NUMERICAL MODELING OF HYDRAULIC TRANSIENT WEST SIDE SYSTEM
ROCHESTER, NEW YORK

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I. INTRODUCTION

In May, 1989, under the agreement between the Seelye Stevenson Valve and Knecht of New York and the University of Minnesota, the St. Anthony Falls Hydraulic Laboratory of the University conducted transient modeling of the West Side Sewer System of the Rochester Pure Water District of Monroe County, New York. Three cases representing three stages of construction were modeled [1]. The first case includes the segments of the Lake Avenue Tunnel, the Dewey–Eastman Tunnel, and the Tiger–Carlisle Tunnel, with a temporary bulkhead in the Lake Avenue Tunnel near Structure No. 41. The second case represents the system that exists today, which includes the segments of the Saxton–Colvin Tunnel, Lyell Avenue Tunnel and the Lake Avenue Tunnel. The third case refers to the completed West Side System.

The fully dynamic transient mixed flow numerical model developed at the University of Minnesota was used for this study. This model has been applied to several case studies [2, 3, 4]. Good comparison between numerically simulated transient process and field data has been reported [3].

In addition to sewer back-up and combined sewer overflow, the column consolidation of surge fronts which may occur in the whole West Side System and create large pressure has been examined. Also, a set of safety criteria were used to test whether or not a geyser process will occur.

The proposed West Side System was analyzed in 1982 according to the design hydrographs given at the time. The present analysis has been carried out with the 1989 revised inflow hydrographs. Because the inflow hydrographs have been revised upward, both the column consolidation and geyser problems have been noted this time. Workable solutions to the hydraulic transient problems are recommended.
II. THE MIXED TRANSIENT FLOW MODEL

A. BASIC EQUATION

The equation of continuity and motion for one-dimensional unsteady flow in an open channel can be written as

\[ \frac{\partial v}{\partial t} + v \frac{\partial v}{\partial x} + \frac{C^2}{g} \frac{\partial v}{\partial x} = 0 \] (1)

and

\[ g \frac{\partial v}{\partial x} + \frac{\partial v}{\partial t} + v \frac{\partial v}{\partial x} + g(S_f - S_0) = 0 \] (2)

in which \( C \) is the gravity wave velocity given by

\[ C = \sqrt{\frac{gA}{T}} \] (3)

In the above equation, \( y \) = depth, \( v \) = mean velocity, \( g \) = gravitational acceleration, \( A \) = cross-sectional area of flow, \( T \) = top width of flow, \( S_f \) = friction slope, \( S_0 \) = bed slope and \( t \) = time.

The corresponding equation for a pressurized or closed conduit flow are

\[ \frac{\partial v}{\partial t} + v \frac{\partial v}{\partial x} + \frac{a^2}{g} \frac{\partial v}{\partial x} = 0 \] (4)

\[ \frac{\partial v}{\partial x} + \frac{\partial v}{\partial t} + v \frac{\partial v}{\partial x} + g(S_f - S_0) = 0 \] (5)

in which \( a \) = pressure wave speed and \( y \) should be regarded as the piezometrical head measured from the tunnel invert. These equations and their derivations are quite well known and can be found in text books [7, 8].

The well known method of characteristics is used to solve Eqs. (1), (2), and Eqs. (4), (5).

For open channel flow regimes, the characteristic equations are:

\[ \frac{dv}{dt} \pm \frac{C^2}{g} \frac{dv}{dt} \pm (S_f - S_0) = 0 \] (6)

\[ \frac{dx}{dt} = v \pm C \] (7)
For pressurized flow, the gravity wave speed \( C \) in the above equations is replaced by the pressure wave speed, \( a \).

For mixed flow regimes, because the transition between open channel flow condition and pressurized flow condition is abrupt, as is the case of a hydraulic jump, the special shock boundary condition must be applied. Song [6] showed that, for a pressurization surge or a positive surge, three characteristic equations plus shock boundary conditions can be used to calculate the five unknowns at the surge front. These five unknowns are \( v \) and \( y \) on both side of the interface and the speed of the interface movement.

The model can handle various boundary conditions representing junction, dropshaft, reservoir, and other accessories. The input data files include the inflow hydrograph, and initial values of \( v \) and \( y \), tunnel size, dropshaft size, etc.
III. THE MODELING RESULTS

A. DESCRIPTION OF THREE CASE STUDIES

Three cases representing three different stages of completion of the West Side System were modeled. The first case includes the segments of the Lake Avenue Tunnel, Dewey–Eastman and Tiger–Carlisle. The second case represents the existing condition, which includes the segments of the Saxton–Colvin Tunnel, the Lyell Avenue Tunnel, and the Lake Avenue Tunnel. The third case represents the completed West Side System, containing all the tunnels in the first two cases.

In November 1982, a comprehensive study of the hydraulic transient problem of the Genesee River Storage–Conveyance System, Rochester, New York, was reported by Song et al. In that study, the combined storm sewer for the West Side System was designed based on the inflow data of 1982. However, for the present study, an updated design inflow was provided which constitutes about a 40 percent increase for the segments of State–Mt. Hope tunnel and Mt.–Hope–Rosedale Tunnel and 70 percent increase for the segment of the Lexington–North Tunnel. This significant increase in inflow rates is shown to cause some transient problems that were not predicted in the previous study.

B. COLUMN CONSOLIDATION AND GEYSER PHENOMENA

1. Column Consolidation

During the filling process, the flow in the tunnel system will go through three regimes: open channel flow regime, mixed flow regime and closed flow regime. In some cases, due to the uneven inflow distribution, pressurization may initiate at more than one station. That means there are two or more pressurized zones separated from each other by an open channel flow zone to occur in the tunnel system. This may lead to consolidation as two interfaces move toward each other. Very high pressure may be generated during a column consolidation process due to either the compression of entrapped air or the collision of two interfaces. Therefore, column consolidation should not be allowed to occur. Ideally, the tunnel should start to pressurize at the downstream end and the interface should be allowed to move upstream in an orderly manner.
2. Geyser Phenomenon

When the water level in a dropshaft rises slowly to the connecting pipe, water may just back up the flow in the connecting pipe, causing overflow or flooding. But if water level rises too rapidly, it may overshoot the connecting pipe and impact the cover. In a severe case, the cover may be broken or lifted and a geyser is produced.

Song et al. [3] and Guo and Song [4] have studied this phenomena based on the independent dropshaft–drift tube system analysis. In order to analytically determine whether or not a geyser process will occur, a set of safety criteria was proposed. They are rewritten as follows:

(1) Free oscillation number:

$$N_f = \frac{2}{H_s - H_m} \sqrt{\frac{H_m}{g}} U_m$$  \hspace{1cm} (8)

(2) Surge tank oscillation number:

$$N_s = \frac{2}{H_s - H_m} \sqrt{\frac{L A_s}{g A_d}} U_m$$  \hspace{1cm} (9)

(3) Water hammer oscillation number

$$N_w = \frac{2}{H_s - H_m} \frac{L}{\pi a} U_m$$  \hspace{1cm} (10)

in which, $H_m$ = the main tunnel water level at the beginning of the time step; $H_s$ = the dropshaft height; $A_s$ = cross-sectional area of the dropshaft; $A_d$ = cross-sectional area of the draft tube; $a$ = pressure wave speed in the drift tube; $U_m$ = water rising speed at dropshaft. When any of the above three numbers is greater than one at a dropshaft, then there is a good possibility that a geyser will occur at the dropshaft.

The data for $A_s$ and $L$ for the three study cases are provided in Table 1. The stability criteria requires that

$$\text{Max} (N_f, N_s, N_w) < 1$$  \hspace{1cm} (11)

C. FIRST CASE

The configuration and stationing layout for this case is shown in Fig. 2. Based on the contractor's original design, a bulkhead is placed about 200 feet north of the construction access dropshaft No. 41 with a 1 foot air shaft opening. The purpose of the bulkhead is to allow construction shaft No. 41 to be used for the construction of the State–Mt. Hope Tunnel and the Mt. Hope–Rosedale Tunnel. Under this condition, the mathematical modeling results indicate that an extremely high pressure and geyser will occur at the air vent when the pressurization surge reaches the bulkhead. This situation is clearly not acceptable.
An alternative solution considered is to move the bulkhead to a location immediately south of dropshaft No. 13. In this way, the 6 ft diameter dropshaft can be used as a new upstream end. As expected, the modeling results indicate a substantial reduction in the severity of hydraulic transients. However, the improvement is not sufficient because two of the three stability numbers exceeded one (\( N_f = 1.14 \) and \( N_s = 2.15 \)). The model was then used to search for possible means of further improvement. A possible solution is, in addition to relocating the proposed bulkhead to the upstream side of dropshaft No. 13, to lower the outflow control weir at structure No. 45 to allow the outflow of 250 cfs.

The results of mathematical modeling for the condition stated above are shown in Fig. 4 to Fig. 14. For this run, the downstream end of the tunnel at structure No. 45 pressurizes first at \( t = 116 \) minutes (see Fig. 8). Then a surge interface moves upstream along each of the two connecting tunnels. As soon as a surge reaches the upstream end, the head rises rapidly at the upstream end (see Figs. 5 to 8 at \( t \geq 130 \) minutes). The time variation of water depth at Station 15 is shown in Fig. 6. The instantaneous hydraulic gradelines are shown in Fig. 4. Note that station numbers in these figures correspond to those shown in Fig. 3. In this case, the inflow is larger than the tunnel and dropshaft capacity at \( t = 135 \) to \( 140 \) minutes and overflow occurred at Station 17, 20, 26, 30, 40 and the downstream end, Station 42 (see Fig. 9 to Fig. 14).

For every dropshaft, we have

\[
\begin{align*}
\text{Max} (N_f) &= 0.70 \\
\text{Max} (N_s) &= 0.86 \\
\text{Max} (N_w) &= 0.15
\end{align*}
\]

Since \( \text{Max} (N_f, N_s, N_w) = 0.86 < 1 \), stability is insured. In summary, when dropshaft No. 13 is used as the upstream end instead of the 1 foot air shaft and the downstream end control weir is fully open, no severe transient problem may occur.

D. SECOND CASE

The configuration and stationing layout is shown in Fig. 15. This system consists of four junctions. Structure No. 45 is at the downstream end at Station 93 and structure No. 41 is an upstream end denoted as station 69. A numerical simulation is performed based on the 1989 updated design inflow. The tunnel is assumed to be initially empty. The instantaneous hydraulic gradelines are shown in Figs. 17 and 18. Note that the station numbers shown in this figure correspond to those indicated in Fig. 16. The hydraulic gradelines show the traveling front, back and forth in the tunnel, and finally damped out due to the friction. The time variation of water depth at a number of stations are shown in Fig. 19 to Fig. 25. Because the inflow is larger than the tunnel capacity, outflow occurs at a number of stations (see Figs. 26 to 29).
In this run, the downstream end was pressurized first. The surge interface develops at the interface between the pressurized zone and the free surface zone. Because the heights of dropshaft Nos. 6 and 7 are relatively small, i.e. 40 ft and 36.5 ft, respectively, overflow occurs there as early as at $t = 117.22$ minutes. Because the flow in the tunnel system is still in the mixed flow regime, when overflow is taking place at dropshaft Nos. 6 and 7 water level rises rather slowly elsewhere and a geyser process is not likely to occur.

The stability coefficients are:

$$\text{Max } (N_f) = 0.40$$
$$\text{Max } (N_s) = 0.71$$
$$\text{Max } (N_w) = 0.15$$

This shows that the stability coefficient is substantially less than one, indicating that surge is not strong enough to produce a geyser.

No inflow control or outflow control is necessary in this case.

E. THIRD CASE

The schematical diagram of the complete west side system is shown in Fig. 30. The downstream end is joined by two branch tunnel. This complete West Side System was simulated using the 1989 updated inflow hydrograph as inputs. A very unsatisfactory filling process, which is quite different from the result of 1982 simulation, has been obtained. The tunnel was assumed to be initially empty. Because of substantially increased design inflow from Station 1 to Station 37 and from Station 134 to Station 149, the system first pressurizes at Station 111 and Station 109 (see Fig. 31a), instead of at the preferred downstream end. If the bold dark line represents the segments in which pressurization occurs, then the time evolution of the surge front movement can be visualized from Fig. 31. This figure shows that at time $= 104.9$ minutes, two locations near Station 110 pressurizes simultaneously. Column consolidation takes place shortly after the formation of entrapped air bubble. As Fig. 31C indicates, another consolidation occurs at Stations 148 at 110.6 minutes.

After several trial runs, a workable solution to the column consolidation problem was found. The proposed solution requires the reduction of maximum inflow rate of 875 cfs to 175 cfs at dropshaft No. 10 and allows an outflow rate of 100 cfs to take place downstream from the beginning of the storm. This change will allow the pressurization to start at the downstream end and gradually spread in the upstream direction.

However, the surge at Station 1 (Structure No. 40A) will still be too severe and geysering is likely to take place there. The best way to solve the geysering problem appears to be to enlarge the diameter of the dropshaft at structure 40A to 25 feet from the original size of 12 feet.
The results of the final run, with the three recommended changes, are described below. Figures 34 and 35 show the instantaneous hydraulic gradelines which represent the traveling of the surge front in the main tunnel. Note that the station number in these two diagrams are from Fig. 36. The time variation of water depth for several locations are shown in Figs. 37 to 47.

For this case, the maximum values of stability coefficients are as follows:

\[
\text{Max}(N_f) = 0.87 \\
\text{Max}(N_s) = 0.75 \\
\text{Max}(N_w) = 0.40
\]

Since all three coefficients are less than one, no geyser problem is expected. The tunnel will eventually fill up and overflow will occur at some dropshaft. The overflow hydrographs are shown in Figs. 48 to 56.
IV. CONCLUSION

For 1989 updated design inflow, three final runs for three study cases were furnished. All results and alternative solutions are reported in Chapter III.

Following is a list of the main conclusions and recommendations for the three cases analyzed:

(1) For the first case, placement of bulkhead at 200 ft downstream of construction shaft No. 41 with a 1 ft diameter air shaft is not appropriate because the geyser phenomena will certainly occur at the location of the air shaft. The recommended solution it to move the bulkhead to a location immediately south (upstream) of dropshaft No. 13 so that dropshaft No. 13 which has a diameter of 6 ft, can be used as the upstream end. Also the downstream control weir is required to be full open to allow a 250 cfs outflow.

(2) For the second case, no major transient problem was encountered. No inflow or outflow control is necessary.

(3) For the third case, the maximum inflow at dropshaft No. 10 should be reduced from 875 cfs to 175 cfs and the control weir should be set to allow outflow of 100 cfs to occur from the beginning of the design storm. This should prevent the occurrence of column consolidation. In addition, the diameter of the dropshaft at structure No. 40A should be increased from 12 feet to 25 feet in order to prevent geysering to occur at this structure.
VI. REFERENCES


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Fig. 1. Location map showing West Side System.
Fig. 2. Configuration and stationing for first case.
Fig. 4. Instantaneous hydraulic gradelines, first case.
TIME VARIATION OF WATER DEPTH
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TIME VARIATION OF WATER DEPTH
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