frac{q_2}{2}) - (Q_1 - \frac{q_1}{2})\right]\frac{t}{A_2} + 2h_{2b} - C_2q_2 + \left[(Q_2 - \frac{q_2}{2}) - (Q_3 - \frac{q_3}{2})\right]\frac{t}{A_3} = \frac{2q_2L_2}{tga_2}$

(3) $\left[(Q_3 - \frac{q_3}{2}) - (Q_2 - \frac{q_2}{2})\right]\frac{t}{A_3} + 2h_{3b} - C_3q_3 + \left[(Q_3 - \frac{q_3}{2}) - (Q_4 - \frac{q_4}{2})\right]\frac{t}{A_4} = \frac{2q_3L_3}{tga_3}$

(4) $\left[(Q_4 - \frac{q_4}{2}) - (Q_3 - \frac{q_3}{2})\right]\frac{t}{A_4} + 2h_{4b} - C_4q_4 + \left[(Q_4 - \frac{q_4}{2}) - (Q_5 - \frac{q_5}{2})\right]\frac{t}{A_5} = \frac{2q_4L_4}{tga_4}$

(5) $\left[(Q_5 - \frac{q_5}{2}) - (Q_4 - \frac{q_4}{2})\right]\frac{t}{A_5} + 2h_{5b} - C_5q_5 + \left[(Q_5 - \frac{q_5}{2}) - (Q_6 - \frac{q_6}{2})\right]\frac{t}{A_6} = \frac{2q_5L_5}{tga_5}$

(6) $\left[(Q_6 - \frac{q_6}{2}) - (Q_5 - \frac{q_5}{2})\right]\frac{t}{A_6} + 2h_{6b} - C_6q_6 = \frac{2q_6L_6}{tga_6}$

Substituting Constants

(1) $+q_1(\frac{20}{4200} + \frac{10}{50.7} + \frac{1}{18.2} + \frac{20.8}{1200}) - q_2\frac{10}{50.7} = Q_1(\frac{20}{2100} + \frac{20}{50.7}) - Q_2\frac{20}{50.7} + 2h_{1b} = T$

(2) $-q_1(\frac{10}{50.7}) + q_2(\frac{20}{50.7} + \frac{1}{56.5} + \frac{1.4}{1200}) - q_3\frac{10}{50.7} = \frac{20}{50.7}(2Q_2 - Q_1 - Q_3) + 2h_{2b} = U$

(3) $-q_2(\frac{10}{50.7}) + q_3(\frac{20}{50.7} + \frac{1}{32.5} + \frac{3.6}{1200}) - q_4\frac{10}{50.7} = \frac{20}{50.7}(2Q_3 - Q_2 - Q_4) + 2h_{3b} = V$

(4) $-q_3(\frac{10}{50.7}) + q_4(\frac{20}{50.7} + \frac{1}{32.5} + \frac{3.2}{1200}) - q_5\frac{10}{50.7} = \frac{20}{50.7}(2Q_4 - Q_3 - Q_5) + 2h_{4b} = W$

(5) $-q_4(\frac{10}{50.7}) + q_5(\frac{20}{50.7} + \frac{1}{65.0} + \frac{1.8}{1200}) - q_6\frac{10}{50.7} = \frac{20}{50.7}(2Q_5 - Q_4 - Q_6) + 2h_{5b} = X$

(6) $-q_5(\frac{10}{50.7}) + q_6(\frac{10}{50.7} + \frac{1}{32.8} + \frac{4.2}{1200}) = \frac{20}{50.7}(Q_6 - Q_5) + 2h_{6b} = Y$

Discharge in C.F.S.

	Time in Seconds	Q_1	Q_2	Q_3	Q_4	Q_5	Q_6
with head losses	0	1800.0000	1800.0000	1800.0000	1800.0000	1800.0000	1800.0000
	20	1671.3672	1708.0380	1735.9129	1752.8154	1761.7193	1767.3461
	40	1457.1888	1489.2326	1525.6467	1557.0291	1577.9418	1592.3715
	60	1241.8593	1243.7307	1240.5154	1239.1219	1240.6706	1242.5272
	80	925.1725	931.8787	935.1374	927.6441	917.2296	908.7364
	100	583.7772	592.5277	606.9404	627.8387	645.1935	657.8787
without head losses	120	277.7281	290.6151	296.9773	298.9728	302.3896	305.6356
	140	-36.7388	-46.9904	-53.0575	-63.1278	-77.6819	-90.1417
	160	-392.8696	-401.4110	-413.3055	-414.4202	-406.3025	-398.1858
	180	-711.4705	-710.7176	-703.3947	-702.6834	-706.1207	-709.3973

Solution of Simultaneous Equations

with head losses assumed to vary uniformly from initial values at + 1800 cfs to 0 at approximately + 600 cfs

$q_1 = 7.5035923\,T + 5.364453\,U + 3.7384198\,V + 2.7524384\,W + 2.2330470\,X + 1.9048149\,Y$

$q_2 = 5.3644530\,T + 7.4597774\,U + 5.1986256\,V + 3.8275248\,W + 3.1052620\,X + 2.6488245\,Y$

$q_3 = 3.7384196\,T + 5.1986248\,U + 7.1560748\,V + 5.2687111\,W + 4.27449320\,X + 3.6461921\,Y$

$q_4 = 2.752438\,T + 3.8275224\,U + 5.2687105\,V + 7.6119487\,W + 6.1755566\,X + 5.2678210\,Y$

$q_5 = 2.2330470\,T + 3.1052607\,U + 4.2744936\,V + 6.17555769\,W + 9.1234952\,X + 7.7824466\,Y$

$q_6 = 1.9048148\,T + 2.6488232\,U + 3.6461923\,V + 5.2678218\,W + 7.7824466\,X + 10.9632842\,Y$

with head losses neglected between approximately + 600 cfs to approximately − 600 cfs

$q_1 = 8.789127\,T + 6.3791819\,U + 4.5416658\,V + 3.4126466\,W + 2.81599806\,X + 2.43899546\,Y$

$q_2 = 6.3791824\,T + 8.3098594\,U + 5.9162138\,V + 4r44549381\,W + 3.6682679\,X + 3.17716438\,Y$

$q_3 = 4.5416658\,T + 5.91621336\,U + 7.8216459\,V + 5.8772519\,W + 4.84970519\,X + 4.2004322\,Y$

$q_4 = 3.4126463\,T + 4.4454930\,U + 5.8772512\,V + 8.22585774\,W + 6.78769358\,X + 5.87896492\,Y$

$q_5 = 2.8159979\,T + 3.6682674\,U + 4.8497049\,V + 6.7876940\,W + 9.78455882\,X + 8.47461322\,Y$

$q_6 = 2.43899540\,T + 3.17716406\,U + 4.2004320\,V + 5.8789654\,W + 8.4746135\,X + 11.73127532\,Y$

Water Elevation

Time in Seconds	Sump	2	3	4	5	6
0	415.0	435.8	437.2	440.8	444.0	445.8
20	431.530303	443.032820	442.698008	444.133826	445.756193	446.909822
40	446.428174	456.585995	455.378284	453.657474	451.637179	450.865739
60	459.280771	463.275383	461.926370	459.572444	456.067435	454.078027
80	469.599960	464.967217	461.934930	457.819624	454.318757	452.769033
100	476.785427	468.015875	465.420413	460.463608	455.687652	453.595857
120	480.887829	472.283626	469.518026	464.979150	459.784614	456.738105
140	482.035395	472.803428	469.576231	463.386488	457.587908	454.920788
160	479.989643	469.096723	466.033510	461.180374	456.318402	454.064161
180	474.730886	467.558162	465.131814	461.100808	457.241557	455.018815

Evaluation of Constants

$\frac{tga_1}{L_1} = \frac{20 \times 32.2 \times 90}{1590} = 36.4$

$\frac{tga_2}{L_2} = \frac{20 \times 32.2 \times 202}{1150} = 113$

$\frac{tga_3}{L_3} = \frac{20 \times 32.2 \times 202}{2000} = 65$

$\frac{tga_4}{L_4} = \frac{20 \times 32.2 \times 202}{2000} = 65$

$\frac{tga_5}{L_5} = \frac{20 \times 32.2 \times 202}{1000} = 130$

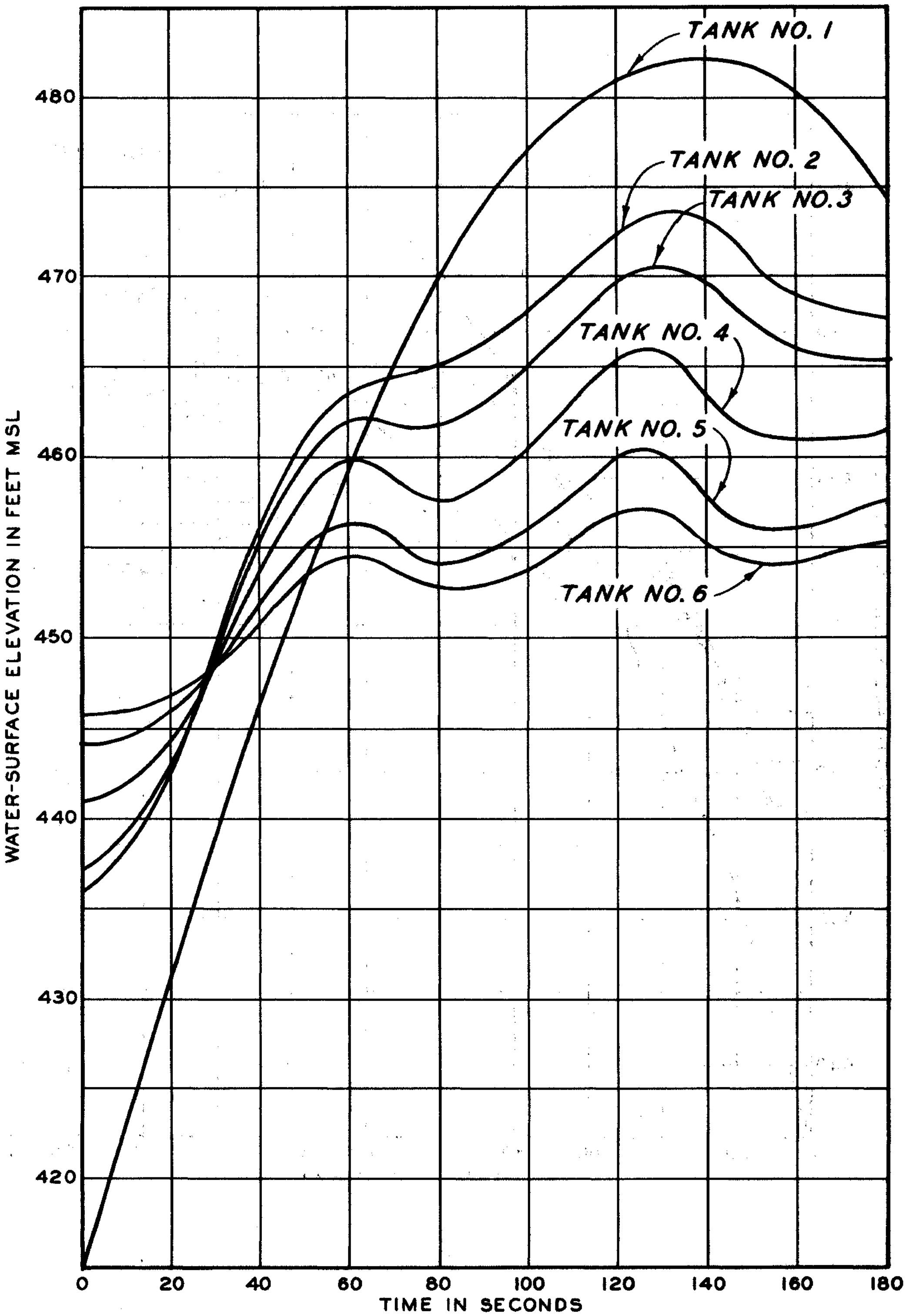
$\frac{tga_6}{L_6} = \frac{20 \times 32.2 \times 202}{1980} = 65.6$

LOUISVILLE FLOOD PROTECTION PROJECT
SOUTHERN OUTFALL SEWER
SURGE ANALYSIS
FOR SHUTDOWN OF FLOW IN MODEL SEWER

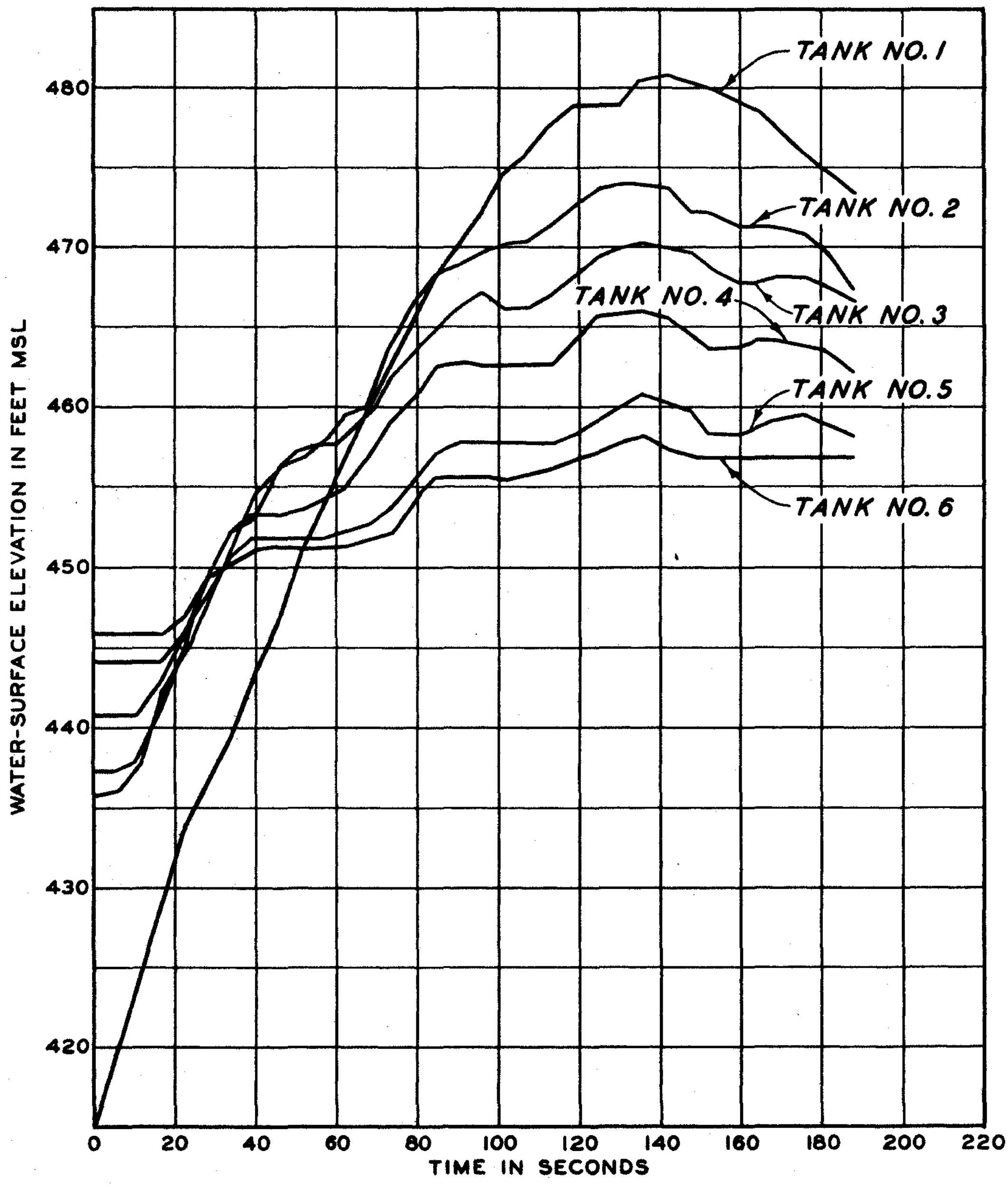
OHIO RIVER DIVISION OVDGW Sheet 1 of 2

Computed by P.F.B. Approved

Checked by R.L.I.



SURGE ANALYSIS
SHUTDOWN OF FLOW IN MODEL SEWER



HYDROGRAPH
FROM MODEL TESTS
SHUTDOWN OF FLOW
IN MODEL SEWER

APPENDIX B

ANALYSIS OF SURGES IN SOUTHERN OUTFALL SEWER (PROTOTYPE)

by F. U. Druml and H. B. Willis*

(November 1950)

General

1. Southern Outfall Sewer drains a large portion of the central urban area to be protected by the Louisville, Kentucky, Flood Protection Project. State Fairgrounds Pumping Plant is planned to pump flood period runoff carried to the line of protection by this sewer when drainage by gravity is blocked by high river stages. This plant will pump combined storm runoff and industrial and sanitary sewage from an area of over seven square miles against static heads ranging from 11 to 67 ft. With the pumping plant operating at or near design capacity, flow conditions in Southern Outfall Sewer will be especially conducive to development of large surges in the pumping plant sump, the main sewer, connecting sewers and manholes, if the pumping plant discharge is suddenly decreased or stopped altogether. Therefore, considerable study has been given to analyses of surges that could develop under various operating criteria in order to evaluate conditions that could develop so as to determine a practicable method for alleviating these surges. This method would then be incorporated into design of the pumping plant and other planned work. The basic report describes surge tests in a greatly simplified model of part of Southern Outfall Sewer in which surges were produced by sudden closure of a valve at the outlet end of the model sewer. Appendix A describes the development of a method for analytically determining surge heights in a simple sewer system, and shows that surges computed by this method for the model sewer compare very favorably with the surges measured in the model tests. The method of computing surges in the prototype sewer is described in the following paragraphs.

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* Louisville District and Office, Chief of Engineers, respectively.

Southern Outfall Sewer

2. The areas served by the major sewer systems in the western portion of Lousiville are shown on plate B1. Southern Outfall Sewer, which is the second largest sewer in the city system, is within this group. It is estimated that a sewered area of about 4,540 acres served by this sewer will contribute storm runoff to the planned State Fairgrounds Pumping Plant within a short time after beginning of rainfall. This area is shown on plate B2. In addition to the 4,540-acre area, an area located near the eastern limits of the sewer system slowly drains into Southern Outfall Sewer through ditches. Because of the very slow runoff from this area, it has been neglected in determining the pumping capacity required at State Fairgrounds Pumping Plant. The drainage area is flat with considerable variation in development. About 75 per cent of the area contains residential developments, 5 per cent commercial developments and 17 per cent industrial developments. The remainder is in parks, cemeteries, and other open spaces. The population of the area served by Southern Outfall Sewer is about 60,000 persons. The area is drained by a well-developed system of sewers, as shown on plate B2. The main sewer, a profile of which is shown on plate B3, is built of reinforced concrete and consists of horseshoe and semielliptical sections of varying sizes. The largest of these is a 15-ft-6-in. x 15-ft-2-in. horseshoe section, which is 3,247 ft long and flows into a sewer of smaller section approximately 1500 ft landside of the line of protection. This next section of sewer is considerably smaller but is on a steeper grade and has a brick paved invert. In addition to runoff from the drainage area shown on plate B1, Southern Outfall Sewer carries sanitary or dry weather flows from the areas to the north that are drained by Western Outfall and Northwestern Trunk Sewers. The dry weather flows of these two sewers reach Southern Outfall Sewer through Western Interceptor at a point approximately 1500 ft landward from the line of protection. At various points near the perimeter of Southern Outfall Sewer drainage area there are sewers that interconnect this area with other sewer systems. However, the probable amounts of flow through these interconnections during

periods of intense rainfall are so small compared to the flow that will reach State Fairgrounds Pumping Plant that these interconnections have been neglected in determining station capacity.

Planned Pumping Capacity

3. Since construction of Southern Outfall Sewer in 1910, the area served has been extended by construction of tributary sewers and has undergone intensive development. As a result, the sewer is now overloaded and, even with low river stages, intense storms frequently produce damaging hydraulic gradients and flooding along several reaches of sewer. Flood-period design rainfalls ordinarily used for determining pumping requirements for interior drainage would cause considerable damage in the area drained by Southern Outfall Sewer because of the inadequate sewer capacity now provided. For this reason the planned pumping capacity at minimum design head for the State Fairgrounds Pumping Plant is based, not upon design rainfall criteria as used at other plants, but on the maximum flow that the existing sewer can deliver to the pumping plant without producing excessive damage within the protected area. In order to fully utilize the capacity of the existing sewer and to provide as high a degree of protection as possible with the conditions that exist, it is planned that during pumping periods the hydraulic gradient at the pumping plant intake will be held at or below the crown of the existing sewer. Computations based upon a value of Kutter's "n" of 0.013 for the main sewer indicate that the maximum discharge that the sewer can deliver to the pumping plant under these conditions without producing considerable damage in the protected area is about 1700 cfs. However, in order to allow an additional margin of safety, it is planned to provide 1800-cfs pumping capacity at minimum design head at State Fairgrounds Pumping Plant. As mentioned above, the computations of discharge capacity of Southern Outfall Sewer were based upon a value of Kutter's "n" of 0.013, which is commonly used for design of concrete sewers. However, measurements of discharge in the 15-ft-6-in. by 15-ft-2-in. concrete horseshoe section of this sewer indicate that a value of "n" of 0.016 would be more

applicable to this section. Rainfall criteria normally used to determine pumping capacities at maximum design heads would not result in runoff rates that exceed the capacity of Southern Outfall Sewer. The required capacity of the State Fairgrounds Pumping Plant at maximum design head based on these criteria is 910 cfs. The computed hydraulic gradient in the main sewer for 1800-cfs inflow to the pumping plant is shown on plate B3. This profile is based upon uniform contribution of runoff from the tributary sewered area. It will be noted that the computed hydraulic gradient on plate B3 shows that approximately 33,000 ft of the main sewer will be surcharged when the inflow to the pumping plant is 1800 cfs. Under these conditions the velocities in the surcharged sections of sewer would be as high as 20 ft per sec (in 10-ft-7-in. by 10-ft-1-1/2-in. sewer near pumping plant). The kinetic energy of the large mass of water moving at high velocities through surcharged sewers introduces the possibility of large surges developing upon pump shutdown.

Generalized Model of Prototype Sewer

4. The analytic method of determining surge heights described in appendix A assumes that the sewer is divided into reaches with the sewer barrel in each reach having a uniform cross-sectional area, and that the rate of flow at any instant is constant throughout each individual reach. The method also assumes a surge tank located between adjacent reaches of sewer with the cross-sectional area of each tank representing the combined horizontal areas of any manholes in the reach or partially filled tributary sewers connecting with the reach that would afford surge relief. By this method, determination of the surges during each short time interval requires the solution of a group of simultaneous algebraic equations, one equation for each reach of sewer in the system. From examination of the plan and profile of Southern Outfall Sewer shown on plates B2 and B3, it is obvious that it would not be practicable to analyze surge action in this sewer by the above-described method if each inflow point, change of sewer section, or opening in the sewer section, or opening in the sewer barrel is considered separately. For this reason

analysis has been based upon a hypothetical sewer system, differing considerably from the prototype but having essentially the same characteristics insofar as the effect upon surge action is concerned. In developing the hypothetical system, several lengths of sewer having slightly different cross-sectional areas and rates of discharge have been considered as one reach having a length equivalent to the corresponding reach in the prototype, but with cross-sectional area and initial rate of discharge roughly equivalent to the weighted averages of the areas and initial discharges of the respective lengths of the reach of prototype sewer. To facilitate computations, the hypothetical system was made as simple as possible without departing too much from those characteristics of the prototype sewer that influence surge heights. For example, since surge is a function of the kinetic energy of moving water in the sewer system, it is apparent that in computing surge heights at points near the river, the kinetic energy of water in small laterals and near the upper end of the main sewer may be neglected if this energy is a very small part of the total energy in the system. With 1800-cfs inflow to the pumping plant, the kinetic energy of the water in all the surcharged laterals would be but a small part of the total kinetic energy of the water in the main sewer. Therefore, surge computations were not extended to the laterals, but the laterals were considered to discharge directly into the surge tanks of the hypothetical sewer system. It was also found that the kinetic energy of water contained in the surcharged main sewer upstream from Third and Avery Streets (point 6 on profile on plate B3) was less than one per cent of the total energy in the system with a pumping plant inflow of 1800 cfs. Thus, surge computations have not been extended upstream from this point, but it has been assumed that a constant level headwater reservoir is located at point 6 on the main sewer. The hypothetical sewer system adopted for the computation of surge heights includes five surge tanks and a constant level reservoir as shown on plate B5. Initial water-surface elevations and discharges required for the computation of surge heights resulting from complete shutdown of the pumping plant from 1800 cfs are also shown.

Method of Analysis

5. The method of determining surge heights provides for the progressive calculation of surges in all risers in the system during successive time intervals. Derivation of basic equations for this method of analysis from the general laws of motion is shown on plate A1 in appendix A. These basic equations express changes in flow in each reach of sewer for a short interval of time in terms of elements of the sewer system, physical constants, length of time interval, and, also, the discharges, water-surface elevations, and hydraulic losses that exist at the beginning of the period. One such equation is derived for each reach of sewer in the system. Thus, for a sewer system with five risers the changes in flow in each reach within a short time interval are expressed by five simultaneous linear equations involving the five unknown changes in discharge. It will be noted that, in order to make the basic equations linear, hydraulic losses between surge tanks are assumed as linear functions of discharge. This, of course, is not true, but such an assumption permits the simplification of the computation procedure and, in the usual case, the errors resulting from this assumption will not greatly affect the computed surge heights. Application of the analytic method to computation of surges that would result from total plant shutdown from 1800 cfs is shown on plate B5. In this and other surge computations it has been assumed that pump discharge would cease instantaneously, although it is known that, owing to inertia of the moving parts of the pumping units and the water within the pump passages, pump discharge would not stop immediately. However, in the model tests of surges described in the basic report, it was found that within certain limits variation in time for complete stoppage of pump discharge made no appreciable differences in the height of surges produced. It will be noted that computations in this example have been carried to several more significant figures than would be justified by the basic data and the nature of the problem. This was done to facilitate arithmetical checks for each interval of time in the computations. In all of the computations of surges in Southern Outfall Sewer the time interval used in the step-by-step computations was 20 sec,

as this interval was found adequate to properly define the surges in this system without requiring an excessive amount of computation. As shown on plate B5, sheet 3, a cross-check on the computations is provided at the end of each successive time interval if the increment of surge in the most downstream tank computed from the basic equations is compared with the increment of surge based on the volume of water entering or leaving the surge tank during the same time interval.

Surge Relief Reservoir

6. Some of the extremely high surges measured in the model tests would not occur in the prototype sewer because the surges measured in the model were generally confined in surge tanks while such surges in the prototype would be higher than the ground surface and would not be confined. However, the model tests and preliminary surge computations indicated that with Southern Outfall Sewer as it now exists there are possibilities of large and possibly destructive surges developing in the pumping plant sump and along the lower reaches of the main sewer during pumping periods. High or moderately high surges could be developed under certain circumstances not only from total shutdown of pumping plant, but also from shutdown of individual pump units. It was found that the pumping plant itself could be protected from damage due to surges in the sump by installation of surge relief valves or flap gates which would open and allow discharge direct to the river during high surges. However, because of the great depth of the planned station, it would be difficult and expensive to install these valves at an elevation low enough to provide surge relief in the protected area. Also, these valves would not be effective in preventing surges at high river stages. In view of these circumstances, it appears that the only method of effectively protecting the pumping plant from damage by surges and at the same time alleviating surges in the main sewer within the protected area consists in providing a surge tank or surge relief reservoir connected to the sewer as close to the pumping plant as possible. This surge relief reservoir would function in the same manner as the simple surge tanks frequently

constructed where long penstocks are used at hydroelectric installations. Fortunately, a site for the development of a surge relief reservoir is available adjacent to Southern Outfall Sewer in the now abandoned channel of Upper Paddy Run within the Kentucky State Fairgrounds. Topography at this site is shown on plate B4. In this low area a reservoir with sufficient storage area to provide effective surge relief can be provided with little excavation by constructing a short levee across the abandoned creek channel. An outlet structure with sufficient capacity to allow high rates of flow from the sewer into the surge relief area with small loss of head will be required. Investigations indicated that, if possible a surge relief reservoir at this site should have about 90,000 sq ft of horizontal area, as a much smaller reservoir would not prevent the development of fairly large surges in the pumping plant sump upon shutdown of individual pump units. It appears practicable to develop a reservoir with about 90,000 sq ft of horizontal area at the State Fairgrounds site and such a reservoir is tentatively planned. Computations of surges for total plant shutdown from 1800 cfs discharge rate, which are shown on plate B5, sheets 1 and 3, are based on a 90,000-sq-ft surge relief reservoir.

Results of Analyses

7. As now planned, State Fairgrounds Pumping Plant will have four large pumping units with 386-cfs capacity each at minimum design head, and two smaller units, each having 128-cfs capacity at minimum design head. Surge analyses have been made for complete plant shutdown when pumping at rated capacity at minimum head (pump plant discharge changing from 1800 cfs to zero), for shutdown of one large pump unit under the same conditions (pump discharge changing from 1800 cfs to 1414 cfs), and for shutdown of one small pump unit (pump discharge changing from 1800 cfs to 1672 cfs). Computed water-surface elevations in risers of the hypothetical system with a 90,000-sq-ft surge relief reservoir in the State Fairgrounds for the first five minutes after complete plant shutdown are shown on plate 5, sheet 4. The table on the following page

shows computed maximum surge heights at the pumping plant sump and at the 90,000-sq-ft surge relief reservoir in the State Fairgrounds for complete plant shutdown from 1800 cfs, for shutdown of one large pump unit when inflow is 1800 cfs, and for shutdown of one small pump unit under the same conditions. It should be noted that the computed surges listed are maxima that would occur in the first four or five minutes after change in pump discharge. It can be seen that, if it is assumed that the inflows to the sewer system remain constant as was done in the surge computations, there would be a gradual filling of the sewer system and connecting low areas because of the storage of inflow. This gradual filling, if continued long enough, could produce higher water-surface elevations than caused by the surges due to kinetic energy of the flowing water. However, records of storm discharge in Southern Outfall Sewer show that peak discharges are of relatively short duration, and, for this reason, flooding from continued sewer discharge would not be as rapid as indicated by the surge computations.

COMPUTED MAXIMUM SURGE HEIGHTS WITH 90,000-SQUARE-FOOT SURGE RELIEF RESERVOIR IN STATE FAIRGROUNDS

(Pumping Plant Inflow = 1800 cfs)

Operating Condition	Maximum Height of Surge in Pumping Plant Sump, ft	Maximum Height of Surge in Surge Relief Reservoir, ft
Complete shutdown of pumping plant	32	4
Shutdown of one large (386 cfs) pumping unit	7	1
Shutdown of one small (128 cfs) pumping unit	2.6	0.3

Possibility of Surge Development

8. Most of the pumping at the State Fairgrounds Pumping Plant will be accomplished under conditions that will not be conducive to surge development upon shutdown of pumping units. The pumping plant

will be operating at high capacity with the attendant surcharge of a considerable part of Southern Outfall Sewer very infrequently. A long reach of surcharged sewer carrying runoff at high velocities is necessary before surges develop in the magnitude of those listed above. It is estimated that a pump plant discharge of 1800 cfs, upon which the computed surges discussed above are based, will occur once in about 25 years on the average. There is little probability of complete shutdown of the pumping plant when pumping at or near maximum capacity. In the design of the plant and its power supply system every practicable safeguard to insure dependable operation will be included. As now planned, power will be supplied to the plant by two three-phase circuits from the Paddy Run Generating Plant of the Louisville Gas and Electric Company, which is located about one mile downriver from the pumping plant site. However, it is possible that inadvertent misoperation at the pumping plant or at the generating plant or a power outage due to transmission line or transformer failure will cause unintentional complete plant shutdown. If such a shutdown should occur as a result of misoperation, it is possible that the pumps could be placed in operation again before serious surge developed. The computations indicate that under certain conditions fairly large surges will develop after the shutdown of one large pumping unit. However, the pumping schedule is to be arranged so that, as the pumps are progressively shut down after a period of high inflow, the maximum sudden decrease of pump discharge will be equivalent to the capacity of one of the smaller pumps. In other words, when it becomes necessary to stop a large pump, the two 128-cfs pumps will be started at the same time.

Conclusions

9. It is estimated that the planned pumping plant could be protected from possible damage due to surge action following complete plant shutdown by installation of high-level surge relief valves in the sump walls at lower cost than by construction of a surge relief reservoir in the State Fairgrounds. However, the surge relief valves would afford

little protection against damages from such surges in the area within the protection works and would not alleviate smaller surges in the pumping plant sump and in Southern Outfall Sewer that could develop during ordinary pumping operations. Since there is little probability of complete plant shutdown when the plant is operating at or near maximum capacity, construction of a surge relief reservoir solely to protect the interior area against damages from such an occurrence may not be justified. However, the computed surges for shutdown of one pumping unit without a surge relief reservoir indicate that rapid changes in sump level would probably make plant operation very difficult and that surges in the main sewer and its laterals might cause some damage in the protected area. For these reasons it is considered that the surge relief reservoir should be provided to insure reasonably satisfactory operating conditions at the pumping plant under all conditions. As indicated by the computed maximum surge heights listed in paragraph 7, even with the planned surge relief reservoir the surges in the pumping plant sump for shutdown of a single pumping unit may cause difficulty in programming pump operation. For this reason consideration is being given to basing the pump operation schedule upon stages at the surge relief reservoir rather than on stages in the pumping plant sump because of the smaller amplitude of the surges in the reservoir.

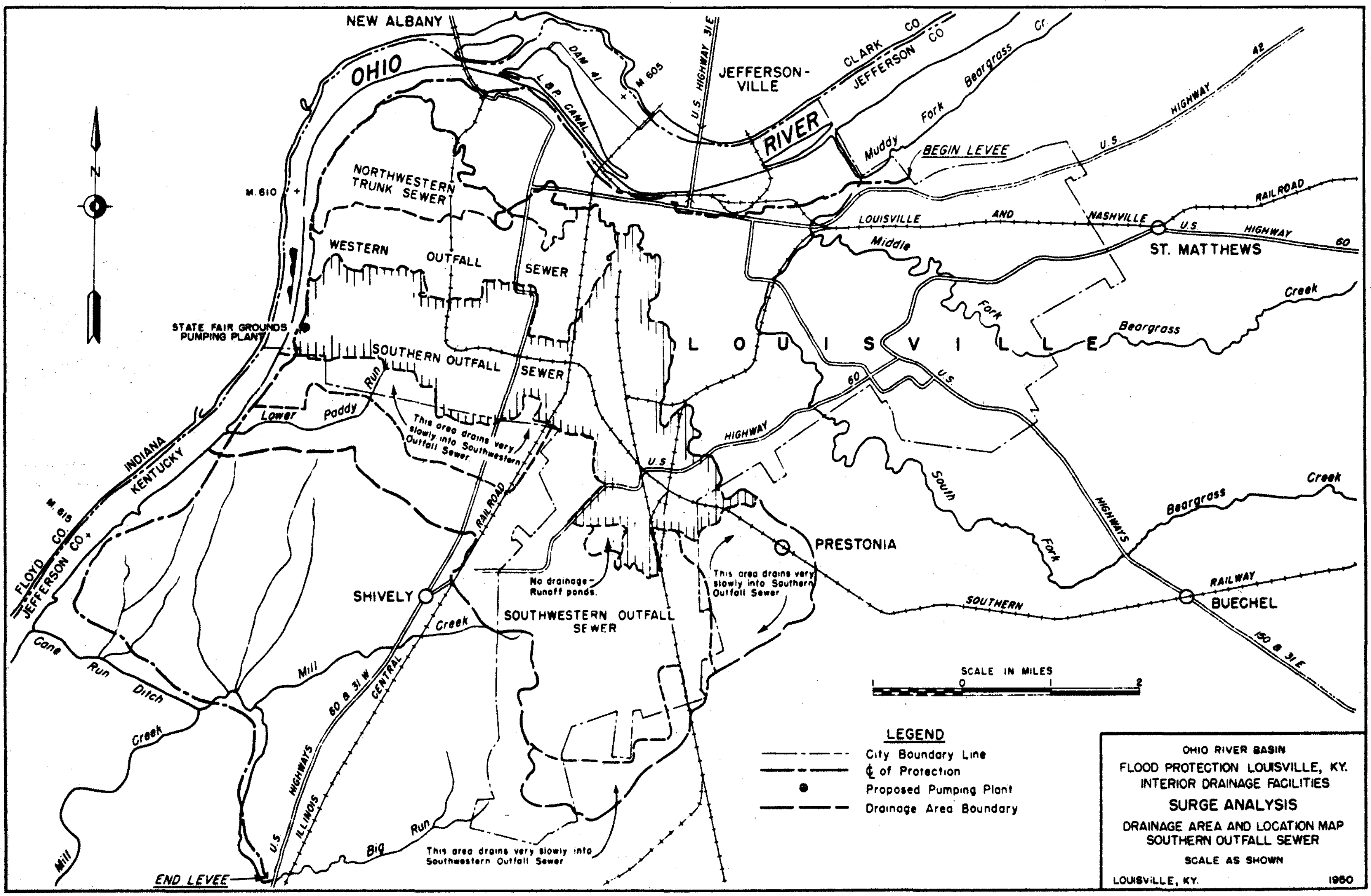
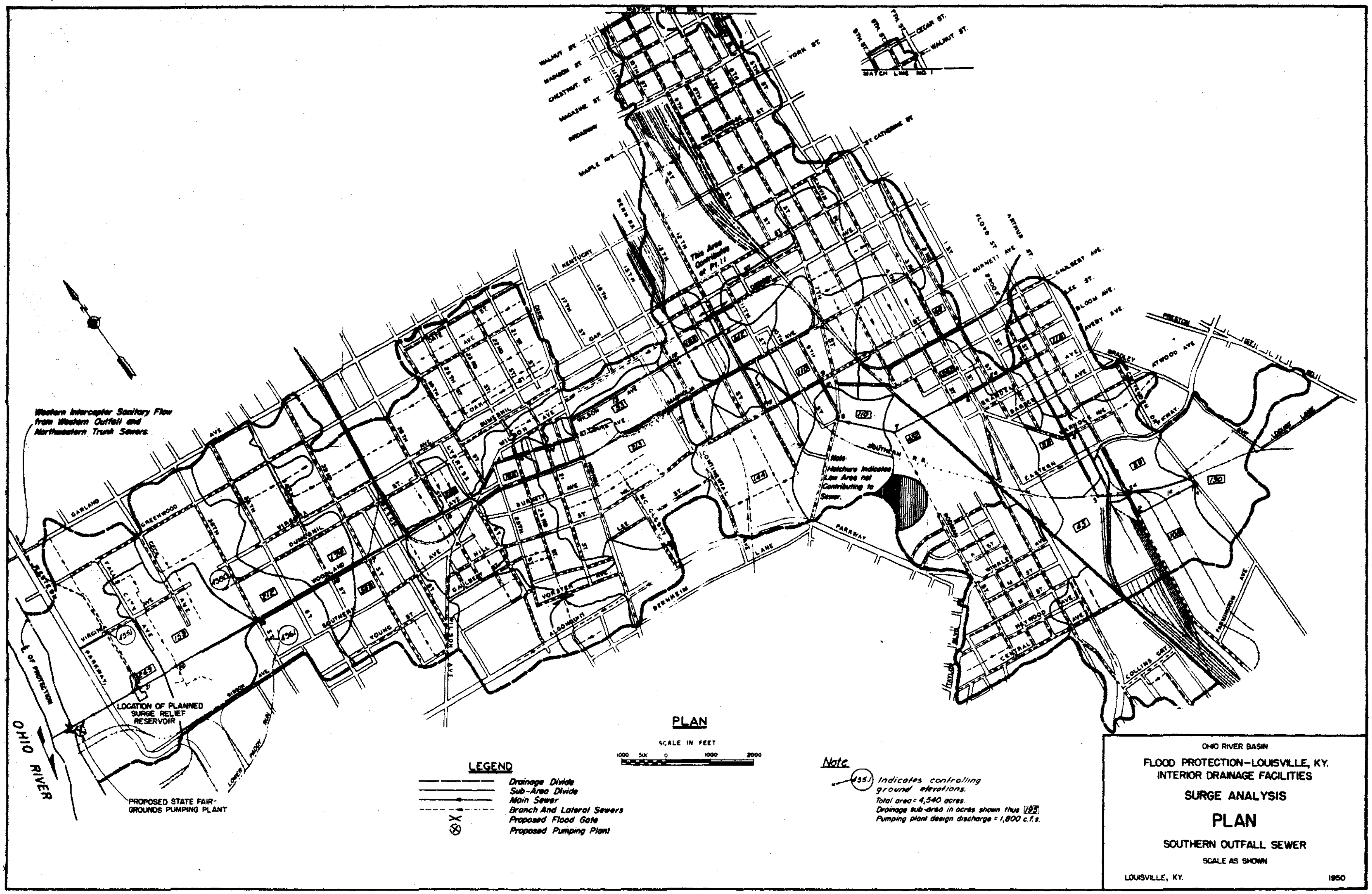
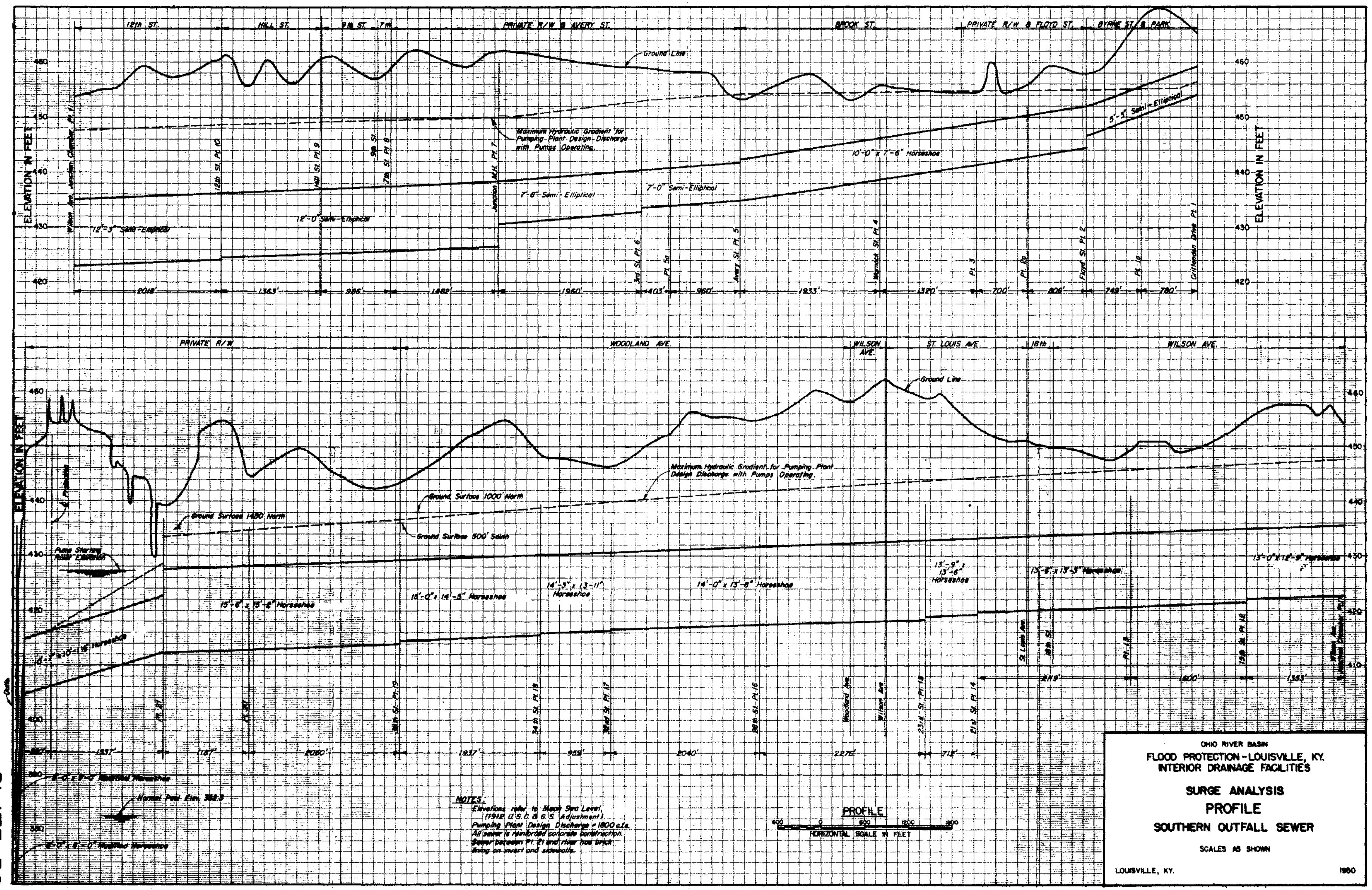

NEW ALBANY
OHIO
RIVER
JEFFERSON-VILLE
CLARK CO.
JEFFERSON CO.
DAM 41
M. 605
M. 610
M. 615
L. & P. CANAL
U.S. HIGHWAY 31E
Muddy Fork Beargrass Cr.
BEGIN LEVEE
U.S. HIGHWAY 42
RAILROAD
LOUISVILLE AND NASHVILLE
U.S. HIGHWAY 60
ST. MATTHEWS
Middle Fork Beargrass Creek
NORTHWESTERN TRUNK SEWER
WESTERN OUTFALL SEWER
STATE FAIR GROUNDS PUMPING PLANT
SOUTHERN OUTFALL SEWER
L O U I S V I L L E
Paddy Run
Lower Paddy
This area drains very slowly into Southwestern Outfall Sewer.
INDIANA
KENTUCKY
FLOYD CO.
JEFFERSON CO.
RAILROAD
HIGHWAY 60
U.S.
South Fork
Beargrass Creek
HIGHWAYS
PRESTONIA
No drainage-Runoff ponds.
This area drains very slowly into Southern Outfall Sewer
SOUTHERN RAILWAY
BUECHEL
150 & 31 E
SHIVELY
SOUTHWESTERN OUTFALL SEWER
Mill Creek
Cane Run Ditch
60 & 31 W
CENTRAL
U.S. HIGHWAYS
ILLINOIS
Creek
Mill
Big Run
This area drains very slowly into Southwestern Outfall Sewer
END LEVEE
SCALE IN MILES
1 0 1 2
LEGEND
City Boundary Line
℄ of Protection
Proposed Pumping Plant
Drainage Area Boundary
OHIO RIVER BASIN
FLOOD PROTECTION LOUISVILLE, KY.
INTERIOR DRAINAGE FACILITIES
SURGE ANALYSIS
DRAINAGE AREA AND LOCATION MAP
SOUTHERN OUTFALL SEWER
SCALE AS SHOWN
LOUISVILLE, KY. 1950

PLATE B1

PLATE B2

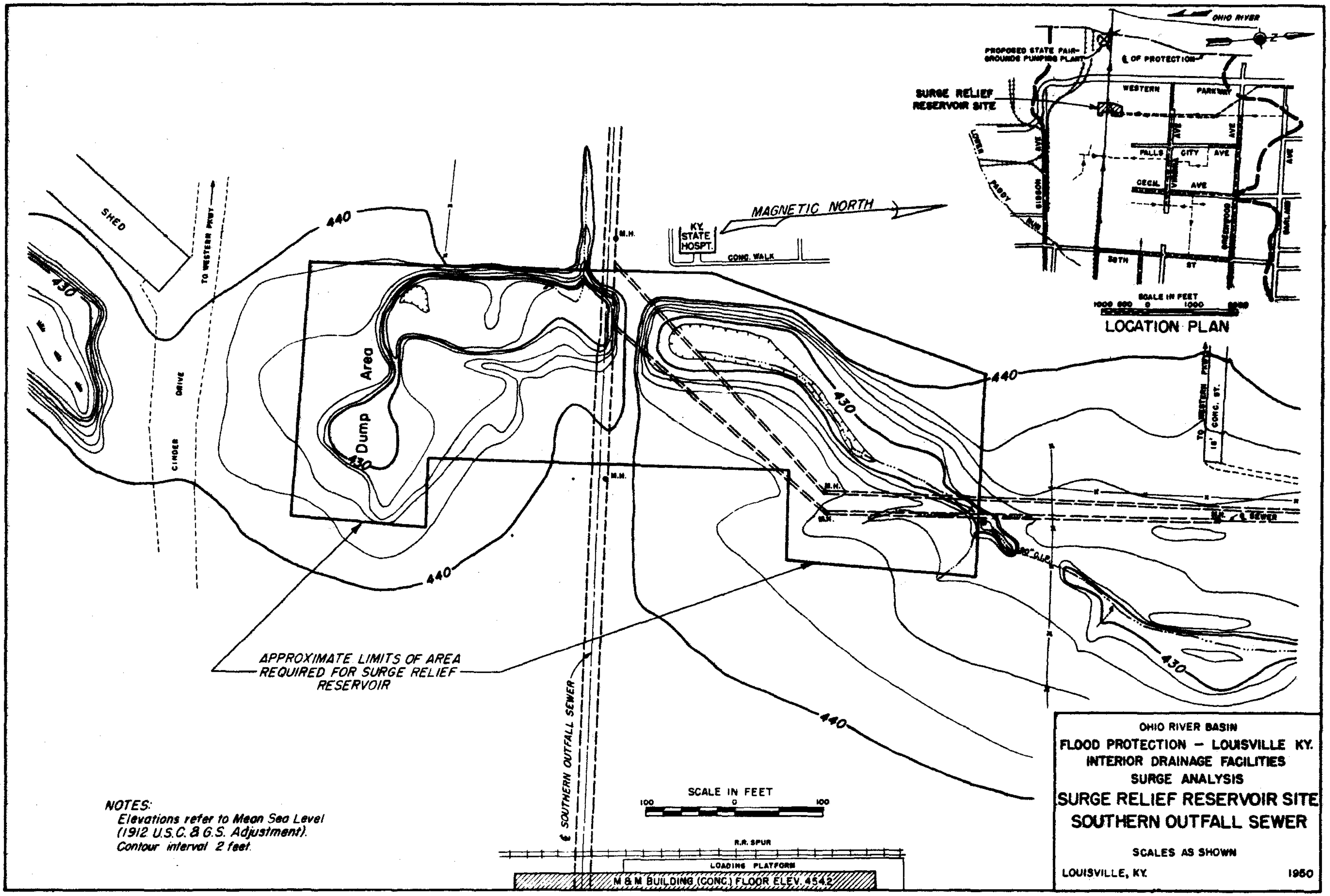

OHIO RIVER BASIN
FLOOD PROTECTION—LOUISVILLE, KY.
INTERIOR DRAINAGE FACILITIES
SURGE ANALYSIS
PLAN
SOUTHERN OUTFALL SEWER
SCALE AS SHOWN
LOUISVILLE, KY.
1950
PLAN
SCALE IN FEET
LEGEND
Drainage Divide
Sub-Area Divide
Main Sewer
Branch And Lateral Sewers
Proposed Flood Gate
Proposed Pumping Plant
Note
Indicates controlling ground elevations.
Total area = 4,540 acres.
Drainage sub-area in acres shown thus 193
Pumping plant design discharge = 1,800 c.f.s.
Western Interceptor Sanitary Flow from Western Outfall and Northwestern Trunk Sewers.
LOCATION OF PLANNED SURGE RELIEF RESERVOIR
PROPOSED STATE FAIR-GROUNDS PUMPING PLANT
OHIO RIVER
This Area Contributes to Pt.11
Note: Hatchure Indicates Low Area not Contributing to Sewer.
MATCH LINE NO. 1



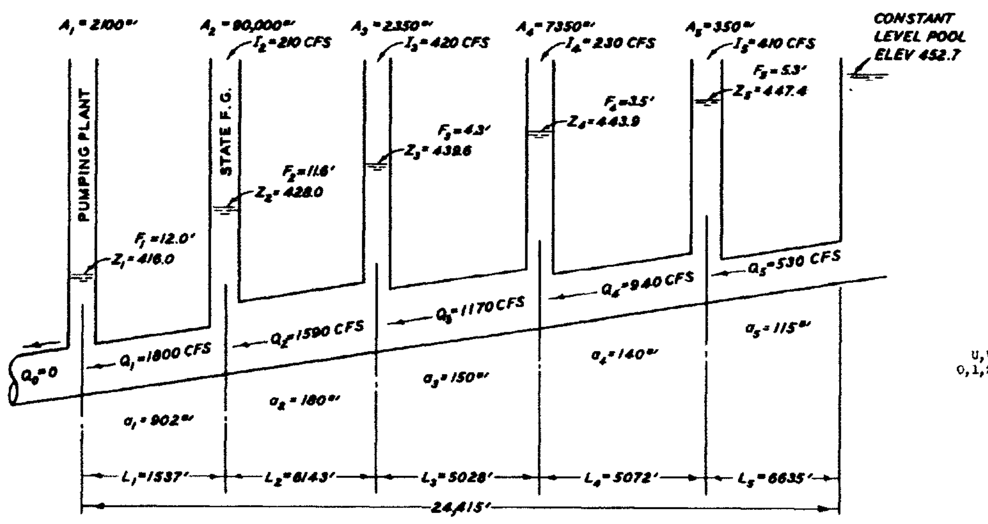
12th ST.
HILL ST.
9th ST.
7th
PRIVATE R/W & AVERY ST.
BROOK ST.
PRIVATE R/W & FLOYD ST.
BYRNE ST. & PARK
Ground Line
Maximum Hydraulic Gradient for Pumping Plant Design-Discharge with Pumps Operating.
10'-0" x 7'-6" Horseshoe
5'-3" Semi-Elliptical
7'-0" Semi-Elliptical
7'-6" Semi-Elliptical
12'-0" Semi-Elliptical
12'-3" Semi-Elliptical
ELEVATION IN FEET
2018'
1363'
985'
1882'
1960'
403'
960'
1933'
1320'
700'
809'
749'
780'
PRIVATE R/W
WOODLAND AVE.
WILSON AVE.
ST. LOUIS AVE.
18th
WILSON AVE.
Ground Line
Maximum Hydraulic Gradient for Pumping Plant Design Discharge with Pumps Operating.
Ground Surface 1000' North
Ground Surface 1450' North
Ground Surface 500' South
Pump Starting River Elevation
15'-6" x 15'-6" Horseshoe
15'-0" x 14'-5" Horseshoe
14'-3" x 13'-11" Horseshoe
14'-0" x 13'-6" Horseshoe
13'-9" x 13'-6" Horseshoe
13'-6" x 13'-3" Horseshoe
13'-0" x 12'-9" Horseshoe
Normal Pool Elev. 382.0
1537'
1187'
2080'
1937'
959'
2040'
2276'
712'
2119'
1600'
1353'
NOTES:
Elevations refer to Mean Sea Level, (1912 U.S.C. & G.S. Adjustment).
Pumping Plant Design Discharge = 1800 c.f.s.
All sewer is reinforced concrete construction.
Sewer between Pt. 21 and river has brick lining on invert and sidewalls.
PROFILE
HORIZONTAL SCALE IN FEET
OHIO RIVER BASIN
FLOOD PROTECTION - LOUISVILLE, KY.
INTERIOR DRAINAGE FACILITIES
SURGE ANALYSIS
PROFILE
SOUTHERN OUTFALL SEWER
SCALES AS SHOWN
LOUISVILLE, KY.
1960

PLATE B3

PLATE B4



SHED
TO WESTERN PKWY
CINDER
DRIVE
Dump
Area
430
440
M.H.
KY. STATE HOSPT.
CONC. WALK
MAGNETIC NORTH
TO WESTERN PKWY
18' CONC. ST.
℄ SEWER
APPROXIMATE LIMITS OF AREA REQUIRED FOR SURGE RELIEF RESERVOIR
℄ SOUTHERN OUTFALL SEWER
SCALE IN FEET
100 0 100
R.R. SPUR
LOADING PLATFORM
M & M BUILDING (CONC.) FLOOR ELEV. 454.2
NOTES:
Elevations refer to Mean Sea Level (1912 U.S.C. & G.S. Adjustment).
Contour interval 2 feet.
OHIO RIVER
PROPOSED STATE FAIR-GROUNDS PUMPING PLANT
℄ OF PROTECTION
SURGE RELIEF RESERVOIR SITE
WESTERN PARKWAY
FALLS CITY AVE
CECIL AVE
38TH ST
SCALE IN FEET
LOCATION PLAN
OHIO RIVER BASIN
FLOOD PROTECTION – LOUISVILLE KY.
INTERIOR DRAINAGE FACILITIES
SURGE ANALYSIS
SURGE RELIEF RESERVOIR SITE
SOUTHERN OUTFALL SEWER
SCALES AS SHOWN
LOUISVILLE, KY. 1950



FIVE TANK SYSTEM ADOPTED TO SIMPLIFY ANALYSIS

NOMENCLATURE

- Q - Discharge in section of sewer upstream from tank at beginning of time period considered (cfs).
- I - Uniform tributary inflow considered to enter tank directly (cfs).
- q - Retardation (or acceleration) in discharge in section of sewer during time period considered (cfs).
- A - Cross-sectional area of tank (sq ft).
- a - Cross-sectional area of sewer (sq ft).
- L - Length of sewer section (ft).
- Z - Elevation of water surface in tank at beginning of time period considered (ft).
- ΔZ - Rise (or fall) in water surface elevation in tank during time period considered (ft).
- F - Hydraulic losses in reach of sewer for initial conditions (ft).
- ΔF - Change in hydraulic losses in reach during time period considered (ft).
- C - Hydraulic loss factor such that $Cq = \Delta F$ (to simplify computations it has been assumed that hydraulic losses are linear functions of sewer discharges).
- h - Retarding (or accelerating) head acting to cause retardation (or acceleration) in discharge (ft).
- t - Time period (sec). (Used as 20 seconds throughout this analysis.)
- g - Acceleration due to gravity (32.2 ft/sec^2).
- U,V,W,X,Y - Functions of $(Q_1 h)$. Used to simplify equations.
- 0,1,2,3,4,5 - Subscripts to denote tank number or reach of sewer.
- b - Subscript to denote beginning of time period.
- a - Subscript to denote average during time period.
- e - Subscript to denote end of time period.

Quantities shown in above sketch may be further defined:

- A_1 represents cross-sectional area of pumping plant sump.
- A_2 represents surge area assumed to be made available at State Fairgrounds.
- A_3, A_4 and A_5 are estimated sewer and manhole areas available on main stem and tributary sewers.
- Z_1, Z_2, Z_3, Z_4 and Z_5 are the hydraulic grade line elevations obtained for flow conditions in the prototype sewer producing 1800 cfs at the pumping plant.
- Q_1, Q_2, Q_3, Q_4 and Q_5 are average initial discharges in the respective reaches.
- L_1, L_2, L_3, L_4 and L_5 are prototype lengths between assumed tank locations.

A constant level pool has been assumed in the upper reaches of the sewer close to the ground surface on the hydraulic grade line where little variation in elevation would occur during the surge period studied.

Basic Equations:

$$(1)\quad \frac{t}{A_1}\left(Q_1 - Q_0 - \frac{q_1}{2}\right) + 2h_{1b} - C_1q_1 + \left[\left(Q_1 - I_2 - \frac{q_1}{2}\right) - \left(Q_2 - \frac{q_2}{2}\right)\right]\frac{t}{A_2} = \frac{2q_1L_1}{tga_1}$$

$$(2)\quad \left[\left(Q_2 - \frac{q_2}{2}\right) - \left(Q_1 - I_2 - \frac{q_1}{2}\right)\right]\frac{t}{A_2} + 2h_{2b} - C_2q_2 + \left[\left(Q_2 - I_3 - \frac{q_2}{2}\right) - \left(Q_3 - \frac{q_3}{2}\right)\right]\frac{t}{A_3} = \frac{2q_2L_2}{tga_2}$$

$$(3)\quad \left[\left(Q_3 - \frac{q_3}{2}\right) - \left(Q_2 - I_3 - \frac{q_2}{2}\right)\right]\frac{t}{A_3} + 2h_{3b} - C_3q_3 + \left[\left(Q_3 - I_4 - \frac{q_3}{2}\right) - \left(Q_4 - \frac{q_4}{2}\right)\right]\frac{t}{A_4} = \frac{2q_3L_3}{tga_3}$$

$$(4)\quad \left[\left(Q_4 - \frac{q_4}{2}\right) - \left(Q_3 - I_4 - \frac{q_3}{2}\right)\right]\frac{t}{A_4} + 2h_{4b} - C_4q_4 + \left[\left(Q_4 - I_5 - \frac{q_4}{2}\right) - \left(Q_5 - \frac{q_5}{2}\right)\right]\frac{t}{A_5} = \frac{2q_4L_4}{tga_4}$$

$$(5)\quad \left[\left(Q_5 - \frac{q_5}{2}\right) - \left(Q_4 - I_5 - \frac{q_4}{2}\right)\right]\frac{t}{A_5} + 2h_{5b} - C_5q_5 = \frac{2q_5L_5}{tga_5}$$

EVALUATION OF CONSTANTS

$$\frac{2L_1}{tga_1} = \frac{2 \times 1537}{20 \times 32.2 \times 90.2} = .052918979 \qquad \frac{10}{2100} = .0047619048$$

$$\frac{2L_2}{tga_2} = \frac{2 \times 6143}{20 \times 32.2 \times 180} = .10598689 \qquad \frac{20}{2100} = .0095238095$$

$$\frac{2L_3}{tga_3} = \frac{2 \times 5028}{20 \times 32.2 \times 150} = .10409938 \qquad \frac{10}{90{,}000} = .00011111111$$

$$\frac{2L_4}{tga_4} = \frac{2 \times 5072}{20 \times 32.2 \times 140} = .11251109 \qquad \frac{20}{90{,}000} = .00022222222$$

$$\frac{2L_5}{tga_5} = \frac{2 \times 6635}{20 \times 32.2 \times 115} = .17917904 \qquad \frac{10}{2350} = .0042553191$$

$$C_1 = \frac{12}{1800} = .0066666667 \qquad \frac{20}{2350} = .0085106383$$

$$C_2 = \frac{11.6}{1590} = .0072955975 \qquad \frac{10}{7350} = .0013605442$$

$$C_3 = \frac{4.3}{1170} = .0036752137 \qquad \frac{20}{7350} = .0027210884$$

$$C_4 = \frac{3.5}{940} = .0037234043 \qquad \frac{10}{350} = .028571429$$

$$C_5 = \frac{5.3}{530} = .010000000 \qquad \frac{20}{350} = .057142857$$

Substituting Constants:

$$(1)\quad \frac{20}{2100}\left(Q_1 - 0 - \frac{q_1}{2}\right) + 2h_{1b} - \frac{12}{1800}q_1 + \left[\left(Q_1 - 210 - \frac{q_1}{2}\right) - \left(Q_2 - \frac{q_2}{2}\right)\right]\frac{20}{90{,}000} = \frac{2q_1 \times 1537}{20 \times 32.2 \times 90.2}$$

$$(2)\quad \left[\left(Q_2 - \frac{q_2}{2}\right) - \left(Q_1 - 210 - \frac{q_1}{2}\right)\right]\frac{20}{90{,}000} + 2h_{2b} - \frac{11.6}{1590}q_2 + \left[\left(Q_2 - 420 - \frac{q_2}{2}\right) - \left(Q_3 - \frac{q_3}{2}\right)\right]\frac{20}{2350} = \frac{2q_2 \times 6143}{20 \times 32.2 \times 180}$$

$$(3)\quad \left[\left(Q_3 - \frac{q_3}{2}\right) - \left(Q_2 - 420 - \frac{q_2}{2}\right)\right]\frac{20}{2350} + 2h_{3b} - \frac{4.3}{1170}q_3 + \left[\left(Q_3 - 230 - \frac{q_3}{2}\right) - \left(Q_4 - \frac{q_4}{2}\right)\right]\frac{20}{7350} = \frac{2q_3 \times 5028}{20 \times 32.2 \times 150}$$

$$(4)\quad \left[\left(Q_4 - \frac{q_4}{2}\right) - \left(Q_3 - 230 - \frac{q_3}{2}\right)\right]\frac{20}{7350} + 2h_{4b} - \frac{3.5}{940}q_4 + \left[\left(Q_4 - 410 - \frac{q_4}{2}\right) - \left(Q_5 - \frac{q_5}{2}\right)\right]\frac{20}{350} = \frac{2q_4 \times 5072}{20 \times 32.2 \times 140}$$

$$(5)\quad \left[\left(Q_5 - \frac{q_5}{2}\right) - \left(Q_4 - 410 - \frac{q_4}{2}\right)\right]\frac{20}{350} + 2h_{5b} - \frac{5.3}{530}q_5 = \frac{2q_5 \times 6635}{20 \times 32.2 \times 115}$$

Collecting terms and setting equations equal to U , V , W , X and Y :

(It is to be noted that numerical values have been computed to eight significant figures in order to obtain seven significant figures accuracy in the results shown at the bottom of the sheet. Seven significant figure accuracy was found necessary to maintain proper evaluation of all factors involved and to obtain arithmetical checks at the end of each time period as shown on sheet (3).)

Or:

$$(1)\quad -q_1(.0047619048 + .0066666667 + .00011111111 + .052918979) + .00011111111q_2 - .00022222222Q_2 - Q_1(.0095238095 + .00022222222) - 2h_1 + .046666666 = U$$

$$(2)\quad .00011111111q_1 - q_2(.00011111111 + .0072955975 + .0042553191 + .10598689) + .0042553191q_3 = .0085106383(Q_3 - Q_2) + .00022222222(Q_1 - Q_2) - 2h_2 + 3.5744681 - .046666666 = V$$

$$(3)\quad .0042553191q_2 - q_3(.0042553191 + .0036752137 + .0013605442 + .10409938) + .0013605442q_4 = .0085106383(Q_2 - Q_3) + .0027210884(Q_4 - Q_3) - 2h_3 - 3.5744681 + .62585033 = W$$

$$(4)\quad .0013605442q_3 - q_4(.0013605442 + .0037234043 + .028571429 + .11251109) + .028571429q_5 = .0027210884(Q_3 - Q_4) + .057142857(Q_5 - Q_4) - 2h_4 - .62585033 + 23.428571 = X$$

$$(5)\quad .028571429q_4 - q_5(.028571429 + .010000000 + .17917904) = .057142857(Q_4 - Q_5) - 2h_5 - 23.428571 = Y$$

And:

$$(1)\quad -.064458662q_1 + .00011111111q_2 = .00022222222Q_2 - .0097460317Q_1 - 2h_1 + .046666666 = U$$

$$(2)\quad .00011111111q_1 - .11764892q_2 + .0042553191q_3 = .0085106383(Q_3 - Q_2) + .00022222222(Q_1 - Q_2) - 2h_2 + 3.5278014 = V$$

$$(3)\quad .0042553191q_2 - .11339046q_3 + .0013605442q_4 = .0085106383(Q_2 - Q_3) + .0027210884(Q_4 - Q_3) - 2h_3 - 2.9486178 = W$$

$$(4)\quad .0013605448q_3 - .14616647q_4 + .028571429q_5 = .0027210884(Q_3 - Q_4) + .057142857(Q_5 - Q_4) - 2h_4 + 22.802721 = X$$

$$(5)\quad .028571429q_4 - .21775047q_5 = .057142857(Q_4 - Q_5) - 2h_5 - 23.428571 = Y$$

By Elimination:

Eq (4) x 7.6212663	(6)	$-1.1139736q_4$	$+ .21775047q_5$	$+ .010369070q_3$		$= 7.6212663X$
Eq (5) + Eq (6)	(7)	$-1.0854022q_4$		$+ .010369070q_3$		$= 7.6212663X + Y$
Eq (3) x 797.77063	(8)	$1.0854022q_4$	$+ 3.3947686q_2$	$- 90.459579q_3$		$= 797.77063W$
Eq (7) + Eq (8)	(9)		$3.3947686q_2$	$- 90.449210q_3$		$= 797.77063W + 7.6212663X + Y$
Eq (2) x 21,255.565	(10)	$2.3617294q_1$	$- 2500.6943q_2$	$+ 90.449210q_3$		$= 21{,}255.565V$
Eq (9) + Eq (10)	(11)	$2.3617294q_1$	$- 2497.2995q_2$			$= 21{,}255.565V + 797.77063W + 7.6212663X + Y$
Eq (1) x 22,475,696	(12)	$-1{,}448{,}753.3q_1$	$+ 2497.2995q_2$			$= 22{,}475{,}696U$
Eq (11) + Eq (12)	(13)	$-1{,}448{,}750.9q_1$				$= 22{,}475{,}696U + 21{,}255.565V + 797.77063W + 7.6212663X + Y$

Eq (13) x .00000069024979	$q_1 = -15.513844U - .014671649V - .00055066101W - .0000052605775X - .00000069024979Y$
Eq (11) x .00040043255	$q_2 = -8.5114201V - .31945333W - .0030518031X - .00040043255Y + .00094571333q_1$
Eq (9) x .011055929	$q_3 = -8.8200954W - .084260179X - .011055929Y + .037532321q_2$
Eq (7) x .92131746	$q_4 = -7.0216057X - .92131746Y + .0095532052q_3$
Eq (5) x 4.5924126	$q_5 = -4.5924126Y + .13121179q_4$

Solving for q in terms of U , V , W , X and Y

	U	V	W	X	Y
$q_1 =$	-15.513844U	-.014671649V	-.00055066101W	-.0000052605775X	-.00000069024979Y
	-.014671649U	-.000013875174V	-.0000005208W	-.000000004975X	-.000000000653Y
		-8.5114201V	-.31945333W	-.0030518031X	-.00040043255Y
$q_2 =$	-.014671649U	-8.511434V	-.31945385W	-.0030518081X	-.00040043320Y
	-.00055066104U	-.31945387V	-.011989844W	-.00011454144X	-.000015029Y
			-8.8200954W	-.084260179X	-.011055929Y
$q_3 =$	-.00055066104U	-.31945387V	-8.8320852W	-.084374720X	-.011070958Y
	-.0000052605779U	-.0030518084V	-.084374722W	-.00080604901X	-.00010576313Y
				-7.0216057X	-.92131746Y
$q_4 =$	-.0000052605779U	-.0030518084V	-.084374722W	-7.0224122X	-.92142322Y
	-.00000069024984U	-.00040043324V	-.011070958W	-.92142327X	-.12090159Y
					-4.5924126Y
$q_5 =$	-.00000069024984U	-.00040043324V	-.011070958W	-.92142327X	-4.7133142Y

It is to be noted that in the equations below the coefficients of U , V , W , X and Y are repeated in a regular pattern, providing an arithmetical check on the computation of these coefficients.

Equations to Seven Significant Figures:

$$q_1 = -15.51384U \quad -.01467165V \quad -.0005506610W \quad -.000005260578X \quad -.0000006902498Y$$

$$q_2 = -.01467165U \quad -8.511434V \quad -.3194538W \quad -.003051808X \quad -.0004004332Y$$

$$q_3 = -.0005506610U \quad -.3194539V \quad -8.832085W \quad -.08437472X \quad -.01107096Y$$

$$q_4 = -.000005260578U \quad -.003051808V \quad -.08437472W \quad -7.022412X \quad -.9214232Y$$

$$q_5 = -.0000006902498U \quad -.0004004332V \quad -.01107096W \quad -.9214232X \quad -4.713314Y$$

COMPLETE SHUTDOWN OF STATE FAIRGROUNDS PUMPING PLANT

(1800 CFS TO 0 CFS)

Time in Seconds	Q_1	Q_2	Q_3	Q_4	Q_5
0	1800.0000	1590.0000	1170.0000	940.0000	530.0000
	265.9516	.2515	.0094	.0001	.0000
20	1534.0484	1589.7485	1169.9906	939.9999	530.0000
	662.4226	1.5665	.0941	.0012	.0002
40	871.6258	1588.1820	1169.8965	939.9987	529.9998
	776.5826	4.7455	.4598	.0077	.0015
60	95.0432	1583.4365	1169.4367	939.9910	529.9983
	632.3287	9.6058	1.4868	.0332	.0085
80	-537.2855	1573.8307	1167.9499	939.9578	529.9898
	326.7667	14.9286	3.6142	.1060	.0340
100	-864.0522	1558.9021	1164.3357	939.8518	529.9558
	-14.3010	19.1477	7.0934	.2713	.1072
120	-849.7512	1539.7544	1157.2423	939.5805	529.8486
	-280.3656	21.0864	11.7453	.5842	.2791
140	-569.3856	1518.6680	1145.4970	938.9963	529.5695
	-406.4720	20.4436	16.8677	1.0958	.6214
160	-162.9136	1498.2244	1128.6293	937.9005	528.9481
	-383.4399	17.8688	21.3681	1.8403	1.2119
180	220.5263	1480.3556	1107.2612	936.0602	527.7362
	-249.0991	14.6444	24.0928	2.8262	2.1096
200	469.6254	1465.7112	1083.1684	933.2340	525.6266
	-67.1182	12.1421	24.2316	4.0324	3.3298
220	536.7436	1453.5691	1058.9368	929.2016	522.2968

Time in Seconds	h_1	h_2	h_3	h_4	h_5
0	0	0	0	0	0
20	14.0738	.02665718	.00098268	.00001014	.00000212
40	20.98083	.1393760	.00881134	.00012260	.00002562
60	20.11513	.3635872	.03905746	.00074638	.00024644
80	13.34705	.6545042	.1157222	.0029134	.00127470
100	3.945110	.9277336	.2605124	.00893988	.00482174
120	-4.701903	1.101672	.4779024	.02158966	.01437790
140	-10.13476	1.133212	.7447730	.04413448	.03562766
160	-11.37532	1.033546	1.011141	.07915180	.07571982
180	-8.91592	.860313	1.213267	.1279072	.1414184
200	-4.266150	.691807	1.294781	.1900690	.2365710
220	.714322	.595095	1.227717	.2636166	.3600630

Time in Seconds	(1) $.0002222222Q_2$	(2) $-.009746032Q_1$	(3) Col (1)+(2)	(4) Col (3) + .04666666	(5) Col (4) - $2h_1$ = (U)	(6) $(Q_3 - Q_2)$	(7) Col (6) x .008510638	(8) $(Q_1 - Q_2)$	(9) Col (8) x .0002222222	(10) Col (7)+(9)	(11) Col (10) + 3.527801	(12) Col (11) - $2h_2$ = (V)
0												
20	.3533333	-17.54286	-17.18953	-17.14286	-17.14286	-420.0000	-3.574468	210.0000	.04666666	-3.527801	0	0
40	.3532774	-14.95088	-14.59760	-14.55093	-42.69871	-419.7579	-3.572408	-55.7001	-.01237780	-3.584786	-.056985	-.1102994
60	.3529293	-8.494893	-8.141964	-8.095297	-50.05696	-418.2855	-3.559876	-716.5562	-.1592347	-3.719111	-.191310	-.470062
80	.3518747	-.9262941	-.5744194	-.5277527	-40.75801	-413.9998	-3.523402	-1488.393	-.3307540	-3.854156	-.326355	-1.053529
100	.3497401	5.236402	5.586142	5.632809	-21.06129	-405.8808	-3.454305	-2111.116	-.4691368	-3.923442	-.3956410	-1.704649
120	.3464227	8.421080	8.767503	8.814170	.923950	-394.5664	-3.358012	-2422.954	-.5384342	-3.896446	-.368645	-2.224112
140	.3421676	8.281702	8.623870	8.670537	18.07434	-382.5121	-3.255422	-2389.506	-.5310013	-3.786423	-.258622	-2.461966
160	.3374817	5.549250	5.886732	5.933399	26.20292	-373.1710	-3.175923	-2088.0536	-.4640119	-3.639935	-.1121340	-2.378558
180	.3329387	1.587761	1.920700	1.967367	24.71801	-369.5951	-3.145490	-1661.1380	-.3691417	-3.514632	.013169	-2.053923
200	.3289679	-2.149256	-1.820288	-1.773622	16.05822	-373.0944	-3.175271	-1259.8293	-.2799620	-3.455233	.072568	-1.648058
220	.3257136	-4.576984	-4.251270	-4.204603	4.32770	-382.5428	-3.255683	-996.0858	-.2213524	-3.477035	.050766	-1.332848
240	.3230153	-5.231120	-4.908105	-4.861438	-6.290082	-394.6323	-3.358573	-916.8255	-.2037390	-3.562312	-.034511	-1.224701

Time in Seconds	(1) $(Q_2 - Q_3)$	(2) Col (1) x .008510638	(3) $(Q_4 - Q_3)$	(4) Col (3) x .002721088	(5) Col (2)+(4)	(6) Col (5) - 2.948618	(7) Col (6) - $2h_3$ = (W)
0							
20	420.0000	3.574468	-230.0000	-.6258502	2.948618	0	0
40	419.7579	3.572408	-229.9907	-.6258249	2.946583	-.002035	-.0040004
60	418.2855	3.559876	-229.8978	-.6255721	2.934304	-.014314	-.031937
80	413.9998	3.523402	-229.4457	-.6243419	2.899060	-.049558	-.127673
100	405.8808	3.454305	-227.9921	-.6203866	2.833918	-.114700	-.346144
120	394.5664	3.358012	-224.4839	-.6108404	2.747172	-.201446	-.722471
140	382.5121	3.255422	-217.6618	-.5922769	2.663145	-.285473	-1.241278
160	373.1710	3.175923	-206.5007	-.5619066	2.614016	-.334602	-1.824148
180	369.5951	3.145490	-190.7288	-.5189898	2.626500	-.322118	-2.344400
200	373.0944	3.175271	-171.2010	-.4658530	2.709418	-.239200	-2.665734
220	382.5428	3.255683	-149.9344	-.4079847	2.847698	-.100920	-2.690482
240	394.6323	3.358573	-129.7352	-.3530209	3.005552	.056934	-2.398500

Time in Seconds	(1) $(Q_3 - Q_4)$	(2) Col (1) x .002721088	(3) $(Q_5 - Q_4)$	(4) Col (3) x .05714286	(5) Col (2)+(4)	(6) Col (5) + 22.80272	(7) Col (6) - $2h_4$ = (X)	(8) $(Q_4 - Q_5)$	(9) Col (8) x .05714286	(10) Col (9) - 23.42857	(11) Col (10) - $2h_5$ = (Y)
0											
20	230.0000	.6258502	-410.0000	-23.42857	-22.80272	0	0	410.0000	23.42857	0	0
40	229.9907	.6258249	-409.9999	-23.42857	-22.80274	-.00002	-.00004	409.9999	23.42857	0	0
60	229.8978	.6255721	-409.9989	-23.42851	-22.80294	-.00022	-.00046	409.9989	23.42851	-.00006	-.00011
80	229.4457	.6243419	-409.9927	-23.42816	-22.80382	-.00110	-.00259	409.9927	23.42816	-.00041	-.00090
100	227.9921	.6203866	-409.9680	-23.42674	-22.80635	-.00363	-.00961	409.9680	23.42674	-.00183	-.00438
120	224.4839	.6108404	-409.8960	-23.42263	-22.81179	-.00907	-.02695	409.8960	23.42263	-.00594	-.01558
140	217.6618	.5922769	-409.7319	-23.41325	-22.82097	-.01825	-.06143	409.7319	23.41325	-.01532	-.04408
160	206.5007	.5619066	-409.4268	-23.39582	-22.83391	-.03119	-.11946	409.4268	23.39582	-.03275	-.10401
180	190.7288	.5189898	-408.9524	-23.36871	-22.84972	-.04700	-.20530	408.9524	23.36871	-.05986	-.21130
200	171.2010	.4658530	-408.3240	-23.33280	-22.86695	-.06423	-.32004	408.3240	23.33280	-.09577	-.37861
220	149.9344	.4079847	-407.6074	-23.29185	-22.88387	-.08115	-.46129	407.6074	23.29185	-.13672	-.60986
240	129.7352	.3530209	-406.9048	-23.25170	-22.89868	-.09596	-.62320	406.9048	23.25170	-.17687	-.89700

Time in Seconds	(1) -15.51384U	(2) -.01467165U	(3) -.0005506610U	(4) -.000005260578U	(5) -.00000006902498U
0					
20	265.9516	.2515140	.00943990	.00009018	.00001183
40	662.4210	.6264605	.02351251	.00022462	.00002947
60	776.5757	.7344182	.02756442	.00026333	.00003455
80	632.3132	.5979873	.02244385	.00021441	.00002813
100	326.7415	.3090039	.01159763	.00011079	.00001454
120	-14.33401	-.01355587	-.00050878	-.00000486	-.00000064
140	-280.4024	-.2651804	-.00995283	-.00009508	-.00001248
160	-406.5079	-.3844401	-.01442893	-.00013784	-.00001809
180	-383.4713	-.3626540	-.01361124	-.00013003	-.00001706
200	-249.1247	-.2356006	-.00884264	-.00008448	-.00001108
220	-67.13925	-.06349450	-.00238310	-.00002277	-.00000299
240	97.58333	.09228588	.00346370	.00003309	.00000434

Time in Seconds	(1) -.01467165V	(2) -8.511434V	(3) -.3194539V	(4) -.003051808V	(5) -.0004004332V	(1) -.0005506610W	(2) -.3194538W	(3) -8.832085W	(4) -.08437472W	(5) -.01107096W
0										
20	0	0	0	0	0	0	0	0	0	0
40	.00161827	.9388061	.03523557	.00033661	.00004417	.00000220	.00127794	.0353319	.00033753	.00004429
60	.00689659	4.000902	.1501631	.00143454	.00018823	.00001759	.01020240	.282070	.00269468	.00035357
80	.01545701	8.967043	.3365539	.00321517	.00042187	.00007030	.04078563	1.127619	.01077237	.00141346
100	.02501001	14.50901	.5445568	.00520226	.00068260	.00019061	.1105770	3.057173	.02920580	.00383215
120	.03263139	18.93038	.7105013	.00678756	.00089061	.00039784	.2307961	6.380925	.06095829	.00799845
140	.03612110	20.95486	.7864846	.00751345	.00098585	.00068352	.3965310	10.96307	.1047325	.01374214
160	.03489737	20.24494	.7598396	.00725890	.00095245	.00100449	.5827310	16.11103	.1539120	.02019507
180	.03013444	17.48183	.6561337	.00626818	.00082246	.00129097	.7489275	20.70594	.1978081	.02595476
200	.02417973	14.02734	.5264786	.00502956	.00065994	.00146792	.8515789	23.54399	.2249206	.02951223
220	.01955508	11.34445	.4257835	.00406760	.00053372	.00148154	.8594847	23.76257	.2270087	.02978622
240	.01796838	10.42396	.3912355	.00373755	.00049041	.00132076	.7662099	21.18376	.2023728	.02655370

Time in Seconds	(1) -.000005260578X	(2) -.003051808X	(3) -.08437472X	(4) -7.022412X	(5) -.9214232X	(1) .-0000006902498Y	(2) -.0004004332Y	(3) -.01107096Y	(4) -.9214232Y	(5) -4.713314Y	(1) Σ (1) = q_1	(2) Σ (2) = q_2	(3) Σ (3) = q_3	(4) Σ (4) = q_4	(5) Σ (5) = q_5
0															
20	0	0	0	0	0	0	0	0	0	0	265.9516	.2515140	.00943990	.00009018	.00001183
40	0	.00000012	.00000337	.000281	.0000369	0	0	0	0	0	662.4226	1.566545	.09408335	.00117976	.00015483
60	0	.00000140	.0000388	.00323	.000424	0	.00000004	.00000122	.000101	.000518	776.5826	4.745524	.4598375	.00772355	.00151835
80	.00000001	.00000790	.0002185	.01819	.002386	0	.00000036	.00000996	.000829	.00424	632.3287	9.605824	1.486845	.03322095	.00848946
100	.00000005	.00002933	.0008108	.06749	.008855	0	.00000175	.00004849	.004036	.02064	326.7667	14.92862	3.614187	.1060448	.03402429
120	.00000014	.00008225	.0022739	.18925	.024832	.00000001	.00000624	.00017249	.014356	.073433	-14.30098	19.14771	7.093364	.2713470	.1071534
140	.00000032	.00018747	.0051831	.43139	.056603	.00000003	.00001765	.00048801	.040616	.207763	-280.3656	21.08642	11.74527	.5841569	.2790815
160	.00000063	.00036459	.010079	.83890	.11007	.00000007	.00004165	.00115150	.095837	.490232	-406.4720	20.44364	16.86767	1.095770	.6214314
180	.00000108	.00062654	.0173221	1.44170	.189168	.00000015	.00008461	.00233930	.194697	.995923	-383.4399	17.86881	21.36812	1.840343	1.211851
200	.00000168	.00097670	.0270033	2.24745	.294892	.00000026	.00015161	.00419158	.348860	1.78451	-249.0991	14.64445	24.09282	2.826176	2.109563
220	.00000243	.00140777	.0389212	3.23937	.425043	.00000042	.00024421	.00675174	.561939	2.87446	-67.11821	12.14209	24.23164	4.032363	3.329820
240	.00000328	.00190189	.0525823	4.37637	.574231	.00000062	.00035919	.00993065	.826517	4.22784	97.60262	11.28472	21.64097	5.409030	4.829119

General Notes: Sheet 2 is used to evaluate (q)s from equations developed on sheet 1. These q values are the bases of determining water surface elevations (Z) shown on sheet 3.

Sheet 2 is used in conjunction with sheet 3. New values of Q and h are inserted as sheet 3 computations progress. The Q values for each period are determined by subtracting the retardation in discharge (q) from the Q values at the beginning of the preceding period. The h values shown are h_e values of the preceding period shown on sheet 3.

Values of Q and h at the beginning of the first period are known since the Q values are those for steady flow conditions at the instant of stoppage and all the h values are zero.

COMPLETE SHUTDOWN OF STATE FAIRGROUNDS PUMPING PLANT

(1800 CFS TO 0 CFS)

Tank Item	1	2	3	4	5	1	2	3	4	5	1	2	3	4	5
L/tga	.02645949	.05299344	.05204969	.05625555	.08958952	.02645949	.05299344	.05204969	.05625555	.08958952	.02645949	.05299344	.05204969	.05625555	.08958952
C	.006666666	.007295598	.003675214	.003723404	.01000000	.006666666	.007295598	.003675214	.003723404	.01000000	.006666666	.007295598	.003675214	.003723404	.01000000
			0-20					20-40					40-60		
(1) q	265.9516	.2515140	.00943990	.00009018	.00001183	662.4226	1.566545	.09408335	.00117976	.00015483	776.5826	4.745524	.4598375	.00772355	.00151835
(2) $\Delta F = Cq$	1.773010	.00183495	.00003469	.00000034	.00000012	4.416150	.01142888	.00034578	.00000439	.00000155	5.177217	.03462144	.00169000	.00002876	.00001518
(3) $\Sigma \Delta F$	1.774880	.00187010	.00003515	.00000046	.00000012	4.427931	.01178060	.00035172	.00000594	.00000155	5.213572	.03635538	.00173394	.00004394	.00001518
(4) h_b	0	0	0	0	0	14.07389	.02665718	.00098268	.00001014	.00000212	20.98083	.1393760	.00881134	.00012260	.00002562
(5) $h_a = q\ L/tga$	7.036944	.01332859	.00049134	.00000507	.00000106	17.52736	.08301661	.00489701	.00006637	.00001387	20.54798	.2514816	.02393440	.00043449	.00013603
(6) h_e	14.07389	.02665718	.00098268	.00001014	.00000212	20.98083	.1393760	.00881134	.00012260	.00002562	20.11513	.3635872	.03905746	.00074638	.00024644
(7) Σh_b	0	0	0	0	0	14.10154	.02765212	.00099494	.00001226	.00000212	21.12917	.1483356	.00895956	.00014822	.00002562
(8) Σh_e	14.10154	.02765212	.00099494	.00001226	.00000212	21.12917	.1483356	.00895956	.00014822	.00002562	20.51877	.4036375	.04005028	.00099282	.00024644
(9) $\Delta Z = \Sigma h_e - \Sigma h_b + \Sigma \Delta F$	15.87642	.02952222	.00103009	.00001272	.00000224	11.45556	.1324641	.00831634	.00014190	.00002505	4.603172	.2916573	.03282466	.00088854	.00023600
(10) ΔZ check	15.87642					11.45559					4.603186				
			60-80					80-100					100-120		
(1) q	632.3287	9.605824	1.486845	.03322095	.00848946	326.7667	14.92862	3.614187	.1060448	.03402429	-14.30098	19.14771	7.093364	.2713470	.1071534
(2) $\Delta F = Cq$	4.215524	.07008023	.00546447	.00012370	.00008489	2.178444	.1089132	.01328291	.00039485	.00034024	-.0953399	.1396940	.02606963	.00101033	.00107153
(3) $\Sigma \Delta F$	4.291277	.07575329	.00567306	.00020859	.00008489	2.301375	.1229312	.0140180	.00073509	.00034024	.07250559	.1678455	.02815149	.00208186	.00107153
(4) h_b	20.11513	.3635872	.03905746	.00074638	.00024644	13.34705	.6545042	.1157222	.00299134	.00127470	3.945110	.9277336	.2605124	.00893988	.00482174
(5) $h_a = q\ L/tga$	16.73109	.5090457	.07738982	.00186886	.00076057	8.646080	.7911189	.1881173	.00596561	.00304822	-.3783966	1.014703	.3692074	.01526477	.00959982
(6) h_e	13.34705	.6545042	.1157222	.00299134	.00127470	3.945110	.9277336	.2605124	.00893988	.00482174	-4.701903	1.101672	.4779024	.02158966	.01437790
(7) Σh_b	20.51877	.4036375	.04005028	.00099282	.00024644	14.12154	.7744924	.1199882	.00426604	.00127470	5.147118	1.202008	.2742740	.01376162	.00482174
(8) Σh_e	14.12154	.7744924	.1199882	.00426604	.00127470	5.147118	1.202008	.274274	.01376162	.00482174	-3.086361	1.615542	.5138700	.03596756	.01437790
(9) $\Delta Z = \Sigma h_e - \Sigma h_b + \Sigma \Delta F$	-2.105953	.4466082	.0856110	.00348181	.00111315	-6.673047	.5504468	.1683038	.01023067	.00388728	-8.160973	.5813795	.2677475	.0242878	.01062769
(10) ΔZ check	-2.105916					-6.673037					-8.160969				
			120-140					140-160					160-180		
(1) q	-280.3656	21.08642	11.74527	.5841569	.2790815	-406.4720	20.44364	16.86767	1.095770	.6214314	-383.4399	17.86881	21.36812	1.840343	1.211851
(2) $\Delta F = Cq$	-1.869104	.1538380	.04316638	.00217505	.00279082	-2.709813	.1491486	.06199230	.00407999	.00621431	-2.556266	.1303637	.07853241	.00685234	.01211851
(3) $\Sigma \Delta F$	-1.667134	.2019703	.04813225	.00496587	.00279082	-2.488378	.2214352	.07228660	.01029430	.00621431	-2.328399	.2278670	.09750326	.01897085	.01211851
(4) h_b	-4.701903	1.101672	.4779024	.02158966	.01437790	-10.13476	1.133212	.7447730	.04413448	.03562766	-11.37532	1.033546	1.011141	.07915180	.07571982
(5) $h_a = q\ L/tga$	-7.418331	1.117442	.6113377	.03286207	.02500278	-10.75504	1.083379	.8779570	.06164314	.05567374	-10.14562	.9469297	1.112204	.1035295	.1085691
(6) h_e	-10.13476	1.133212	.7447730	.04413448	.03562766	-11.37532	1.033546	1.011141	.07915180	.07571982	-8.91592	.860313	1.213267	.1279072	.1414184
(7) Σh_b	-3.086361	1.615542	.5138700	.03596756	.01437790	-8.17701	1.957747	.8245351	.07976214	.03562766	-9.17576	2.199559	1.166013	.1548716	.07571982
(8) Σh_e	-8.17701	1.957747	.8245351	.07976214	.03562766	-9.17576	2.199559	1.166013	.1548716	.07571982	-6.57301	2.342906	1.482593	.2693256	.1414184
(9) $\Delta Z = \Sigma h_e - \Sigma h_b + \Sigma \Delta F$	-6.75778	.544175	.3587974	.04876045	.02404058	-3.48713	.463247	.413765	.0854038	.04630647	.27435	.371214	.414083	.1334249	.07781709
(10) ΔZ check	-6.75779					-3.48714					.27435				
			180-200					200-220					220-240		
(1) q	-249.0991	14.64445	24.09282	2.826176	2.109563	-67.11321	12.14209	24.23164	4.032363	3.32982	97.60262	11.28472	21.64097	5.409030	4.829119
(2) $\Delta F = Cq$	-1.660661	.1068400	.08854627	.01052300	.02109563	-.4474547	.08858381	.08905646	.01501412	.0332982	.6506841	.08232878	.07953520	.02014000	.04829119
(3) $\Sigma \Delta F$	-1.433656	.2270049	.1201649	.03161863	.02109563	-.2215021	.2259526	.1373688	.04831232	.0332982	.8809793	.2302952	.1479664	.06843119	.04829119
(4) h_b	-8.91592	.860313	1.213267	.1279072	.1414184	-4.26615	.691807	1.294781	.1900690	.2365710	.714322	.595095	1.227717	.2636166	.3600630
(5) $h_a = q\ L/tga$	-6.591035	.7760598	1.254024	.1589881	.1889947	-1.775914	.6434511	1.261249	.2268428	.2983170	2.582516	.5980161	1.126406	.3042880	.4326385
(6) h_e	-4.26615	.691807	1.294781	.1900690	.2365710	.714322	.595095	1.227717	.2636166	.3600630	4.450710	.6009372	1.025095	.3449594	.5052140
(7) Σh_b	-6.57301	2.342906	1.482593	.2693256	.1414184	-1.85292	2.413228	1.721421	.4266400	.2365710	3.160814	2.446492	1.851397	.6236796	.3600630
(8) Σh_e	-1.85292	2.413228	1.721421	.4266400	.2365710	3.160814	2.446492	1.851397	.6236796	.3600630	6.926916	2.476206	1.875268	.8501734	.5052140
(9) $\Delta Z = \Sigma h_e - \Sigma h_b + \Sigma \Delta F$	3.28643	.297327	.358993	.188933	.116248	4.792232	.2592166	.2673448	.2453519	.1567902	4.647081	.2600092	.1718374	.2949250	.1934422
(10) ΔZ check	3.28644					4.792233					4.647070				

Cross Check

$$Q_o + \frac{\Delta Z_1 A_1}{t} = 1800 - \Sigma q_{1(t-1)} - \frac{q_{1t}}{2} \qquad \text{Or:} \qquad 0 + 105\ \Delta Z_1 = 1800 - \Sigma q_{1(t-1)} - \frac{q_{1t}}{2}$$

Final Tabulations

Time in Seconds (1)	$\Sigma q_{1(t-1)}$ (2)	$\frac{q_{1t}}{2}$ (3)	1800-col (2)-(3) (4)	105 ΔZ (5)	$\Delta Z = \frac{\text{col 4}}{105}$ (6)	Tank 1 ΔZ_1	Tank 1 Z_1	Tank 2 ΔZ_2	Tank 2 Z_2	Tank 3 ΔZ_3	Tank 3 Z_3	Tank 4 ΔZ_4	Tank 4 Z_4	Tank 5 ΔZ_5	Tank 5 Z_5
0	0						416.000		428.000		439.600		443.900		447.400
20	0	132.9758	1667.0242	1667.0241	15.87642	15.876	431.876	.030	428.030	.001	439.601	0	443.900	0	447.400
40	265.9516	331.2113	1202.8371	1202.8338	11.45559	11.456	443.332	.132	428.162	.008	439.609	0	443.900	0	447.400
60	928.3742	388.2913	483.3345	483.3331	4.603186	4.603	447.935	.292	428.454	.033	439.642	.001	443.901	0	447.400
80	1704.9568	316.1644	-221.1212	-221.1251	-2.105916	-2.106	445.829	.447	428.901	.086	439.728	.003	443.904	.001	447.401
100	2337.2855	163.3834	-700.6689	-700.6699	-6.673037	-6.673	439.156	.550	429.451	.168	439.896	.010	443.914	.004	447.405
120	2664.0522	-7.1505	-856.9017	-856.9022	-8.160969	-8.161	430.995	.581	430.032	.268	440.164	.024	443.938	.011	447.416
140	2649.7512	-140.1828	-709.5684	-709.5669	-6.757794	-6.758	424.237	.544	430.576	.359	440.523	.049	443.987	.024	447.440
160	2369.3856	-203.2360	-366.1496	-366.1487	-3.487139	-3.487	420.750	.463	431.039	.414	440.937	.085	444.072	.046	447.486
180	1962.9136	-191.7200	28.8064	28.8068	.27435	.274	421.024	.371	431.410	.414	441.351	.133	444.205	.078	447.564
200	1579.4737	-124.5496	345.0759	345.0752	3.28644	3.286	424.310	.297	431.707	.359	441.710	.189	444.394	.116	447.680
220	1330.3746	-33.5591	503.1845	503.1844	4.792233	4.792	429.102	.259	431.966	.267	441.977	.245	444.639	.157	447.837
240	1263.2564	48.8013	487.9423	487.9435	4.647070	4.647	433.749	.260	432.226	.172	442.149	.295	444.934	.193	448.030

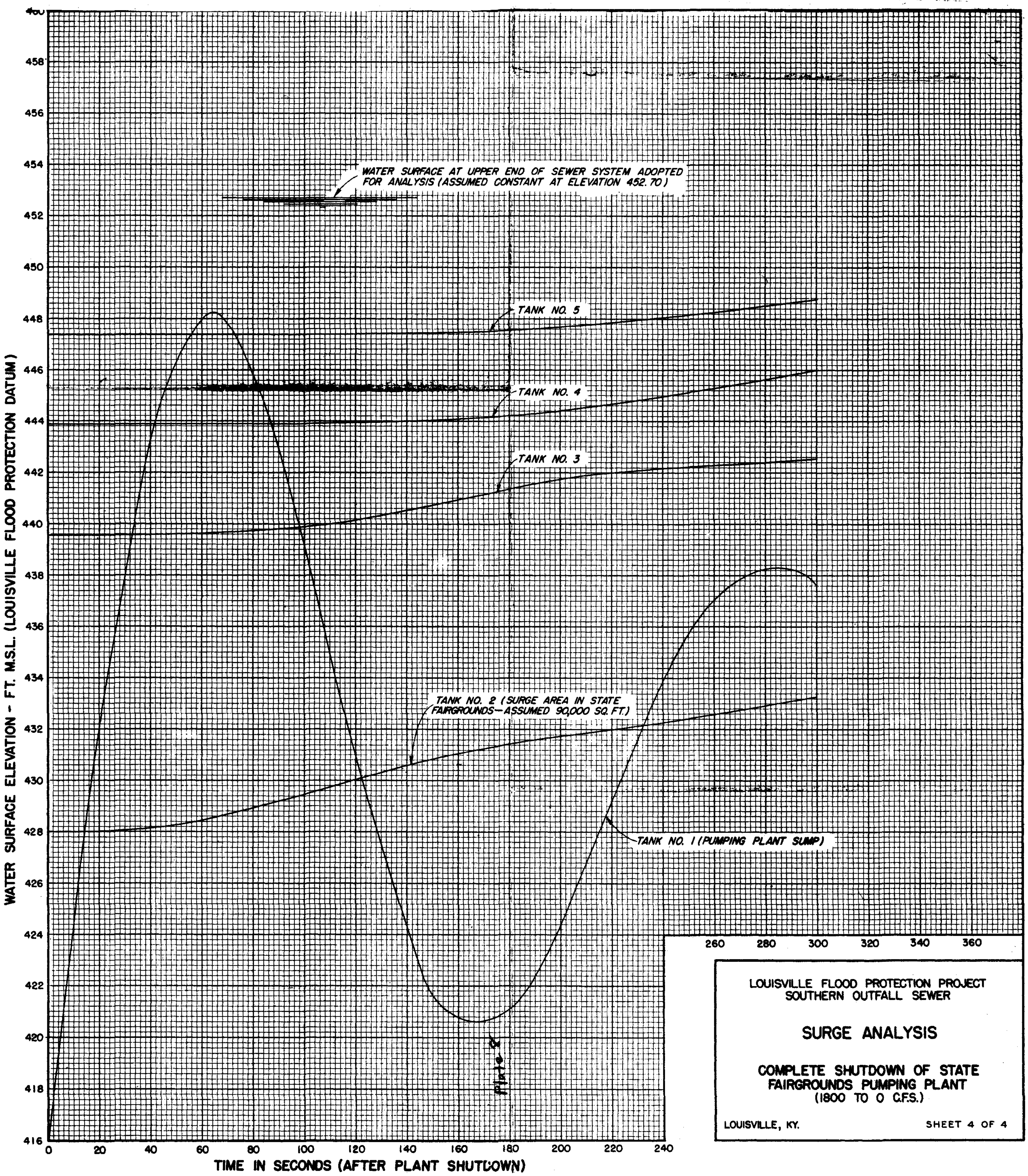
The items shown in a typical 20-second computation are obtained as follows:

(1)	q	computed on sheet 2
(2)	ΔF	Cq (as shown)
(3)	$\Sigma \Delta F$	summation of hydraulic loss changes of reaches upstream from tank during time period considered
(4)	h_b	same as h_e of previous time period
(5)	h_a	q L/tga (as shown)
(6)	h_e	$2h_a - h_b$
(7)	Σh_b	summation of h_b of reaches upstream from tank
(8)	Σh_e	summation of h_e of reaches upstream from tank
(9)	ΔZ	$\Sigma h_e - \Sigma h_b + \Sigma \Delta F$ (as shown)
(10)	ΔZ check	volumetric check as shown in column (6) of cross check and final tabulations.

COMPLETE SHUTDOWN OF STATE FAIRGROUNDS PUMPING PLANT

(1800 CFS TO 0 CFS)

SHEET 3 OF 4



LOUISVILLE FLOOD PROTECTION PROJECT
SOUTHERN OUTFALL SEWER

SURGE ANALYSIS

COMPLETE SHUTDOWN OF STATE FAIRGROUNDS PUMPING PLANT
(1800 TO 0 C.F.S.)

LOUISVILLE, KY. SHEET 4 OF 4

PLATE B5