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NUMERICAL SIMULATION OF HYDRAULIC TRANSIENTS IN DRAINAGE SYSTEMS

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Abstract: *Most Cities count on networks of underground tunnels for the conveyance of storm water and wastewater. A storm drainage system usually is designed to operate with free-surface flow regime, however when a storm exceeds the design event, the flow in the tunnels may transition from free-surface flow to pressurized flow. During the pressurization a moving air-water interface advances into the free-surface region with the potential for generating unacceptable hydraulic transients.*

A dynamic transient model has been developed to simulate the complex and highly dynamic flow during the pressurization of drainage systems. The model is based on the Interface Tracking Method and the Characteristics Method. The numerical results are compared against measurements.

The code was used to predict the potential for hydraulic transients in the West Area CSO Tunnel System of the City of Atlanta. Different design alternatives were evaluated to mitigate pressure oscillations, backflows and flooding during storm events.

1 INTRODUCTION

A storm drainage system is usually designed to operate with free-surface flow throughout during two or five year precipitation events. If the precipitation exceeds the design event, parts of all of the system may become pressurized. The pressurization usually starts at the downstream end of the system due to the tunnel slope. As the tunnels fill up, a pressurization wave travels toward the upstream end of the system. The regime with both free surface and pressurized flows is called *mixed-flow* and it is highly dynamic. If inflow rate is significantly larger than outflow rate, the speed with which the pressurization wave moves upstream can be very significant. At the end of the filling process when the upstream end of the tunnels get completely filled, the velocity of the pressurization wave is so large that it may cause pressure transients and backflow in the tunnels, overflow in shafts, flooding, damage to the system in the form of blow-off of dropshaft or manhole covers, and possibly even "geysering".

Many experimental and numerical studies have been conducted on mixed-flow in conduits[1,2,3]. Two of the most common numerical approaches are: (1) the Priessman Slot Method and (2) the Interface Tracking Method (ITM). The Priessmann Slot Method has been extensively used[4,5,6]. This method uses the Saint-Venant equations throughout the flow domain. It simulates the pressurized flow portion assuming a hypothetical narrow slot at the crown of the pipe. The major drawback with this method is that it assumes ventilated flow everywhere in the system and therefore cannot simulate sub-atmospheric flow conditions. In addition, it is numerically unstable when the slot width must necessarily be small to represent a large pressure-wave speed.

The ITM considers the free-surface and pressurized flows as two separate flow-regime domains and integrates them across the interface as it moves. This approach can simulate negative pressures and allow a free-surface interface to develop only in ventilated points. To simulate the flow in sewer systems, the moving interface can be modeled as a shock wave[7,8,9]. This "*shock-fitting model*" was also used to simulate mixed flow with air pockets[10] and implemented with different approaches to simulate the portions of tunnels with free surface or pressurized flows obtaining good agreement with measurements[3]. The shock-fitting model is especially useful and works well when the energy contained in the inflow is sufficient to create a regular hydraulic jump or bore that pressurizes the duct. The principal drawback of this model is that it needs to maintain a hydraulic jump even if the pressurization takes place with gradually varied flow.

This study was conducted to support the design of the West Area Combined Sewer Overflow (CSO) Storage Tunnel Facilities of the City of Atlanta. The study included hydraulic design of drop structures, analysis of hydraulic transients, and recommendations for design changes to reduce the adverse effects of transients. The following is a brief description of the numerical model used to simulate the flow during the filling of the tunnels. The model is based on ITM and it includes a novel treatment of the air-water interface when the energy contained in the inflow is insufficient to create a regular hydraulic jump. Examples of such a flow condition include pressurization in subcritical flow after hydraulic jumps, and pressurization with moderate inflow rates in long, sloped, partially full conduits. The description includes a comparison of numerical results with experimental data. The magnitude of the pressures transient during tunnel filling in the original and improved designs are presented and discussed.

2 MATHEMATICAL AND NUMERICAL MODEL

For the purpose of the hydraulic model the drainage system is considered a network of one-dimensional ducts connected by zero-dimensional components such as shafts, junctions, pump, expansions/contractions, etc (see Fig. 1). The mixed-flow in the ducts is modeled as two separate flow regimes (free-surface or pressurized flows) using ITM. The components are the boundary conditions of the system.

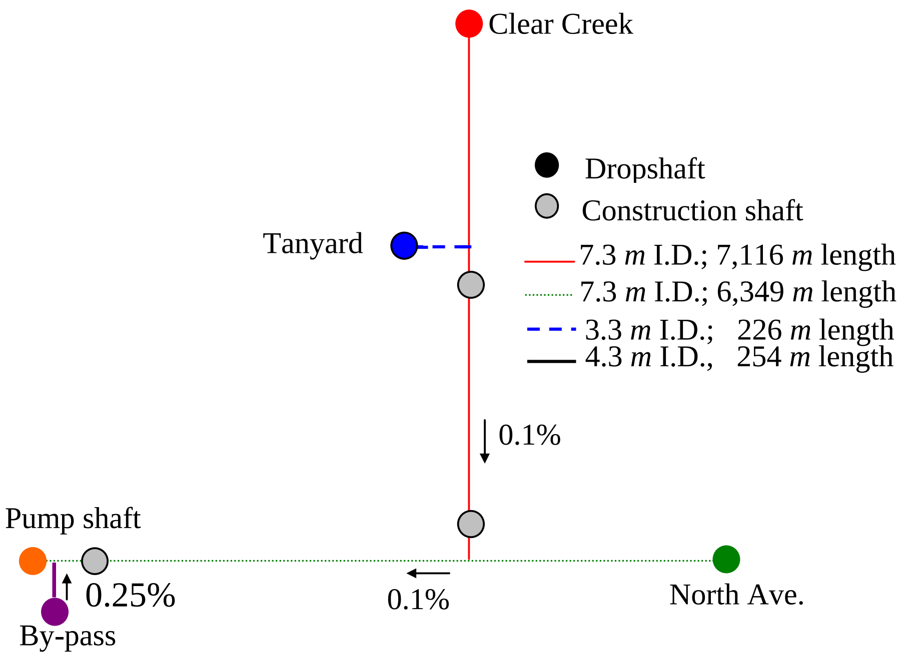


Figure 1: West Area CSO tunnels system configuration for modeling purposes

2.1 Modeling of regimes with free-surface or pressurized flow

The continuity and momentum equations for pressurized flows read[11]:

$$\frac{\partial H}{\partial t}+\frac{a^2}{g}\frac{\partial V}{\partial x}+V\,S_o+V\frac{\partial H}{\partial x}=0 \tag{1}$$

$$g\frac{\partial H}{\partial x}+V\frac{\partial V}{\partial x}+\frac{\partial V}{\partial t}+S_f=0 \tag{2}$$

Where H is the piezometric head measured from the tunnel invert, a the pressure-wave speed, *V* the velocity, *g* the acceleration due to gravity, *x* the distance along the tunnel, *t* the time and S_f the friction slope. The St. Venant equations for free-surface flows are[11]:

$$\frac{\partial y}{\partial t}+\frac{c^2}{g}\frac{\partial V}{\partial x}+V\frac{\partial y}{\partial x}=0 \tag{3}$$

$$g\frac{\partial y}{\partial x}+V\frac{\partial V}{\partial x}+\frac{\partial V}{\partial t}+g\left(S_f-S_o\right)=0 \tag{4}$$

Where *y* is the water depth measured from the tunnel invert and S_o is the slope of the tunnel. The friction slope is given by $S_f=\dfrac{f\,V\,|V|}{2\,D}$ for pressurized flows and $S_f=\dfrac{n^2\,V\,|V|}{R_h^{4/3}}$ for free-surface flows, with f the Darcy-Weisbach friction factor, *n* the Manning coefficient, *D* the tunnel diameter and R_h the hydraulic radius. The gravity wave-speed is defined as $c=\sqrt{g\,A/T}$, with T the top width of flow. The cross sectional area normal to the flow *A* for circular ducts is:

$$A=\begin{cases}\dfrac{D^2}{8}\left(\alpha-sin(\alpha)\right) & ;\ \cos\left(\dfrac{\alpha}{2}\right)=1-\dfrac{2y}{D} \qquad if\ y\le\dfrac{D}{2}\\[2ex] \dfrac{D^2}{4}\left[\pi-\dfrac{\left(\alpha-sin(\alpha)\right)}{2}\right]; & \cos\left(\dfrac{\alpha}{2}\right)=\dfrac{2y}{D}-1 \qquad if\ y\ge\dfrac{D}{2}\end{cases} \tag{5}$$

The two partial differential equations ((1) and (2) or (3) and (4)) are converted into four total differential equations using the characteristic method[11]. The resulting equations for the positive characteristic line C^+ and negative characteristic line C^- are:

Pressurized flow:

$$C_p^+\begin{cases}\dfrac{dx}{dt}=V+a\\[1ex]\dfrac{dH}{dt}+\dfrac{a}{g}\dfrac{dV}{dt}+V\,S_o+a\,S_f=0\end{cases} \qquad C_p^-\begin{cases}\dfrac{dx}{dt}=V-a\\[1ex]\dfrac{dH}{dt}-\dfrac{a}{g}\dfrac{dV}{dt}+V\,S_o-a\,S_f=0\end{cases} \tag{6}$$

Free-surface flow:

$$C_{fs}^+\begin{cases}\dfrac{dx}{dt}=V+c\\[1ex]\dfrac{dy}{dt}+\dfrac{c}{g}\dfrac{dV}{dt}+c\left(S_f-S_o\right)=0\end{cases} \qquad C_{fs}^-\begin{cases}\dfrac{dx}{dt}=V-c\\[1ex]\dfrac{dy}{dt}-\dfrac{c}{g}\dfrac{dV}{dt}-c\left(S_f-S_o\right)=0\end{cases} \tag{7}$$

2.2 Interface tracking method

The flow conditions near the air-water interface are calculated using either the shock-fitting model or a mass and momentum balance in a control volume. The shock-fitting model is appropriate when the energy contained in the inflow is sufficient to pressurize the flow through a hydraulic jump. The water depths and velocities near the interface are obtained using two shock-boundary conditions plus three characteristic equations[7,8,9].

If the velocity changes are more gradual, the acceleration of the flow between two adjacent sections can be neglected and the flow near the interface can be simulated using momentum and mass balance on a moving control volume. Fig. 2 shows a generic control volume where the pressurization takes place between nodes *I-1* and *I*. Node *I* is in pressurized flow and Node *I-1* in free-surface flow. The dotted line indicates the water surface. The interface moves towards Node *I-1* by either an increase in pressure head or decrease in velocity in Node *I* or by an increase in velocity in Node *I-1*. As the pressurization wave moves towards *I-1*, the water elevation at this node rises. When the water level at *I-1* reaches the crown of the pipe, the interface is shifted upstream and now located between Nodes *I-2* and *I-1*. The mass and momentum equations in a control volume between nodes *I-1* and *I* are:

$$\frac{d\left(A_{I-1} - A_d\right)}{dt}\Delta x = A_{I-1} V_{I-1} - A_d V_I \tag{8}$$

$$0 = A_{I-1}\, V_{I-1}^2 - A_I V_I^2 + g\left(\overline{y_{I-1}}\, A_{I-1} - \left(H_I - z_I - \frac{D}{2}\right) A_d + \Delta x \left(\frac{A_{I-1} + A_d}{2}\right) S_o\right) \tag{9}$$

where $\overline{y} = y - D/2 + C_g$, with C_g distance from the center of the tunnel to the centroid of the flow cross sectional area. Equations. (8) and (9) together with the positive characteristic equation for free surface flow, C_{fs}^{+}, and the negative characteristic equation for pressurized flow, C_p^{-}, determine water depth and velocity at the stations adjacent to the interface. This method allows for accurate tracking of the interface conserving mass.

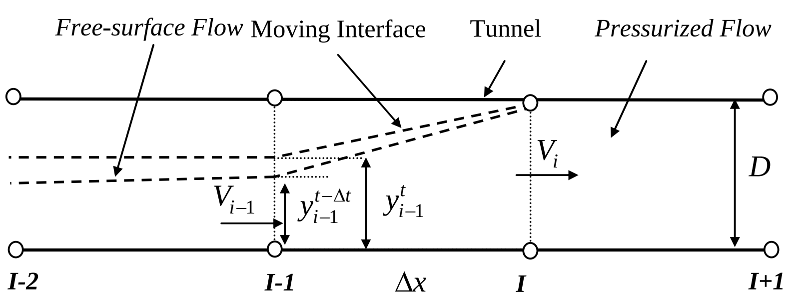


Figure 2: Control volume for the moving interface

2.3 Boundary conditions

A component with N_c connections have $2N_c$ unknown variables, which can be determined using one mass conservation equation, $N_c - 1$ momentum equations and N_c characteristic equations. All the shafts overflow if the water elevation reaches the ground level. It is common to assume that overflowed water is temporarily contained at ground level at the top of the shafts[5], and that the overflowed water re-enters the system when able. However, in the CSO system simulated herein the overflow is diverted and cannot reenter the system; therefore the water level in the shafts drops as soon as the pressure in the tunnel drops.

The model equations are discretized using a fixed-grid method with a first-order finite difference approximation. The resulting nonlinear equations are solved using the Newton-Raphson Method. The time step is selected as the smaller of two values, either that determined by the Courant's criteria or the time it takes to produce pressurized flow in the subsequent node of the *ITM* model.

3 COMPARISON AGAINST EXPERIMENTAL DATA

The model was validated with measurements of mixed flows in circular pipes[6,8]. Cardle and Song's measurements were made in a 48.8 *m* long 0.1626 *m* I.D. circular pipe with slope $S_o = 0.001$[8]. An advancing interface was generated by a sudden gate closure at the downstream end of the pipe. The variation in piezometric head was measured in three points along the pipe, P_1, P_2 and P_3 located at 9.1 *m*, 21.3 *m* and 39.6 *m*, respectively, from the downstream end of the pipe. The water level in the downstream reservoir was at elevation 0.15 *m* and the inflow rate was 0.0068 m^3/s. As seen in Fig. 3, there is a good agreement between measurements and predicted piezometric head in the three points. However, the piezometric head in the measured jump as the interface advances past the three points is not as abrupt as that predicted. This might be because the gate closure in the experiment occurred over a finite time while the model assumes a sudden shutoff of all the flow.

Figure 4 shows a comparison between measurements and computed piezometric heads in a circular pipe 10 *m* long 0.10 *m* I.D. on a slope $S_o = 0.027$[6]. The points of measurements are located at 0.6 *m* (P_1) and 4.5 *m* (P_2) from the downstream end. At the upstream end, a tank with overflow kept constant the water level. The pressurization wave is generated from steady supercritical free surface flow condition closing suddenly a sluice gate at the downstream end. However, in this case, the hydraulic jump itself does not pressurize the pipe. The pressurization takes place from a subcritical free surface flow condition. As seen in Fig. 4, the main features of the mixed transient flow are predicted by the model and a good agreement between experimental and numerical results is found. As in the previous simulation, the model overpredicts the height of the hydraulic jump because the assumption of sudden shutoff of all the flow at the downstream gate.

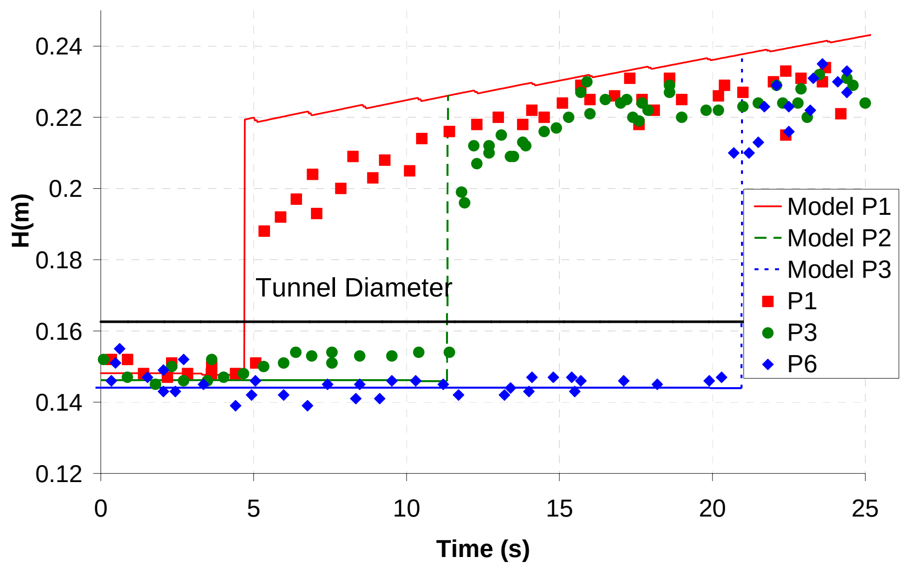


Figure 3: Comparison of measurements (Cardle & Song[8]) and predicted piezometric head in pipe flow

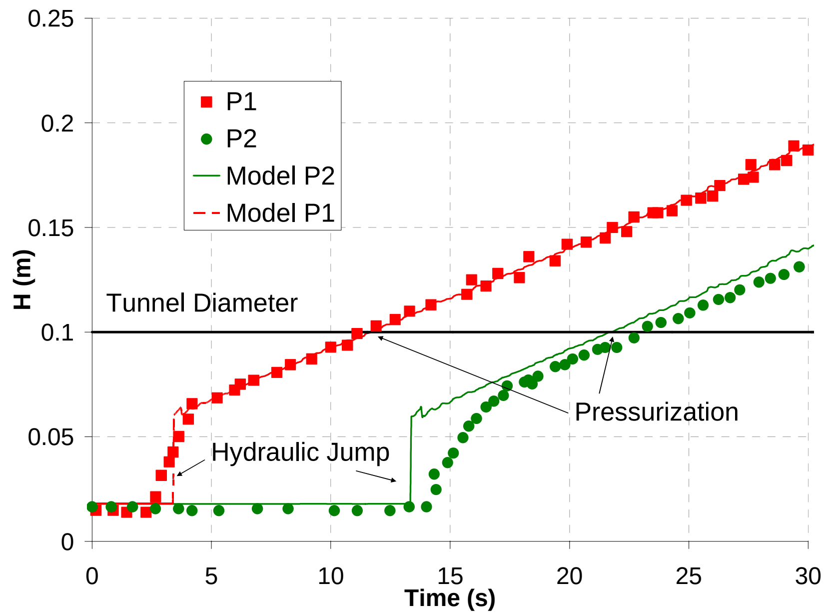


Figure 4: Comparison of measurements (Trajkovick[6]) and predicted piezometric head in pipe flow

4 APPLICATION: WEST AREA CSO TUNNEL SYSTEM

The model was used to predict the hydraulic transients in the West Area CSO Tunnel System for the City of Atlanta. As shown in Fig. 1, the system consists of two 7.3-*m* diameter tunnels: One running from North Avenue to a Pump Station, the other from Clear Creek to a junction with the North Avenue tunnel. The lengths of the tunnels are 7,116 *m* and 6,349 *m*, respectively. To facilitate the conveyance to the pump station the tunnels are laid on a slope of 0.001. The flow to the tunnels is received through two 7.3-*m* diameter dropshafts: one at North Avenue, the other at Clear Creek, and one 3.8-*m* diameter dropshaft at Tanyard. The Tanyard dropshaft is located at the upstream end of a short 3.3-*m* diameter sub-branch to the Clear Creek Branch. Three 12.2-*m* diameter construction shafts are located along the tunnels. The total storage capacity of the system is about $6\ 10^6\ m^3$.

The simulations were performed using the 25-year hydrograph shown in Fig. 5. The initial condition was mixed-flow with the tunnels 60% full. Water was admitted to the system through the dropshafts and it was pumped out at the pump station at a rate of $3.72\ m^3/s$. The distance between two grid points was $\Delta x = 12.2\, m$. The Manning's roughness coefficient was $n = 0.015$ and the speed of the pressure wave in water was $a = 762\ m/s$.

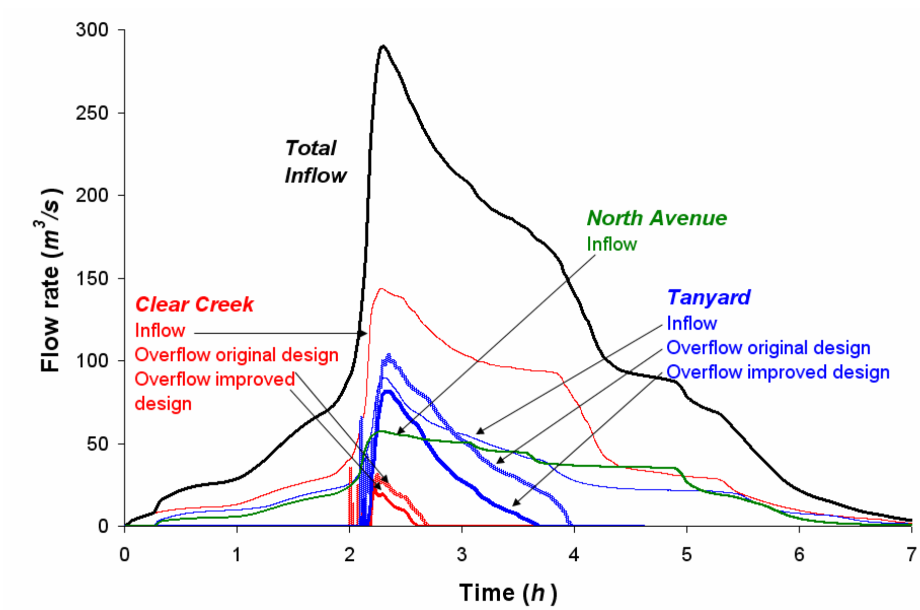


Figure 5: Rates of inflows and overflows for 25-year hydrograph

The velocity and pressure were calculated as a function of time at all grid points and plotted at key points. Simulations of the original tunnel system design showed severe pressure oscillations, backflows, and overflows. An alternative network configuration was proposed to mitigate these problems. Simulations showed that the most effective and feasible improvements were (1) to let the 12.3 *m*. diameter Clear Creek and North Avenue construction shafts be part of the flow system functioning as surge tanks, and (2) to design of a 7.3-*m* diameter by-pass tunnel just upstream from the pump station. The improvements were so significant that considerations were subsequently made to increase the elevation of the overall tunnel system by 12.2 *m*. The final simulations were made with the tunnels at this higher elevation.

4.1 Hydraulic transients in the original and improved tunnel system design

During the early stages of the storm, the flow in the tunnels is mixed-flow; the interface between free-surface and pressurized flow is located in the North Avenue tunnel between the downstream construction shaft and the Clear Creek branch. Upstream of the interface, the water surface is below the crown of the tunnels. As the tunnels fill, the interface propagates upstream. The velocity with which the interface travels upstream increases with time as the inflow rate increases (see Fig. 5) and the volume available for storage becomes increasingly smaller.

Figure 5 shows the rates of overflow in the dropshafts of the original and improved designs. At about 2 *h* Clear Creek and Tanyard start to overflow. No overflow occurs in the North Avenue dropshaft because it is further from the upstream end and the ground level is higher. Over a period of about 40 *min*, the overflow rate at Tanyard is greater than the inflow rate meaning that water is flowing backwards in the branch toward the dropshaft. The overflow in Tanyard is bigger and of longer duration due in part to the smaller tunnel diameter of the Tanyard branch, which results in reduced conveyance. As shown in Fig. 5, the overflows in the improved design in both Tanyard and Clear Creek dropshafts are smaller.

Figure 6 shows cumulative water volumes as a function of time. The horizontal line designated “Tunnel System” indicates the available storage volume of the tunnel system considers the initial condition of 60% full. At any given time, the total volume shown represents the total inflow volume less the volume that has been pumped out up. In this simulation, the 25-year hydrograph fills the tunnels at about 2 hours. It is also seen that, given the initial condition and pumping rate, only about 12% of the net inflow is stored in the tunnel system. When the system reaches capacity in the original system, most of the water is released through the pump shaft causing overflows at the pump station. In the improved design, the by-pass handles most of the excess inflow and prevents overflow and flooding of the pump station. Although early overflow at Tanyard and Clear Creek cannot be prevented with the new design, both the magnitude of the cumulative volume during the overflow and the duration of overflow are significantly reduced.

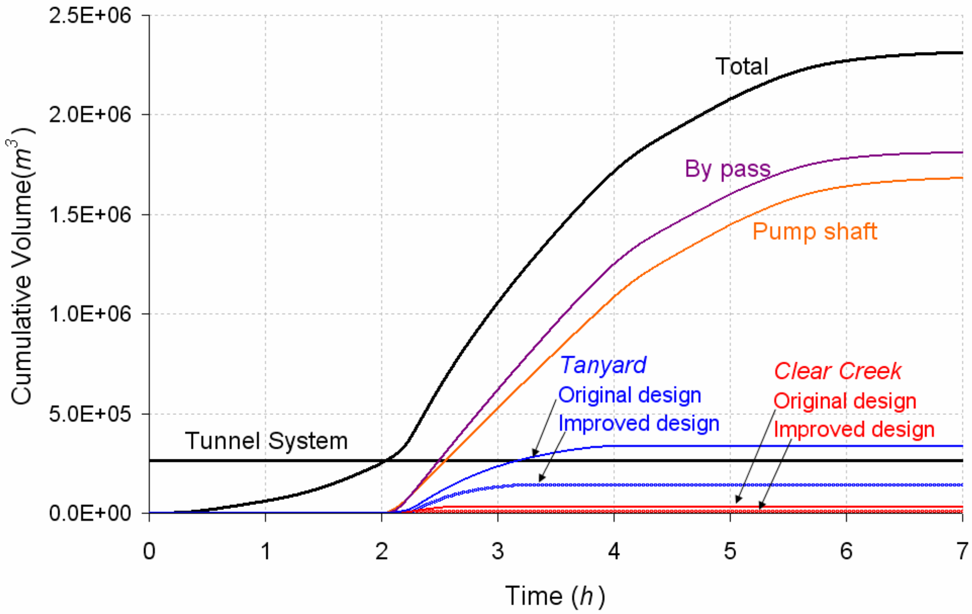
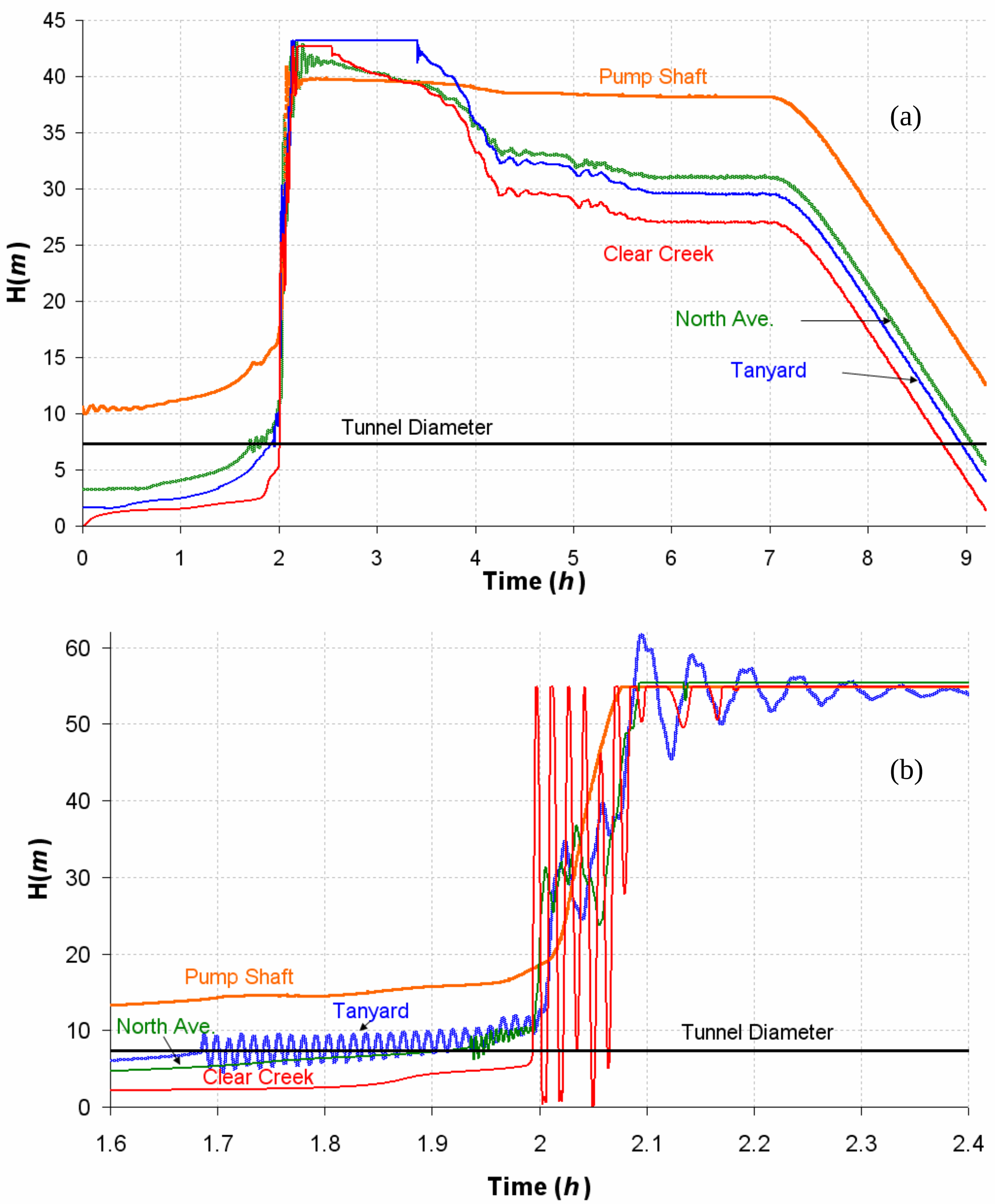


Figure 6: Cumulative volume in original and improved tunnel system

Figure 7 shows the water surface elevation/piezometric head measured from the tunnel invert for the original design (b) and for the improved design (a,c). The interface reaches the North Avenue shaft after approximately 1.7 *h*, the Tanyard shaft at 1.9 *h*, and the Clear Creek shaft after about 2 *h*. Note that when the interface or surge front hits Clear Creek and the system reaches capacity, negative velocities occurs in the tunnel at Clear Creek. This backflow together with the inflow of water to the system through the dropshaft causes the water level in the Clear Creek shaft to rise rapidly. The pressure is immediately transmitted as a pressure wave throughout the rest of the system causing significant pressure oscillations in all of the dropshafts. It is seen that the hydraulic transients generated when Tanyard and North Ave. pressurize are negligible compared with those developed when the pressurization wave generated in Clear Creek is transmitted. Because of its upstream location, the Clear Creek dropshaft reacts with particular large pressure fluctuations. As seen in Fig. 7b, in the original design, the pressure head drops intermittently below the crown of the tunnel allowing a free-surface to develop before the next surge hits the shaft. A total of five pressurization-depressurization waves occur at this location over a period of four minutes. The pressure oscillations are coincident with intermittent backflows. When the pressurization wave hits Tanyard, large velocity fluctuations are generated in the Tanyard tunnel (about ± 4 m/s) because the smaller tunnel and dropshaft size. However, at this moment the system is under pressure and no free-surface flow develops in the tunnel.

The above simulations show that as the system reaches capacity there is a potential of air entrapment and bubble collapse which are important structural design concerns. The simulations also show that by allowing the construction shafts to act as surge tanks, the risk of such problems are reduced. As seen in Fig. 7c, when the improved tunnel system reaches capacity, pressure heads remain above of the crown of the tunnel preventing free-surface flow from developing until the system starts to depressurize.



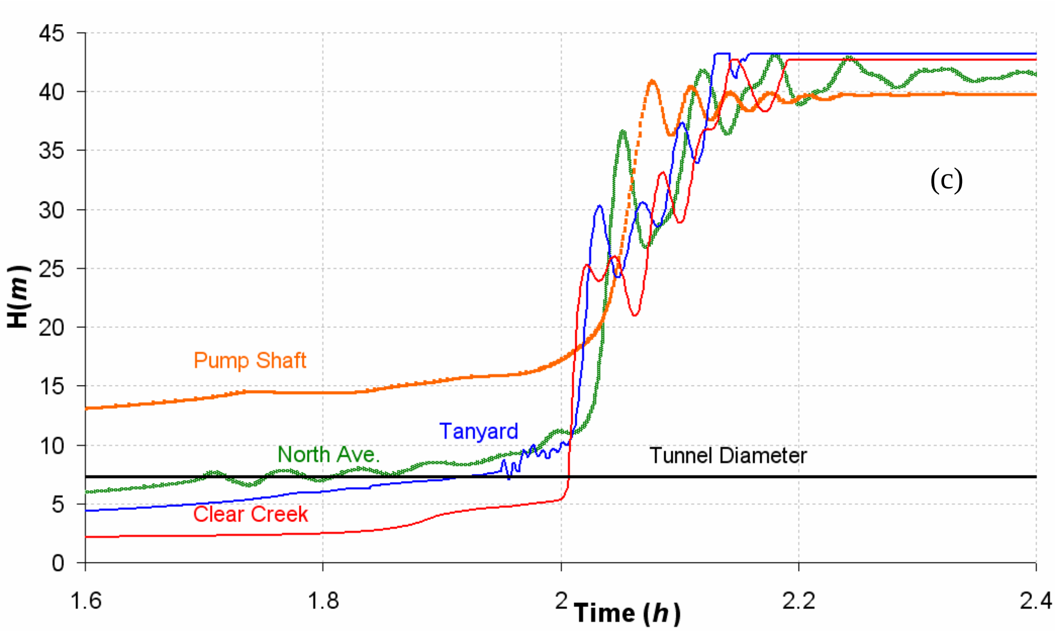
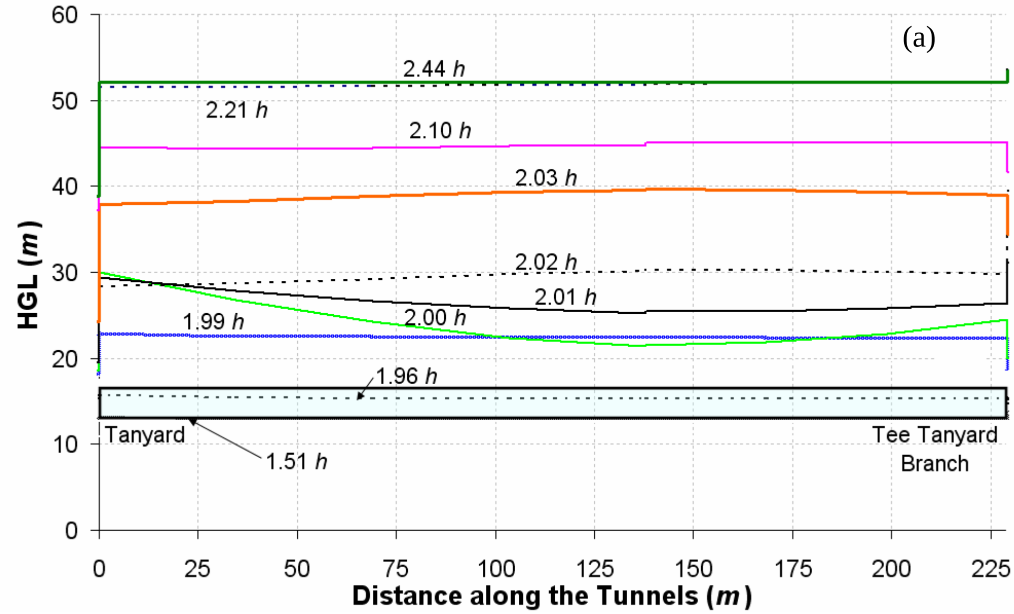


Figure 7: Piezometric heads in original (b) and improved tunnel system (a-c)

The simulations show that the by-pass has little effect on the transients. This is because no significant bypass flow develops before the surge reaches the upstream end of the tunnel system and the entire system is pressurized. However, the by-pass is effective in controlling the ultimate water level in the pump station.

As seen in Fig. 7a, toward the end of the storm when runoff and inflow rate diminishes, the overflow and water level in the shafts decrease. As the water is pumped out, the flow regime in the tunnel becomes mixed again at about 8.8 *h*. The subsequent pressure variation is smooth and responding to the changes in inflow hydrograph.

Fig. 8 shows the hydraulic grade lines (HGL) along the tunnels at different times for the improved design. The Tanyard branch is initially empty and starts to fill from the upstream as the water is admitted through the Tanyard dropshaft. The oscillations generated when the system is completely full are very small and affect only the upstream location of this tunnel during a short time. As seen in Fig. 8a, the upstream portion of the system is subjected to the most important oscillations. Note the pressurization wave traveling upstream at 1.98 *h*. At this time, the pressure oscillations observed downstream of the surge wave do not imply free-surface flow since the water level is always above the tunnel invert. Note also that the most significant oscillations occur at the portion of the tunnel upstream of the Tanyard branch. The maximum pressure at the tunnels occurs before the stabilization of the system, at 2.21 *h*, near Clear Creek dropshaft, and it is approximately 55 *m*.



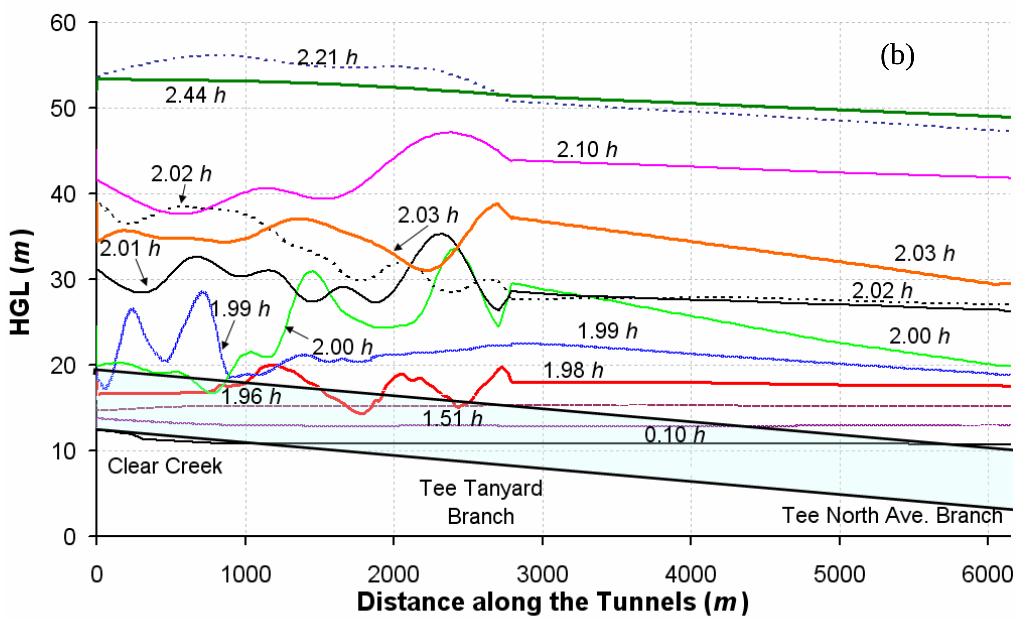


Figure 8: HGL in the CSO tunnels for 25 years hydrograph

5 CONCLUSIONS

A dynamic, transient numerical model has been developed to simulate the flow during pressurization of a drainage system. The model is based on Interface Tracking Method and solves the air-water interface when the energy contained in the inflow creates a regular hydraulic jump or when the pressurization takes place with gradually varied flow. The characteristic method is used to solve the regions of the system with pressurized or free-surface flows. The numerical results compared well with experimental data.

The code was used to predict the hydraulic transients for the West Area CSO tunnel system of Atlanta city. Different design alternatives were evaluated, the numerical results showed that by allowing upstream construction shafts act as surge tanks the hydraulic transients can be significantly reduced. Also, the inclusion of a by-pass near the downstream end of the system effectively prevents overflows and flooding in the pump station.

6 ACKOWLEDGMENT

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