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16. Abstract

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SI CONVERSION FACTORS

To Convert From	To	Multiply By
Length:		
in	cm	2.54
in	m	0.025 4
ft	m	0.304 8
yd	m	0.914 4
mi	km	1 . 609 344
Area:		
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ft^2	m^2	9.290 304 E-02
yd^2	m^2	8.361 274 E-01
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acre (a)	Hectares	4.046 856 E-01
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pt	m^3	4.731 765 E-04
qt	m^3	9.463 529 E-04
gal	m^3	3.785 412 E-03
in^3	m^3	1.638 706 E-05
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yd^3	m^3	7.645 549 E-01

NOTE: $1m^3 = 1,000$ L

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dwt	kg	1.555 174 E-03
lb	kg	4.535 924 E-01
ton (2000 lb)	kg	9.071 847 E+02
Mass per Unit Volume:		
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lb/ft^3	kg/m^3	1.601 846 E+01
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Velocity: (Includes Speed)		
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knot	m/s	5.144 444 E-01
mi/h	km/h	1.609 344 E+00
Force Per Unit Area:		
lbf/in^2 or psi	Pa	6.894 757 E+03
lbf/ft^2	Pa	4.788 026 E+01
Viscosity:		
cS	m^2/s	1.000 000 E-06
P t	Pa's	1.000 000 E-01

Temperature: $(°F-32)^5/9 = °C$

INTERIM REPORT

A MICROCOMPUTER MODEL FOR SIMULATING PRESSURIZED FLOW IN A STORM SEWER SYSTEM

by

Shaw L. Yu
Faculty Research Scientist

and

Yin Wu
Graduate Research Assistant

Virginia Transportation Research Council
(A Cooperative Organization Sponsored Jointly by the Virginia Department of Transportation and the University of Virginia)

In Cooperation with the U.S. Department of Transportation
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INTERIM REPORT

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INTRODUCTION

Highway drainage has long been a major area of concern for transportation engineers. This is not only because of its obvious social and economic impact but also because of the complexity of the various physical processes involved. When a drainage pipe is not flowing full, a condition known as "gravity flow" or "open-channel flow" exists. On the other hand, when the sewer pipe is flowing full and under pressure, a condition known as "surcharging flow" or "pressurized flow" exists. Physical principles governing open-channel flow no longer apply when the flow becomes pressurized. Presently, several advanced computer models are available that simulate sewer flow using various forms of fully dynamic equations under unsteady open-channel and pressurized conditions. Typically, however, these routing models are extremely complex and require considerable computer time even on mainframe computers. Several of these models are discussed later in Section II. The principle objective of this study is to carry out a preliminary investigation for developing a model or modifying an existing computer model to be attached to the FHWA Pooled Fund Storm Sewer Program (PFP-HYDRA) for the analysis of storm sewer flow under pressurized flow. Such a model would accurately predict hydraulic gradeline and flow conditions under both open-channel and pressurized flow and would run on a microcomputer as part of the Pooled Fund Drainage Design Package.

REVIEW OF EXISTING STORM SEWER MODELS

Through the years a large number of sewer models have been developed, ranging from the popular simplistic rational method (ASCE, 1969) to the complex storm sewer network models such as the Storm Water Management Model (SWMM) (Roesner et al., 1981). According to the level of complexity in hydraulics, these models can be classified as follows: dynamic wave models, noninertia models, nonlinear kinematic wave models, and linear kinematic wave models (Yen, 1986). Using the above classification, a review of the major existing sewer models was made based on the available information in the literature. The SWMM is perhaps the best known among all the sewer models. The Extended Transport Block (EXTRAN) was added on SWMM Version III to provide a model with dynamic wave simulation capability. The program simulates branched or looped networks, backwater resulting from tidal or nontidal conditions, free-surface flow, pressurized flow or surcharge, flow reversals, flow transfer by weirs, orifices, pumping facilities, and storage at on- or off-line facilities. Types of channels that can be simulated include circular, rectangular, horseshoe, egg, basket handle pipes, and trapezoidal channels. Simulation output takes the form of water surface elevations and discharges at selected system locations. For surcharge flow, an assumption is made that excess surface water is lost and not recoverable. EXTRAN, being an explicit difference formulation, solves the flow sewer by sewer. Therefore, it is relatively easy to program. Nonetheless, because of the assumptions regarding the excess water under surcharge and also the stability and convergence problems of the explicit solution scheme for the open-channel condition, EXTRAN is theoretically inferior to other dynamic wave models (Yen, 1986). The most versatile storm water model, the Danish Stormwater Model (Price, 1984), was developed at the computer center KOMMUNEDATA in Denmark. The main features of the Danish model, called "SVK-SYSTEM," are shown in Figure 1. It can be seen that for pipe flow routing, three options are available to the user for simulating a sewer system: (1) time-area approach, (2) kinematic wave approach, and (3) fully dynamic wave approach. Therefore, the user is offered a very flexible model and can choose the level of sophistication desired for the numerical solution. Although a powerful model, the SVK-SYSTEM is proprietary and is extremely expensive.

Wood (1983) developed a dynamic, lumped parameter model that provides a simple and reliable method for the analysis of a storm sewer system under surcharge. In this model, water is assumed to act as a rigid column in which the inertial effects are lumped over the pipe length. When surface flooding occurs, the excess water is assumed to be stored temporarily in a surface detention area connected to the manhole and will return to the sewer system at a later time without any volume loss. Another model, a linear kinematic wave model called "ILLUDAS" (Terstriep and Stall, 1974), utilizes storage routing methods in computing sewer flows. Under surcharge the sewer is assumed to have steady uniform and full pipe discharges. Like the Wood's model, excess water is stored upstream to be released later when sewer capacity is available. One of the most detailed treatments of surcharge flow in storm sewer systems is given by Yen (1981) in developing a kinematic wave surcharge model named "SURKNET." The hydraulics of surcharge sewer flow along with open channel flow are developed using the dynamic wave equations together with Manning's formula for calculating the frictional slope. Manhole storage and surface flooding are accounted for through use of the unsteady junction continuity equation. The SURKNET model solves for flows in each of the pipes independently in a cascading manner from upstream toward downstream. Several storm sewer flow models treat surcharge flows by using the so-called Preissmann Slot technique. These included the French Model "CAREDAS" (Chevereau, 1978) and the Danish Hydraulic Institute model, "System 11 Sewer" (Hoff-Clausen et al., 1982). In these models, pressurized flow is transformed into open-channel flow artificially by the introduction of a friction slot at the sewer crest that runs the entire sewer length (Figure 2). Consequently, both open channel and surcharge flows are handled using the full Saint-Venant Equations.

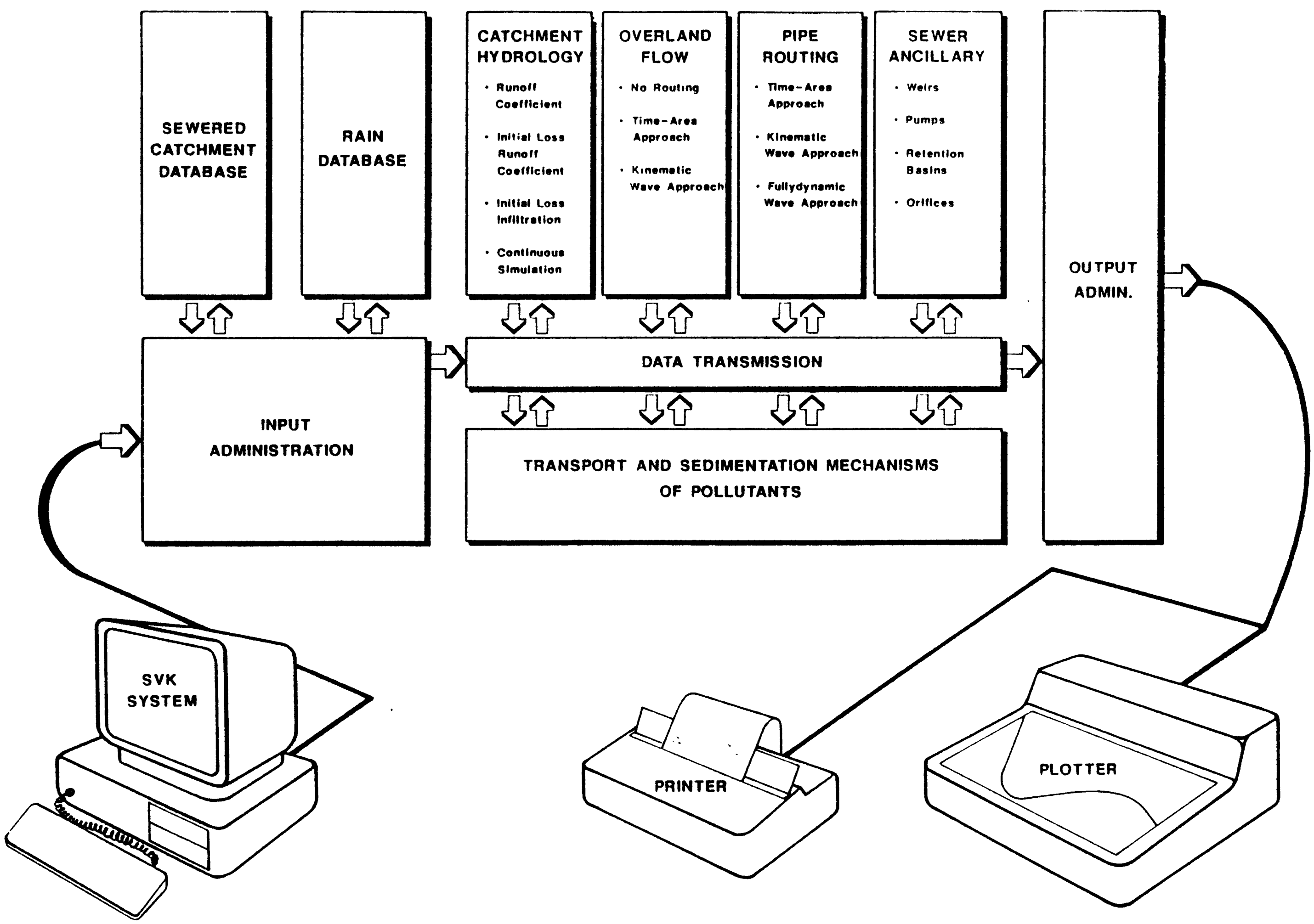

Figure 1. The prospective goal for the development of the SVK-SYSTEM.

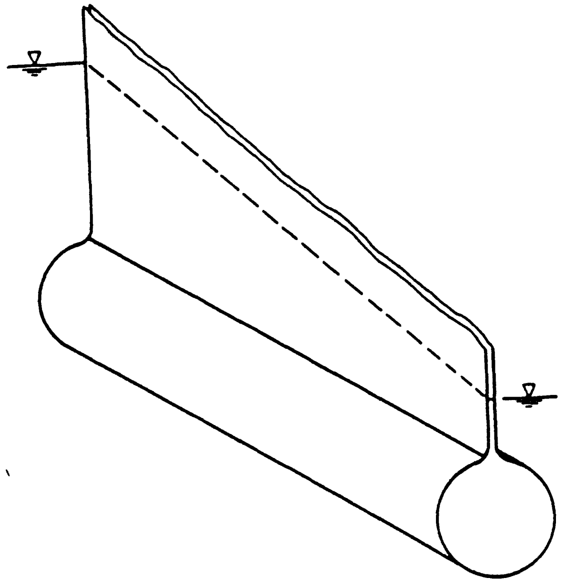

Figure 2. Hypothetical Preissmann piezometric open slot.

THEORY OF ONE-DIMENSIONAL UNSTEADY FLOW

Governing Equations

The flow in a sewer follows the physical principles of conservation of mass, momentum, and energy. The mass conservation principle yields the continuity equation, whereas Newton's second law yields the momentum equation. The two equations can be expressed in terms of either discharge Q or flow cross-sectional average velocity V (=Q/A) (Figure 3).

Continuity: $$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad (1)$$

Momentum: (2)

$$\frac{1}{gA}\frac{\partial Q}{\partial t} + \frac{1}{gA}\frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + \cos\theta\frac{\partial h}{\partial x} - (S_o - S_f) = 0$$

kinematic wave

noninertia

quasi-steady dynamic wave

dynamic wave

$$\frac{1}{g}\frac{\partial V}{\partial t} + \frac{V}{g}\frac{\partial V}{\partial x} + \cos\theta\frac{\partial h}{\partial x} - (S_o - S_f) = 0$$

in which,

Q = discharge	S = sewer slope or channel slope
t = time	Sf = friction slope
A = area	g = gravitational acceleration
h = depth of flow	q = lateral flow rate

These two equations are referred to as Saint-Venant equations for unsteady flow in open channels or sewers.

In the surcharge phase, the flow cross-sectional area is constant, being equal to the full pipe area, Af; hence $\partial A/\partial x=0$. The continuity and momentum equations can be rewritten as.

Continuity: $$\frac{1}{V}\frac{\partial H}{\partial t}+\frac{\partial H}{\partial x}+\frac{c^2}{gV}\frac{\partial V}{\partial x}+\sin\theta=0 \quad (3)$$

Momentum: $$\frac{1}{g}\frac{\partial V}{\partial t}+\frac{V}{g}\frac{\partial V}{\partial x}+\frac{\partial H}{\partial x}+S_f=0 \quad (4)$$

These are quasi-linear hyperbolic partial differential equations containing two dependent variables (P,V) and two independent variables (x,t). Pressure and velocity are a function of both location and the time from which the steady state conditions are disturbed.

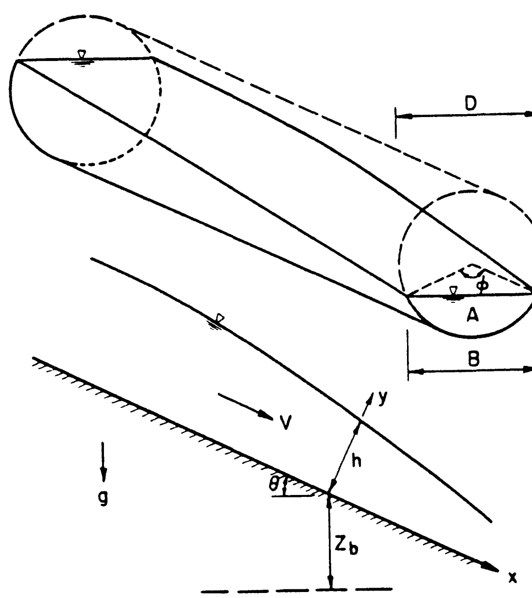

Figure 3. Open-channel flow in a sewer.

Approximations to the Saint-Venant Equations

The dynamic wave equations [(1) and (2)] are often referred to as "complete" because they contain all of the terms describing the dynamic effects of an unsteady open-channel flow. To solve these equations for specific initial and boundary conditions is rather tedious and computationally costly. Therefore, both efficient solution methodologies and acceptable simplifications of the equation have been proposed.

Different levels of approximation of the dynamic equation can be obtained by dropping certain terms in the equation. Referring to Equations (1) and (2), if the local acceleration term $\partial Q/\partial t$ is dropped, the approximation is called a quasi-steady dynamic wave equation. The noninertia approximation is formed by dropping both the local and convective acceleration terms. If the pressure term $\partial h/\partial t$ is dropped in addition to dropping both inertia terms, the approximation is known as the kinematic wave assumption.

MODEL TESTING

In EXTRAN, the entire sewer length is considered a single computational reach, and the dynamic wave equation is written in backward time difference between time level n+1 and n for the sewer and expressed explicitly as

$$Q_{n+1} = \left(1 + \frac{gn^2\,\Delta t}{2.21R_n^{4/3}}|V_n|\right)^{-1}\left[Q_n + 2\bar{V}_n\Delta A + \bar{V}_n^2\frac{A_{u,n} - A_{d,n}}{L}\Delta t - g\bar{A}_n\frac{h_{u,n} - h_{d,n}}{L}\Delta t\right] \tag{5}$$

in which all the symbols are as previously defined. The subscript u denotes the upstream end of a sewer (i.e., entrance) and d denotes the downstream end (i.e., exit). The bar indicates the average of values at the entrance and exit locations. The junction condition used is the continuity equation written as

$$H_{n+1} = H_n + \frac{\Delta t}{A_j}(\Sigma Q_{i,n} + Q_{j,n}) \tag{6}$$

Equations (5) and (6) are solved explicitly by using a modified Euler method and half-step and full-step calculations. Courant's stability criterion should be satisfied with the following inequality (Roesner, 1981):

Conduit: $\Delta t \leq \frac{L}{\sqrt{gD}}$

Node: $\Delta t \leq \frac{C'\,A_s\,H_{max}}{\Sigma Q}$

where: L = pipe length
C' = dimensionless constant (0.1)
D = pipe depth
H_{max} = maximum water-surface rise
A_s = corresponding surface area
ΣQ = net inflow to the junctions

Based on past experience with EXTRAN (Roesner, 1981), a time-step of 10 seconds is nearly always sufficiently small to produce outflow hydrographs and state-time traces.

Dynamic Model

This lumped parameter model (Wood and Heitzman, 1983) has certain simplifying assumptions:

- The convective terms are neglected.
- The elastic behavior of the fluid is considered negligible as compared to friction and inertial effects.
- Pipe slope is assumed to be very small ($\sin\theta = 0$) and Equation (3) reduces to $dH/dt = 0$ or $Q = A * V$ (7)

This is an incompressible, steady-state continuity equation.

- Liquid mass is treated as a rigid column in which the inertial forces are lumped together over the pipe length L.

The modified lumped parameter momentum equation is an ordinary differential equation of the form

$$\frac{L}{gA}\frac{dQ}{dt} = H_1 - H_2 - h_L \tag{8}$$

The term H represents the hydraulic gradeline or head at a given point in the system and is measured from an arbitrary datum, and h_L represents head loss. The final computational equation using an explicit forward difference scheme is expressed as

$$Q_{t+\Delta t} = Q_t + \frac{gA\Delta t}{L}(H_1 - H_2 - K_t Q_t |Q_t|) \tag{9}$$

where H_1, H_2, K_t, and Q_t are values of hydraulic grade, pipe constant, and flow rate at the beginning of the time interval. Small time steps (Δt < 5.0 seconds) should be used for all simulation.

This model can be used to analyze sewer pipe systems only under surcharge conditions.

Test Example

Figure 4 illustrates a three-sewer-line system. The system contains sewer pipes of various lengths, diameters, and slopes as listed in Table 1. Concrete sewer lines are used that have a roughness of 0.001 ft. The manhole and inflow hydrograph properties are also shown in Table 1. The total simulation time was set as 15 minutes and outfall had a constant head of 55.0 ft. The three-pipe storm sewer system is relatively flat with pipe slopes ranging from 0.001 ft/ft to 0.002 ft/ft. Systems such as this generally experience surcharge and flooding problems and are often numerically unstable (Wood and Heitzman, 1983). This is due primarily to the small difference in head between adjacent manholes resulting in unstable flow rates. In addition, the small potential head tends to minimize the system flows, resulting in larger storm detention and increasing the chances of surface flooding. The three-pipe system is analyzed using both EXTRAN and DYNAMIC. Both models provide reliable and accurate solutions under surcharge. The maximum time step for EXTRAN and DYNAMIC was 10 seconds and less than 5 seconds, respectively. This difference is simply due to the different numerical schemes used by the models. The results for hydraulic gradeline (head) computations are plotted for comparison in Figures 5 to 8. Figure 5 shows the hydraulic gradeline profile at junction 10001. In this graph, both models give the same hydraulic head up to a simulation time of 6 minutes. When simulation time increases, the head values become different. The head predicted by DYNAMIC is greater than that given by EXTRAN. The difference in head is about 10 ft at 10 minutes simulation time. This indicates the highly unstable nature of flow for this system. After 12 minutes of simulation, the two grade lines tend to merge to a constant hydraulic gradeline. At the other three junctions (shown in Figures 6 to 8) the two models yield similar results.

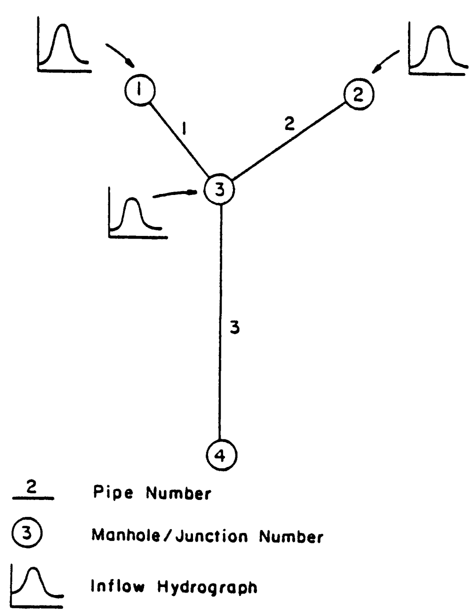

Figure 4. Three Pipe Sewer System, Example.

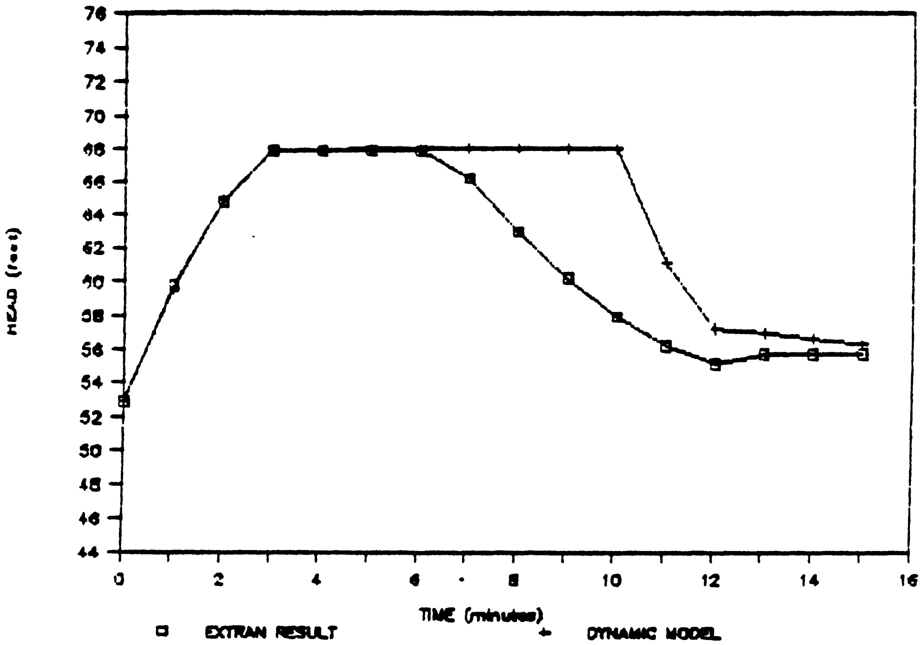

Figure 5. Total head graph for Junction 10001.

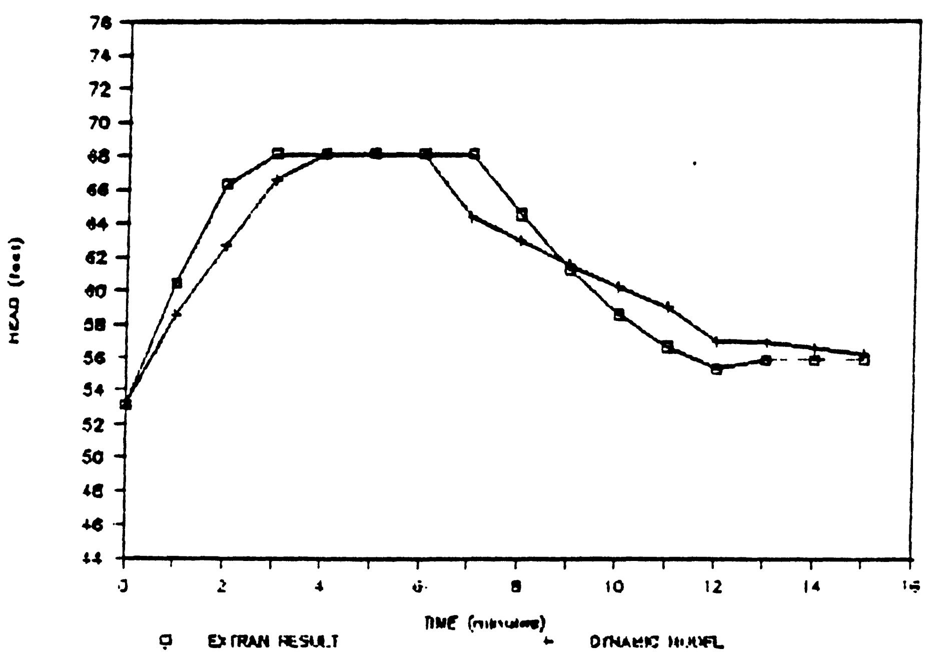

Figure 6. Total head graph for Junction 10002.

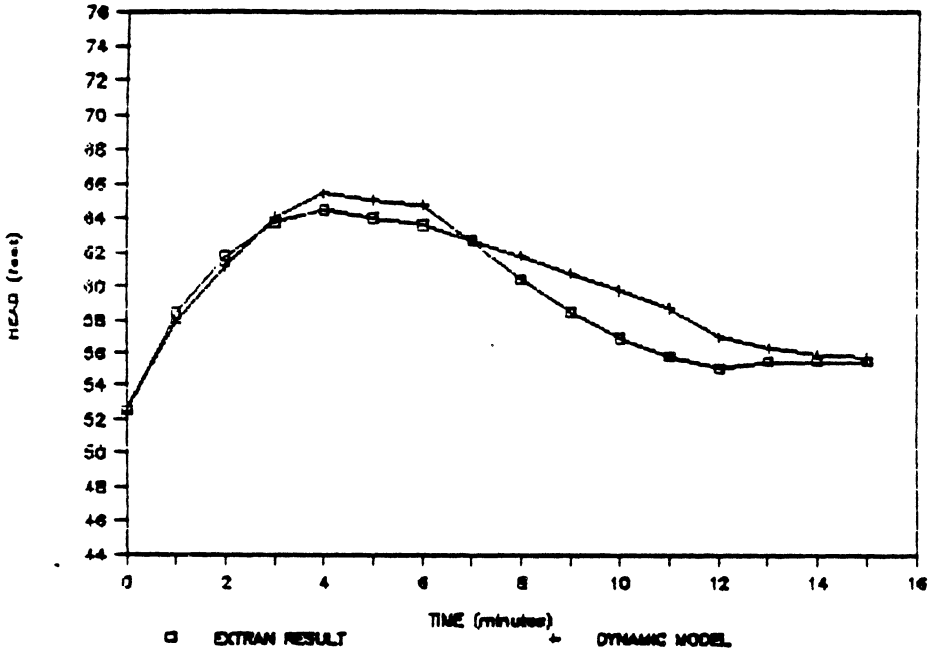

Figure 7. Total head graph for Junction 10003.

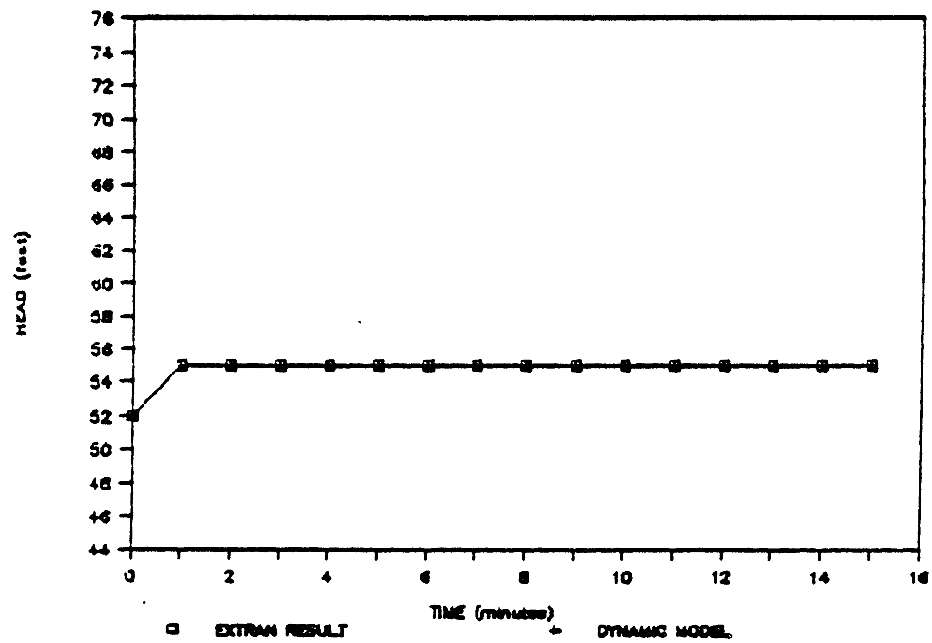

Figure 8. Total head graph for Junction 10004.

TABLE 1

Original Data Summary

The Darcy-Weisbach Head Loss Equation is used, the Kinematic Viscosity = 0.00001059 ft^2/sec

Pipe No.	Node	Numbers	Length (feet)	Diameter (inches)	Roughness (feet)	M-Loss	Initial Flowrate (CFS)
1	1	3	200.00	18.00	0.00100	0.0	5.00
2	2	3	300.00	24.00	0.00100	0.0	5.00
3	3	4	500.00	30.00	0.00100	0.0	15.00

Manhole Data

Junction No.	Elevation	Height (feet)	Diameter (inches)	Storage Diameter (feet)	Initial Head (feet)
1	52.90	15.00	36.0	150.00	55.800
2	53.10	15.00	36.0	150.00	55.600
3	52.50	15.00	48.0	150.00	55.490
4	52.00	This Junction has Fixed Head of 55.00 feet			

Hydrograph Information

Junction No.	Initial Flow (CFS)	Peak Flow (CFS)	Time Lag (Minutes)	Time To Peak (Minutes)	Time Base (Minutes)
1	5.00	30.00	0.00	4.00	12.00
2	5.00	30.00	0.00	4.00	12.00
3	5.00	30.00	0.00	4.00	12.00

CONCLUSIONS AND RECOMMENDATIONS

Among the models reviewed, only two, DYNAMIC and EXTRAN, had detailed documentation and program source listing or tapes that were available to us. These two models were tested and found to be compatible in simulating surcharge sewer flows. Neither, however, was judged totally suitable for being attached to PFP-HYDRA without certain modifications.

As a feasible alternative to using the above models, it is proposed that EXTRAN be modified and attached to PFP-HYDRA. EXTRAN uses a full dynamic wave approach that can better simulate unsteady flow characteristics in a sewer system. In addition it has the capability to handle both free-surface flow and pressurized flow. EXTRAN can be modified in several ways:

- Excess surface water could be treated as it is in DYNAMIC, i.e., water could be stored in a detention area connected to the manhole and treated as if it will return to the sewer system at a later time.

- The numerical scheme could be modified by increasing the accuracy of the solution of the differential equations.

- Depending on the preference of VDOT, we could drop some less important hydraulic structures and pipe shapes and plot subrountines from EXTRAN in order to reduce the running time.

- A modified EXTRAN could aid PFP-HYDRA in its analysis mode to give the user options to route free surface flow or open-channel and surcharge flows. It would predict the location of the surcharge pipe, the duration of the surcharge, and the flow and hydraulic gradeline at selected locations in the system.

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