HYDRAULIC ANALYSIS OF THE PROPOSED ADDITIONAL TREATMENT FACILITIES

by

Charles C. S. Song and

S. Gavali

Prepared for

MONROE COUNTY DEPARTMENT OF ENGINEERING
Rochester Pure Waters District

Submitted to

O'BRIEN & GERE ENGINEERS, INC.
Syracuse, New York

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Minneapolis, Minnesota
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I. INTRODUCTION

The existing Van Lare Treatment Plant has a design capacity of 150 MGD. As a part of the combined sewer overflow abatement program an additional primary treatment facility is being designed. The new primary treatment facility will be located next to the existing facility and will have the design treatment capacity of 270 MGD and the hydraulic capacity of 400 MGD.

A new distribution structure will be constructed to collect the flows coming from the existing St. Paul Boulevard Interceptor, the Cross-Irondequoit pump station, and the new West Side System and will distribute flow to the new and the old treatment facilities.

The existing facility disposes the treated water through the existing 120 in. diameter outfall. There is also an old 66 in. outfall which is not currently in use but could be reactivated during high flow periods. Both outfalls are connected to an existing distribution structure. It is proposed to construct a new diversion structure on the existing 120 in. line immediately downstream of the existing distribution structure. The outflow from the new treatment facility will be conveyed to the new distribution structure. The combined outfall capacity will be approximately equal to the combined hydraulic capacity of the expanded treatment plant. A new flow control gate is to be added to the existing diversion structure so that the 66 in. outfall will be activated only during large flow periods.

Because the combined new and old treatment plant will operate under different modes over a wide range of flow rate, unsteady free-surface and pressurized flow conditions exist. To assist the hydraulic design of the system, an unsteady flow mathematical model study was carried out.

The total system is divided into two subsystems: the treatment facility and the outfall facility. The flow in the treatment facility is expected to change very slowly so that a kinematic model should suffice. The flow in the outfall subsystem is of unsteady mixed-flow type requiring a dynamic model.
II. DESCRIPTION OF MATHEMATICAL MODELS

A. Overall System Configuration

The overall system configuration is sketched in Fig. 1. The influent distribution structure (IDS) is divided into two reservoirs separated by a weir. The five inflows shown are:

- QSP = inflow from St. Paul Boulevard tunnel
- QFD = dry weather flow from force main
- QFW = wet weather flow from force main
- QWD = dry weather flow from West Side System
- QWW = wet weather flow from West Side System

The existing grid chambers are lumped to form Reservoir No. 2 which receives flow from the dry weather flow compartment of IDS, Reservoir No. 1. The flow over the weir downstream of the grit chamber goes into a distribution system represented by Reservoir No. 3. The two primary basins are lumped and represented by Reservoir No. 4. In a similar manner, all aeration basins are lumped as Reservoir No. 6, and the final basins are represented by Reservoir No. 8. The chlorine contact basin is represented by Reservoir No. 9 to which a critical flow measuring flume is attached. This is the downstream end of the existing treatment facility.

The new treatment facility starts at the wet weather flow compartment of IDS, represented by Reservoir No. 10. The new screening facility is divided into two reservoirs, Nos. 11 and 12. Since the screening facility consists of three parallel compartments which may not be operating simultaneously all the time, three gates are used to separate the two reservoirs. The number of gates open will depend on the flow rate. The new grit removal facility is represented by Reservoir No. 13. A weir at reservoir 13 represents the downstream end of the new treatment facility. The combined new and old treatment facility is shown in Fig. 1 as model I.

The outfall model is shown in Fig. 1 as Model II. This model has two upstream ends and two downstream ends. The outflow from the existing treatment facility enters the existing 120 inch pipe as an inflow and flows to the old distribution structure (ODS). The outflow from the new treatment facility enters a new 120 inch pipe as another inflow to the outfall model. This portion of the inflow is led to the new distribution structure (NDS) which is connected to ODS by a 120 inch pipe. The existing 120 inch outfall pipe is connected to NDS and the existing 66 inch outfall pipe is connected to ODS.
B. Alternative Treatment Facility Model Configuration

The model configuration described above assumes that the new treatment facility is to be used only during high flow periods. The second alternative considered herein assumes that the new facility will replace the existing grid chambers. As shown in Fig. 2, the outflow from GRF is routed to PB during normal conditions. In this case, GC is isolated from IDS and acts as a dead end.

C. The Dynamic Model

Large portions of the existing outflow pipes are submerged under water all the time, and the flow there is a pressurized type. For this reason the flow in the system will be either a mixed type or a pressurized type. The flow could be very dynamic under variable inflow conditions.

The transient mixed-flow model developed at the St. Anthony Falls Hydraulic Laboratory was adopted for this model. However, because of the uniqueness of the system configuration having NDS and ODS located so close to each other, a special junction boundary condition was needed. This boundary condition is quite complex and took considerable amounts of development effort.

Figure 3 shows the dynamic model configuration and the variables related to the junction boundary condition. The entrance to ODS, the entrance to NDS, and the exit of NDS are represented by Stations 3, 9, and 10, respectively. There are three unknowns at each of the three stations. If a pressurization interface is located near a station, then the location and speed of the interface must also be considered with other variables. Because the 66 inch outfall is not to be used frequently, it is lumped and the discharge \( Q_0 \) is calculated by a steady flow equation. The discharge between the two diversion structures, \( Q_{ON} \), and the depths in the structures \( y_N \) and \( y_0 \) are also unknowns. Therefore, there are at least 13 unknowns to be solved simultaneously. (Two additional variables must be added for each pressurization interface.) The required equations are listed as follows.

1. Compatibility condition, \( y_3 = y_0 \)
2. Compatibility condition, \( y_9 = y_N \)
3. Compatibility condition, \( y_{10} = y_N \), if station 10 is pressurized. Otherwise, critical flow condition is assumed to exist because the slope of the pipe is steep.
4. Positive characteristic equation at Station 3.
5. Positive characteristic equation at Station 9.
6. Negative characteristic equation at Station 10, if it is pressurized there. Otherwise assume the depth to be equal
to normal depth there.

7. Orifice equation, \( Q_{ON} = \pm C_D A_{AN} \sqrt{2g|y_0 - y_N|} \).

8. Energy equation, \( Q_0 = K \sqrt{2g(y_0 - y_L)} \), where \( y_L \) is the lake level.

9. Storage equation for ND, \( A_N \frac{dy_N}{dt} = Q_9 + Q_{ON} - Q_{10} \).

10. Storage equation for OD, \( A_0 \frac{dy_0}{dt} = Q_3 - Q_{ON} - Q_0 \).

11. \( Q_3 = V_3 A_3 \).

12. \( Q_9 = V_9 A_9 \).

13. \( Q_{10} = V_{10} A_{10} \).

These 13 or more simultaneous equations are solved by the Runge-Kutta method.

The shock boundary condition related to the interface and other boundary conditions are referred to in other papers [1, 2].

D. The Kinematic Model

Because the treatment facility consists of a series of relatively large reservoirs joined by weirs or pipes, the flow is expected to be not very dynamic. For this reason, a kinematic model based on the unsteady storage equation of the following form is used to represent the condition of each reservoir.

\[
A \frac{dy}{dt} = \Sigma Q_i \tag{1}
\]

in which \( Q_i \) is either represented by a weir equation or a steady flow equation through a short pipe. There are as many equations like Eq. 1 as the number of reservoirs modeled. The system of first order differential equations were solved by the Runge-Kutta method.
III. MODELING RESULTS

A. Model Calibration

The existing treatment facility portion of the kinematic model was calibrated using the flow and water level data taken in the field by O'Brien and Gere Engineers. The model is sensitive to the weir elevations but not very sensitive to energy losses in pipes. The water surface elevation in Reservoir 13 was found to be sensitive to the size of the critical flow measuring flume at its exit. Very little adjustment of parameters were needed, and the calibration was completed in three runs.

B. Description of Kinematic Model Runs

A total of six kinematic model runs as described below were carried out.

1. Run No. K1

Model I configuration, as sketched in Fig. 1, was used for this run. The two-year design hydrographs furnished by O'Brien and Gere Engineers were divided into two groups as shown in Fig. 1. Hydrographs QSP, QFD, and QWD were assumed to enter Reservoir 1 and hydrographs QFW and QWW were introduced to Reservoir 10.

In order to avoid instability and excessive computational requirement, all existing reservoirs were assumed to be initially filled to the top of their weirs, and the inflows start at 120 minutes. This is the time when the combined hydrograph starts to rise sharply. There was assumed to have a flow regulating device that limits the flow between Reservoir 1 and Reservoir 2 to 150 MGD maximum. A bypass was located between Reservoirs 3 and 9 so that only up to 90 MGD will pass through Reservoirs 4, 5, 6, 7, and 8.

The outfall hydrographs for reservoirs 1, 9, and 13 are shown in Fig. 4. The corresponding water surface elevations are plotted in Fig. 5. The flow at these reservoirs are quite stable. Because the flow into the existing facility was limited to 150 MGD, the last reservoir in the chain reached an equilibrium condition in about one hour after the beginning of the flow.

2. Run No. K2

This run differs with Run No. K1 only in that the entire hydrographs are inputted to the dry weather compartment of IDS, Reservoir 1. The predicted hydrographs from Reservoirs 1, 9, and 13 are shown in
Fig. 6. The corresponding water depth changes are plotted in Fig. 7. There is hardly any change in the system's response as compared with Run K1.

3. Run No. K3

Originally this run was intended to be only a small modification of Run K1 by removing the flow regulator between Reservoirs 1 and 2. However, it was found that the model was numerically not very stable. Because Reservoir 8 had a very long weir, but Reservoir 9 had a very short weir, the two reservoirs were acting like a single reservoir. That is, the water surface elevations in these two reservoirs were nearly identical. This means that a small difference in water surface elevation caused a large change in discharge. To avoid this instability problem, the two reservoirs were combined and named Reservoir 8. Therefore, this system has only 12 reservoirs.

The computed discharge hydrographs for Reservoirs 1, 8, and 12 are shown in Fig. 8. This graph shows minor instabilities at Reservoirs 1 and 12. By removing the flow regulator, the flow through the existing facility is greatly increased. It appears that the weir in the IDS should be lowered somewhat. The corresponding water surface elevations are shown in Fig. 9.

4. Run No. K4

The condition for this run was the same as that of Run K1 except that the inflow hydrograph distribution was changed slightly. Only QSP was introduced to the dry weather chamber and all other flows were introduced to the wet weather chamber. As in the case of Run K3, the 12 reservoir systems were used.

The outflow hydrographs from Reservoirs 1, 8, and 12 are shown in Fig. 10. The corresponding water surface elevations are shown in Fig. 11.

5. Run No. K5

The run was made for the alternative configuration shown in Fig. 2. A constant inflow rate of 90 MGD was introduced at the dry weather chamber of IDS. The flow over the weir in IDS would gradually increase, and the flow in the new treatment facility would start.

Computation shows that the flow will pass through Reservoirs 1 through 10 but no flow occurs at Reservoirs 11 and 12. Figure 12 shows the outflow hydrographs from Reservoirs 1, 6, and 10. The water surface elevations at Reservoirs 1, 6, and 12 are shown in Fig. 13. Note that the water surfaces at Reservoirs 6 and 11 are at the same elevation indicating no flow between the two reservoirs.
6. Run No. K6

The condition for this run is identical to that of Run K5 except that the inflow rate was increased to 150 MGD. Figure 14 shows the resulting outflow hydrographs from Reservoirs 1, 6, and 10. Figure 15 shows the water surface elevations at Reservoirs 1, 6, and 11. Qualitatively, the system responds similarly to 150 MGD and 90 MGD inflows.

C. Description of Dynamic Model Runs

The Dynamic System's configuration and the junction configuration are shown in Fig. 3. The outflow hydrographs of Reservoirs 9 and 13 as calculated by Run K1 were used as the inflow hydrographs for Station 1 and 4, respectively. A total of 3 runs were carried out. These runs are described as follows.

1. Run No. D1

The gate to the 66 inch outfall was assumed closed at all times and \( Q_0 = 0 \). Average lake level was assumed. Stations 12 to 47 were pressurized initially because these stations are submerged under the lake.

As the flow increases, the pressurization interface gradually moves from the lake towards the new distribution structure. The two distribution structures would then be pressurized almost simultaneously and produced two pressurization interfaces, one near Station 3 and the other near Station 9. Because the outfall capacity is limited, the whole system will eventually pressurize and overflow will occur at Station 1. Clearly the 66 inch outfall is needed to take care of the two-year hydrograph.

Instantaneous hydraulic gradelines showing the pressurization process are plotted in Figs. 16 and 17.

2. Run No. D2

The gate to the 66 inch outfall which is initially closed would start to open when the depth in ODS exceeds 14 feet. The gate would open at constant rates until it is fully opened after 10 minutes. Once the gate is opened, it will remain open until the end of the storm runoff to avoid gate instability.

The resulting hydrographs for the two outfalls are shown in Fig. 18. At peak discharge the flow to the 66 inch outfall and the 120 inch outfall are, respectively, 216 cfs and 660 cfs. Instantaneous hydraulic gradelines are shown in Figs. 19 and 20. This run indicates that there is sufficient outfall capacity so that there would be no overflow from the upstream ends. The result also shows that the manholes at the two distribution structures should be at least 32 feet above the floor to avoid overflow. Because the inflows to this system change very slowly and because air pockets are not likely to be produced, the transient pressure in this system is very mild.
3. Run No. D3

This run is identical to Run D2 except that high lake level, 92.72 ft, was assumed.

The results are basically the same as that of Run D2. A rise in lake level of 2.71 ft did not cause any problem to the system. No overflow at the upstream end or large transient pressure occurred. The maximum discharge to the 66 inch outfall and the 120 inch outfall were 225 cfs and 648 cfs, respectively.
IV. CONCLUSIONS AND RECOMMENDATIONS

1. It appears desirable to slightly lower the weir elevation of the proposed inflow distribution structure. This would have the following beneficial effects: (a) provide additional head for incoming flow and (b) cause the natural flow capacity of the existing and the new treatment facilities to more nearly balance.

2. At least 32 feet from the floor to the top of the manhole should be provided for the new and old distribution structures at the outfalls. About 16 ft of space between floor and roof is desirable for these distribution structures.

3. No other undesirable hydraulic characteristics were found for the proposed system. No severe hydraulic transient should exist if gate operations are carefully designed.
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