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16. Abstract

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SI CONVERSION FACTORS

To Convert From	To	Multiply By
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in	m	0.025 4
ft	m	0.304 8
yd	m	0.914 4
mi	km	1 . 609 344
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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
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lb/yd^3	kg/m^3	5.932 764 E-01
Velocity: (Includes Speed)		
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mi/h	m/s	4.470 400 E-01
knot	m/s	5.144 444 E-01
mi/h	km/h	1.609 344 E+00
Force Per Unit Area:		
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lbf/ft^2	Pa	4.788 026 E+01
Viscosity:		
cS_t	m^2/s	1.000 000 E-06
P	Pa·s	1.000 000 E-01

Temperature: $(°F-32)^5/9 = °C$

FINAL REPORT

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

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and

Yin Wu
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(The opinions, findings, and conclusions expressed in this report are those of the authors and not necessarily those of the sponsoring agencies.)

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ABSTRACT

A review was made of several computer programs capable of simulating sewer flows under surcharge or pressurized flow conditions. A modified version of the EXTRAN module of the SWMM model, called PFSM, was developed and attached to the FHWA Pooled Fund PFP-HYDRA package. The microcomputer-based PFSM can be used to compute flow rate, velocity, and gradeline elevations under surcharge conditions. It will also determine the location and duration of the surcharge.

FINAL 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. Certain physical principles governing open-channel flow no longer apply when the flow becomes pressurized. Presently, several advanced computer models are available that simulate sewer flows 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 when run on mainframe computers.

The principal objective of this study was to carry out an investigation for developing a model or modifying an existing computer model to be either used as a stand-alone program or attached to the FHWA Pooled Fund Storm Sewer Program (PFP-HYDRA) for analyzing storm sewer flow under pressurized conditions. The model should accurately predict hydraulic gradeline and flow conditions under both open channel and pressurized flow conditions and will be microcomputer based. Such a model will serve as a useful tool to highway drainage engineers in checking the sewer system performance under various flow (from partial to surcharged) conditions.

REVIEW OF EXISTING STORM SEWER MODELS

Over the years, a large number of sewer models have been developed, ranging from the popular simplistic rational method (ASCE and WPCF, 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 complexing 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 this classification, a review of the major existing sewer models was made based on information available in the literature.

The SWMM is perhaps the best known among all the sewer models. The Extended Transport Block (EXTRAN) was added to SWMM Version III to provide the 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; and pumping facilities; and storage at on-line or off-line facilities. Types of channels that can be simulated include circular, rectangular, horseshoe, egg, basket handle pipes, and trapezoidal. 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, using an explicit finite difference formulation, solves for flows, 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 (Jacobsen et al., 1984), was developed at the computer center KOMMUNE-DATA in Denmark. The main features of the Danish model, called SVK-SYSTEM, are shown in Figure 1. 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 and Heitzman (1983) developed a dynamic, lumped parameter model called DYNAMIC, which 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. As with DYNAMIC, excess water is stored upstream to be released later when sewer capacity is available. Some of the most detailed treatments of surcharge flow in storm sewer systems is given by Yen (1980) and Pansic (1982) in describing 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 (see equation 6) for calculating the frictional slope. Manhole storage and surface flooding are accounted for through the 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.

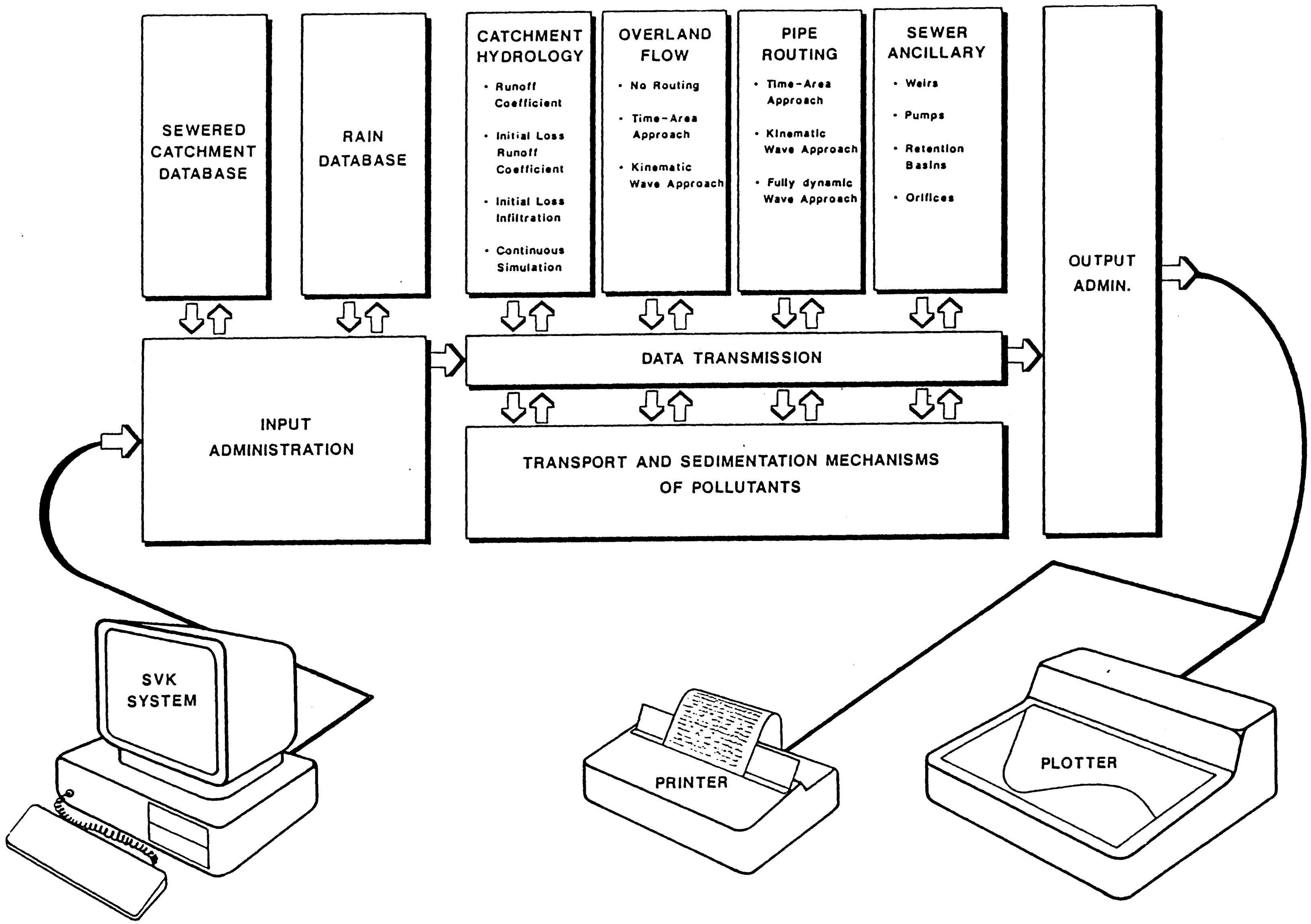

Figure 1. The prospective goal for the development of the SVK-SYSTEM.

Several storm sewer flow models treat surcharge flows by using the so-called Preissman slot technique (Wood & Heitzman, 1983). These include the French model CAREDAS (Chevereau et al., 1978) and the Danish Hydraulic Institute model, System 11 Sewer (Hoff-Clausen et al., 1982), among others. In these models, pressurized flow is artificially transformed into open-channel flow 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.

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 VDOT's use or for being attached to PFP-HYDRA without certain modifications. It was therefore decided to modify the model EXTRAN as a pressurized flow computation subroutine, called PFSM (pressurized flow simulation model), and attach it to PFP-HYDRA or use it as a stand-alone program.

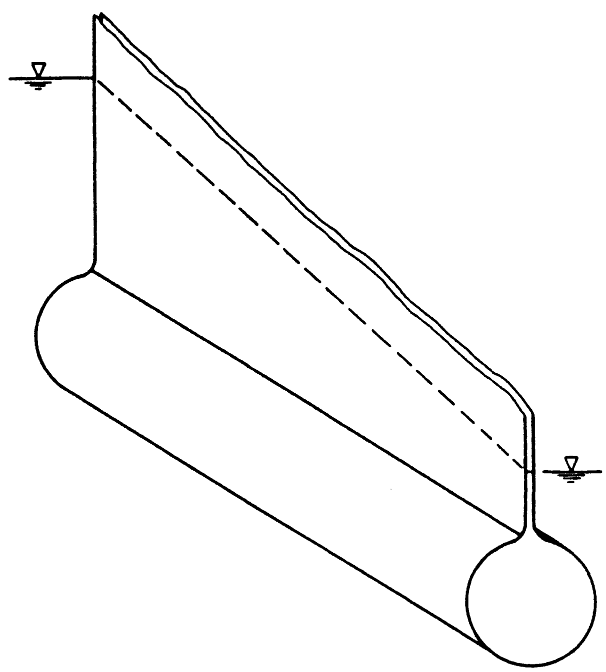

Figure 2. Hypothetical Preissman piezometric open slot.

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 (Yen, 1986) can be expressed in terms of either discharge, Q, or flow cross-sectional average velocity, V (= Q/A) (Figure 3).

Continuity:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad \text{(eq. 1)}$$

Momentum:

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

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_f = friction slope
t = time	S = sewer slope or channel 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.

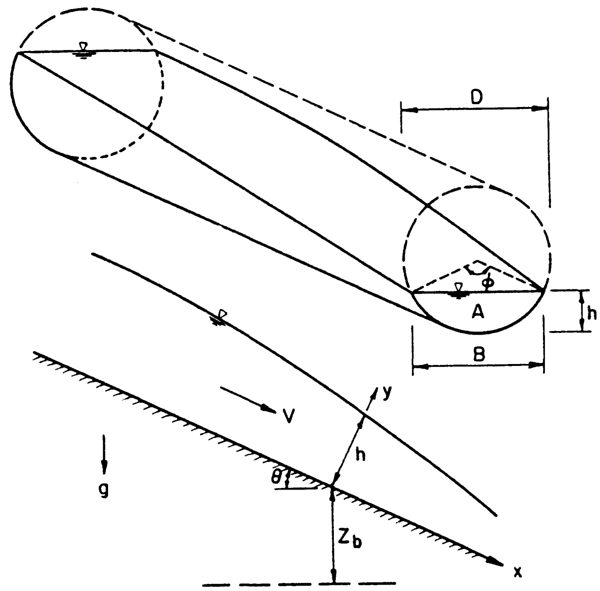

Figure 3. Open-channel flow in a sewer.

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

Continuity: $$\frac{1}{V}\frac{\partial H}{\partial t} + \frac{\partial H}{\partial x} + \frac{c^2}{gV}\frac{\partial V}{\partial x} + Sin\theta = 0 \qquad \text{(eq. 3)}$$

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

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 the location and the time from which the steady-state conditions are disturbed.

Approximations of 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 equations 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 $\partial h/\partial x$ is dropped in addition to both inertia terms, the approximation is known as the kinematic wave assumption.

METHOD OF COMPUTING PRESSURIZED FLOW IN A SEWER SYSTEM

The basic differential equation for unsteady spatially varied discharge can be written:

$$\frac{\partial Q}{\partial t} = -gAS_f + 2V\frac{\partial A}{\partial t} + V^2\frac{\partial A}{\partial x} - gA\frac{\partial H}{\partial x} \qquad \text{(eq. 5)}$$

The friction slope is defined by Manning's equation:

$$S_f = \frac{k}{gAR^{4/3}}\, Q\, |V| \qquad \text{(eq. 6)}$$

where $k = g\left[\frac{n}{1.49}\right]^2$.

Use of the absolute value sign on the velocity term makes S_f a directional quantity and ensures that the frictional force always opposes the flow. 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. It is expressed explicitly as

$$Q_{n+1} = \left(1 + \frac{gn^2\Delta t}{2.21R_n^{4/3}}|V_n|\right)^{-1}\left[Q_n + 2\bar{V}_n\Delta A + \bar{V}_n^{\,2}\frac{A_{u,n}-A_{d,n}}{L}\Delta t - g\bar{A}_n\frac{h_{u,n}-h_{d,n}}{L}\Delta t\right] \qquad \text{(eq. 7)}$$

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. Presumably, $\Delta A = A_{n+1} - A_n$ is also the average of the values at the sewer ends. The junction condition used is the continuity equation for a constant cross-sectional storage junction written as

$$\Sigma Q_i + Q_j = A_j \frac{dH}{dt} \qquad \text{(eq. 8)}$$

in which Q_i is the flow into or out of the junction by the ith joining sewer, being positive for inflow and negative for outflow; Q_j represents the direct, temporally variable water inflow into (positive) or the pumpage or overflow or leakage out of (negative) the junction, if any; $H = Y + Z$ is the water surface elevation above the reference datum and is the elevation of the junction bottom. Equation 8 can be expressed explicitly in terms of the depth and discharge values at the time $n\Delta t$ as

$$H_{n+1} = H_n + \frac{\Delta t}{A_j}(\Sigma Q_{i,n} + Q_{j,n}) \qquad \text{(eq. 9)}$$

Equations 7 and 9 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 et al., 1981):

Conduit: $$\Delta t \le \frac{L}{\sqrt{gD}} \qquad \text{(eq. 10)}$$

Node: $$\Delta t = \frac{C' A_s H_{\max}}{\Sigma Q} \qquad \text{(eq. 11)}$$

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.

Examination of equations 10 and 11 reveals that the maximum allowable time, Δt, will be determined by the shortest, smallest pipe having high inflows. Based on past experience with EXTRAN (Roesner et al., 1981), a time-step of 10 seconds is nearly always sufficiently small to produce outflow hydrographs and state-time traces. In most applications, 15- to 30-second time-steps are adequate. Occasionally, time steps up to 60 seconds can be used.

Head Computation During Surcharge and Flooding

Surcharge occurs when all pipes entering a node are full or when the water surface at the node lies between the crown of the highest entering pipe and the ground surface.

During surcharge, the head calculation in equation 8 is no longer possible because the surface area of the surcharged node is zero. Thus, the continuity equation becomes

$$\Sigma Q_i(t) + Q_j(t) = 0 \qquad \text{(eq. 12)}$$

Since the flow and continuity are not solved simultaneously in the model, the flows computed in the links connected to node j will not satisfy equation 12. However, computing Q/H_j for each link connected to node j, a head adjustment can be computed such that the continuity equation is satisfied. Rearranging equation 12 in terms of the adjusted head gives

$$\Delta H_j(t) = -\frac{\Sigma Q_i(t) + Q_j}{\Sigma \dfrac{\partial Q(t)}{\partial H_j}} \qquad \text{(eq. 13)}$$

This adjustment is made by half-steps during surcharge by the introduction of an adjustment factor and by the assumption that either the numerical iterations will reach a maximum number set by the user or the algebraic sum of the inflows and outflows of a junction will be less than tolerance.

Flooding is a special case of surcharge. It takes place when the hydraulic gradeline breaks to ground surface and water is lost from the sewer node to the overlying surface system. Once flooding occurs, the program allows the excess junction inflow to "overflow onto the ground" and become lost from the system for the remainder of the simulation period.

Figure 4 shows each of the possibilities and shows the way in which surface area is assigned to the nodes. Assumptions made for the special pipe flow are

1. Normal case: Computed flow from motion equation. Assign half of the surface area to each node.

2. Critical depth downstream: Use lesser of critical or normal depth downstream. Assign all surface area to upstream node.

3. Critical depth upstream: Use critical depth. Assign all surface area to downstream node.

4. Flow computed exceeds normal flow at a supercritical depth: Set flow to normal value. Assign surface area in usual manner as in 1.

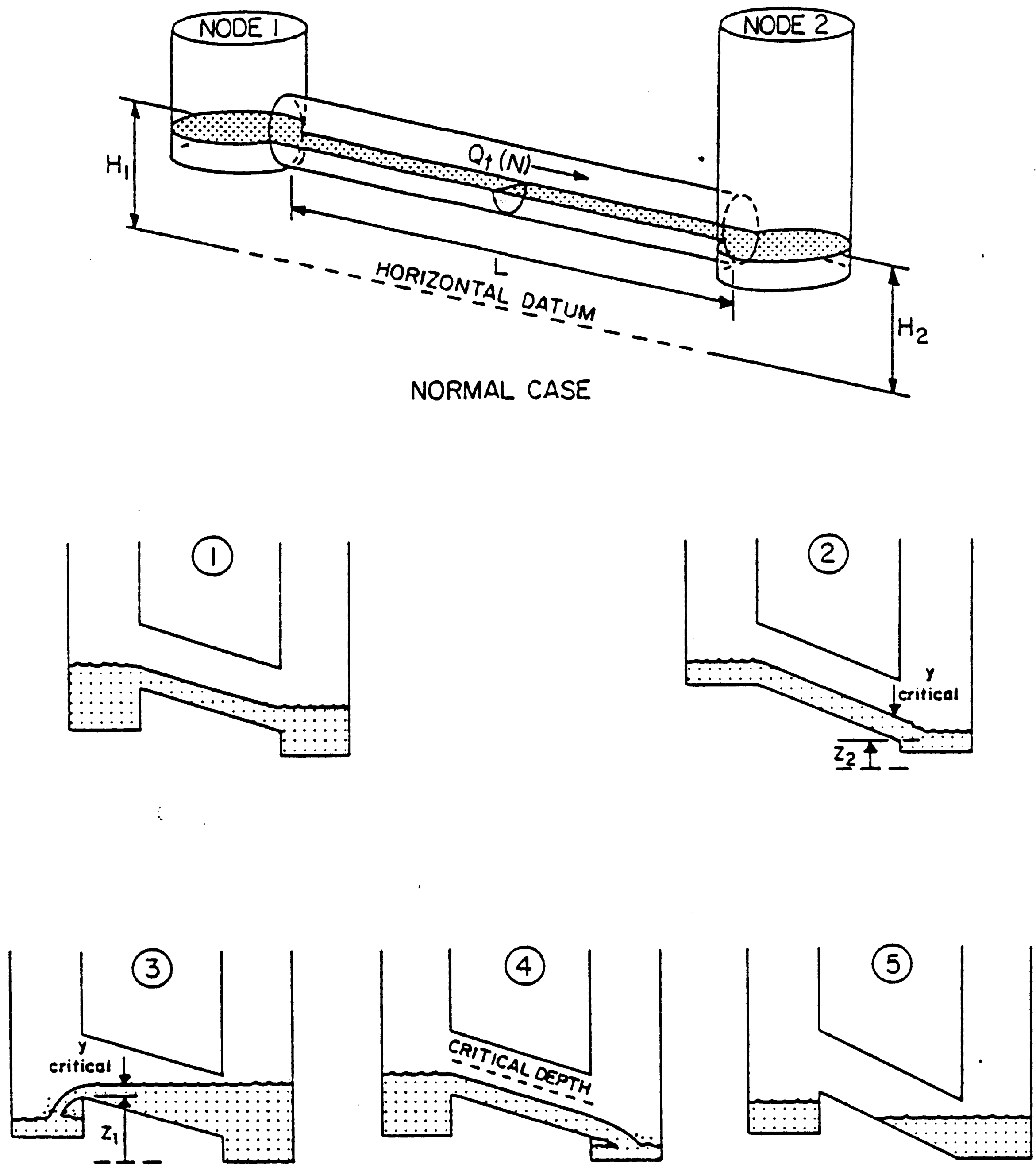

Figure 4. Special hydraulic cases in PFSM pipe flow calculations.

5. Dry pipe: Set flow to zero. If any surface area exists, assign to downstream node.

Once these depth and surface area corrections are applied, the computations of head and discharge can proceed in the normal way for the current time-step. Any of these special situations may begin and end at various times and places during simulation. PFSM detects these automatically.

DEVELOPMENT OF THE PFSM SUBROUTINE FOR PFP-HYDRA

The PFSM is essentially a modified EXTRAN module, which follows the theoretical background and numerical algorithm of EXTRAN. As a result of suggestions made by VDOT, the PFSM has been developed by dropping some less important hydraulic structures, pipe shapes, and plot subroutines from EXTRAN in order to reduce the running time. In addition, two subroutines were developed: one of them carries out the connection between the PFP-HYDRA and PFSM, and the other generates a triangular hydrograph at each junction based on the rational formula with data obtained from the STO command statement in PFP-HYDRA.

The Rational Method as a Hydrograph

Consider a rainfall of constant intensity, I, uniformly distributed over a particular drainage basin and of a duration, D, as shown in Figure 5 (Wanielista, 1978). The volume of runoff (rainfall excess), V_1, is equal to CIDA, where C is the runoff coefficient in the rational method that represents the ratio of the total volume of the runoff to the total volume of rainfall; I is the intensity of rainfall in in/hr; D is the rainfall duration in hr; and A is the area of the drainage basin in acres.

The following assumptions are made:

1. The duration of rainfall, D, is equal to the time of concentration (i.e., $D = T_c$, where T_c is the time of concentration of the drainage area).

2. The time to peak is equal to the time of concentration (i.e., $T_p = T_c$).

3. The recession curve portion of the hydrograph length is equal to T_c.

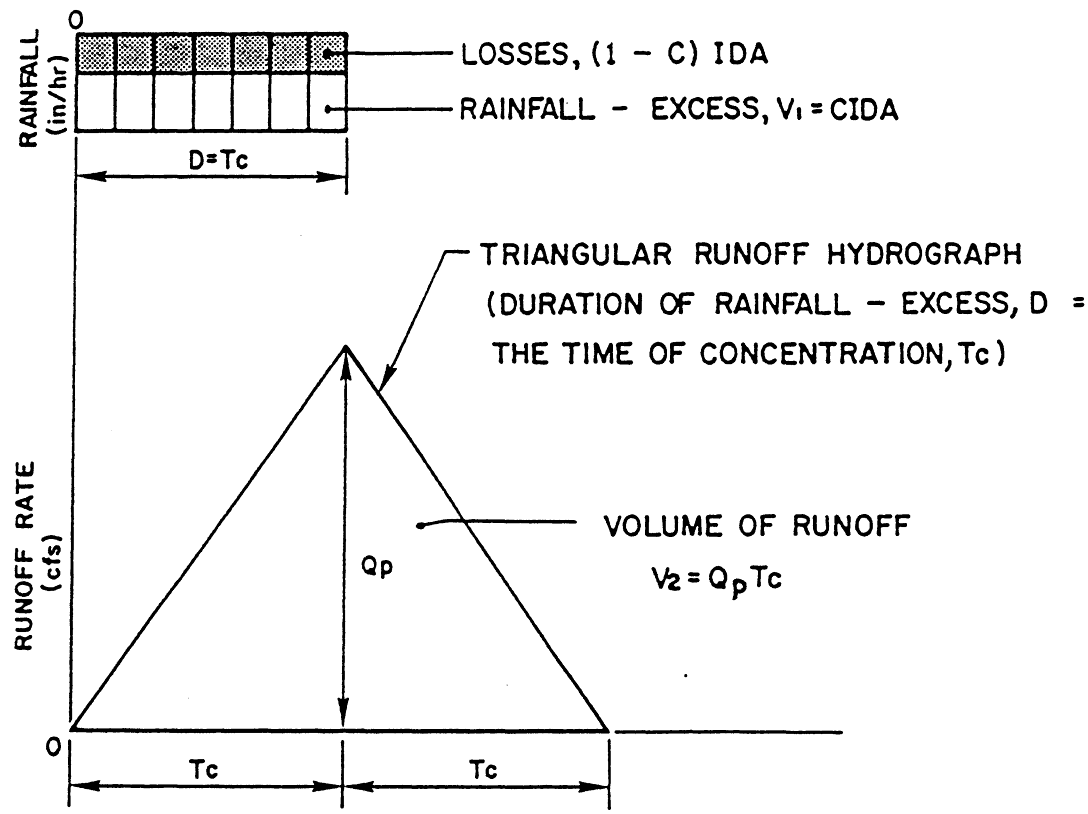

Figure 5. Rainfall hyetograph (rational method).

From Figure 5, the volume of runoff, V_1, as represented by the lower portion of the hyetograph, must equal the volume of the triangular hydrograph, V_2. Therefore, CIDA = Q_pT_c and T_c = D; the peak Q_p of this hyetograph is computed to be equal to CIA. Thus, a triangular hydrograph has been established.

Structure of PFSM

PFSM is a set of computer subroutines organized to simulate the unsteady, gradually varied movement of stormwater in a sewer network composed of pipes, junctions, and a free outfall. The PFSM contains one main controlling program, nine subprograms, and one function. The organization of each subprogram and its relation to the main program, PFP-HYDRA, and the hydraulic gradeline computation subroutine, GRADE, is shown in the flow chart in Figure 6. A description of each subroutine follows:

Subroutine PFSM — PFSM is the main controlling subroutine of pressurized flow computation that drives all other subroutines. It carries out the modified Euler method and uses half-step and full-step calculations.

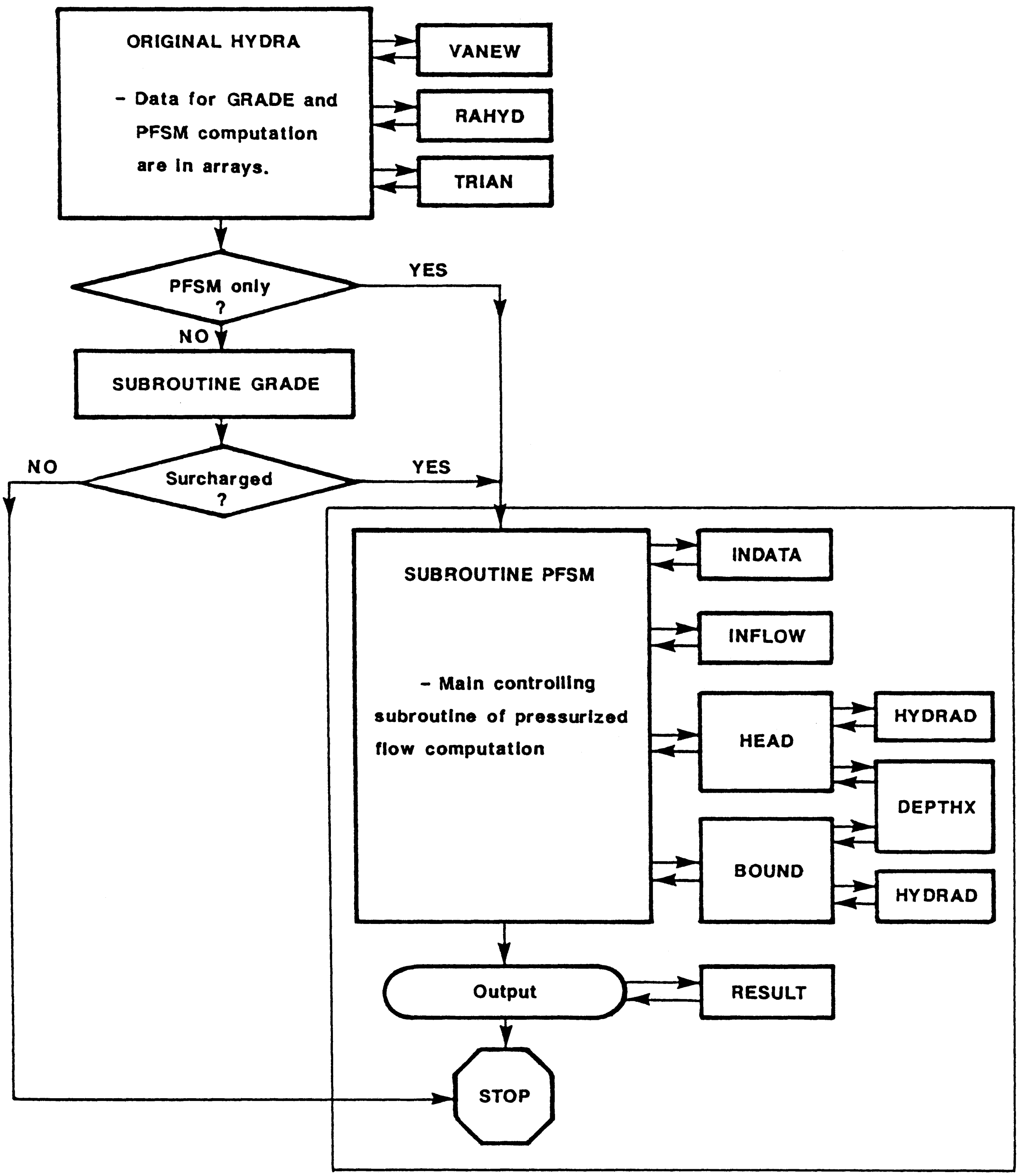

Figure 6. Flow chart of the modified HYDRA.

Principal steps in PFSM are

1. Call INDATA for transferring input data cards from PFP-HYDRA (these cards will be used in pressurized flow computation).

2. Initialize system-flow properties, and set time = TZERO.

3. Advance time = t + Δt, and begin main computation loop (steps 4 through 10).

4. Select current value of inflow hydrographs for all input nodes by call to INFLOW.

5. For all physical pipes in the system, compute the following time-changing properties based on the last full-step values of depth and flow: compute the full-step area, velocity, and hydraulic radius; compute the half-step flow, and then check for normal flow; compute the half-step of depth and discharge at the outfall by a call to BOUND; and average flow in all pipes connected to junctions in surcharge. A fraction of this value is used as the tolerance of the surcharge iteration loop.

6. For all physical junctions in the system, compute the half-step depth at time t = t + Δt.

7. Compute all physical pipe properties based on the last half-step values of depth and flow [repeat step 5 for time t + ($\Delta t/2$)].

8. For all junctions, repeat the nodal head computation of step 6 for time t + Δt.

9. Repeat steps 7 and 8 for surcharged links and nodes until the sum of the flow difference from step 8 is less than the tolerance from step 5 or a maximum number of iterations is exceeded.

10. Store nodal water depths and water surface in junction print arrays to be used later by RESULT. Also, store pipe discharge and velocities for later printing.

11. Return to step 3 and repeat through step 10 until the transport simulation is complete for the entire period.

12. Call subroutine RESULT for printing pipe flows and junction water surface elevations.

Subroutine BOUND   Compute the current level backwater from the receiving water and determine discharge through the system outfall.

Subroutine HEAD	Convert nodal depths to pipe depths, and assign surface area to the upstream and downstream node.
Subroutine HYDRAD	Compute average values of hydraulic radius, cross-sectional area, and surface width for all pipes.
Subroutine INDATA	Establish the connection with PFP-HYDRA and set up internal numbering system input data for junctions and pipes. The junction invert elevation is defined as the invert elevation of the lowest pipe connected to the junction. The explanation of ground and invert elevations is shown in Figure 7.
Subroutine INFLOW	Compute the current value of hydrograph inflow at each node at the half-step time, $t + (\Delta t/2)$.
Subroutine RAHYD	Generate a triangular hydrograph at each junction.
Subroutine RESULT	Print water depths and water surface elevations at each junction, print discharge and velocity at each pipe, and store all these values for a later plot.
Subroutine TRAIN	Store the time and inflow values of hydrographs.
Subroutine VANEW	Store pressurized flow computation parameters and new command-card information, such as PHJ, PFP, HHD, etc.
Function DISS	Perform a linear interpolation between hydrograph points.

For preparing the input data (i.e., EXAMPLE.HDA) for PFP-HYDRA, users are required to put at least three new command statements in the original HYDRA. The new command statements are PFA, PHJ, and PFP, which provide the simulation time, integration time step, and some output specifications. Other cards are optional depending on whether the information is available to users. Those new commands should be placed after the last PNC statement in the input file. These commands are summarized in Appendix A.

In PFP-HYDRA, the original HYDRA will be run first. It will then carry out hydraulic gradeline computations. After this is done, if the system has the possibility of surcharge or surface flooding during storms, the program will continue the pressurized flow simulation. Otherwise, it will stop the computation. Another option is to run only the pressurized flow simulation by giving a parameter indicated in the PFA statement if inflow hydrographs are given. This means that the program will go directly into the PFSM subroutine rather than computing hydraulic gradeline by using the GRADE subroutine. In this case, the command cards HHJ and HHD will be needed. HHD gives the time and inflow values at each junction, and it repeats four times to perform a triangular hydrograph and an extension point of the hydrograph, whose time value is either the same or longer than the total simulation period (by default, the fourth time value = 50 T_c).

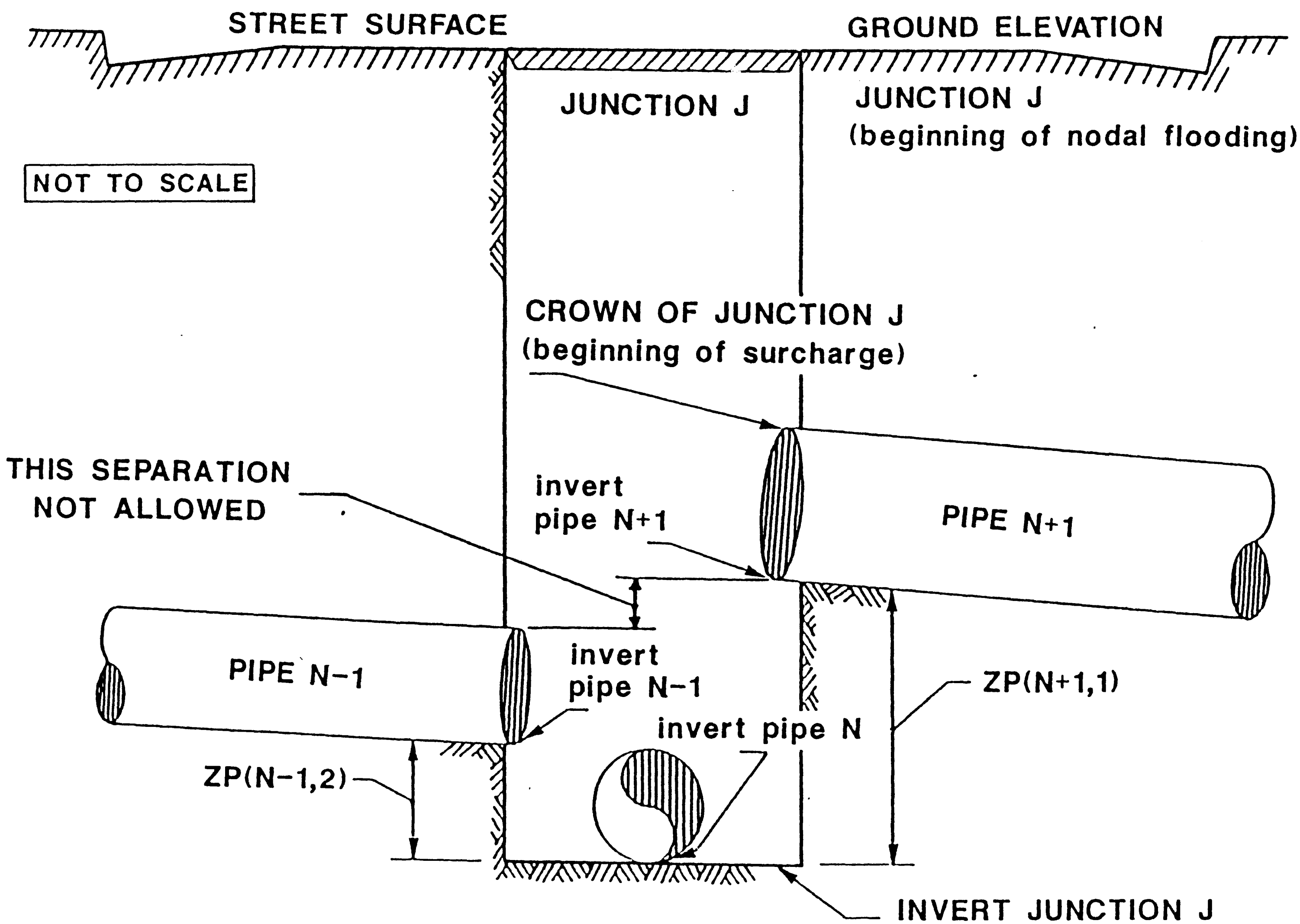

Figure 7. Definition of elevation terms for three-pipe junction.

Change of PNC and SWI Command

The PNC command requires three more parameters to fit the needs of the PFSM and GRADE subroutines. One of them indicates the inlet shaping; if there is inlet shaping, the total minor losses will be reduced by 50 percent (VDOT, 1980).

The SWI command adds one more switch to carry out both open-channel and surcharged flow computations (see Appendix A).

Application of the Modified PFP-HYDRA

Two examples tested by the modified PFP-HYDRA illustrate both open-channel and pressurized flow computations for common design and analysis purposes. The application of the new commands are introduced here.

Example 1

The sample sewer network contains four sewers of different lengths and ground elevations (see Figure 8). It gives the intensity curve and parameters required by the rational method. The Manning roughness factor is 0.013 for all the sewers. The tailwater elevation of 100 ft is assumed in order to generate surcharging situation. The time step used in the numerical computation is t = 10 sec, and the total simulation time is 20 min. The interval between each printout is 1 min.

The complete output for Example 1 is found in Appendix B. The output is divided into three parts, namely, the output of original HYDRA, that of the open-channel hydraulic gradeline, and that of pressurized flow. In the last section for pressurized flow results, it shows first an echo of the input data for simulation and a listing of pipes and junctions to be printed. It lists system inflows as they are given by hydrographs and gives the depth at each junction and flow in each pipe in the system at a user-input time interval. A junction in surcharge is indicated by printing an asterisk beside its depth. Also, if surcharge iterations are occurring at the time of the intermediate printout, HYDRA prints the flow differential over all surcharged junctions and the number of iterations required. An asterisk beside a pipe flow indicates that the flow is the normal flow for the pipe. The intermediate printout ends with the printing of a continuity balance of the water passing through the system during the simulation. Finally, it obtains the time history of depths and flows for those junctions and pipes by users and a summary for all junctions and pipes in the system (see Appendix B).

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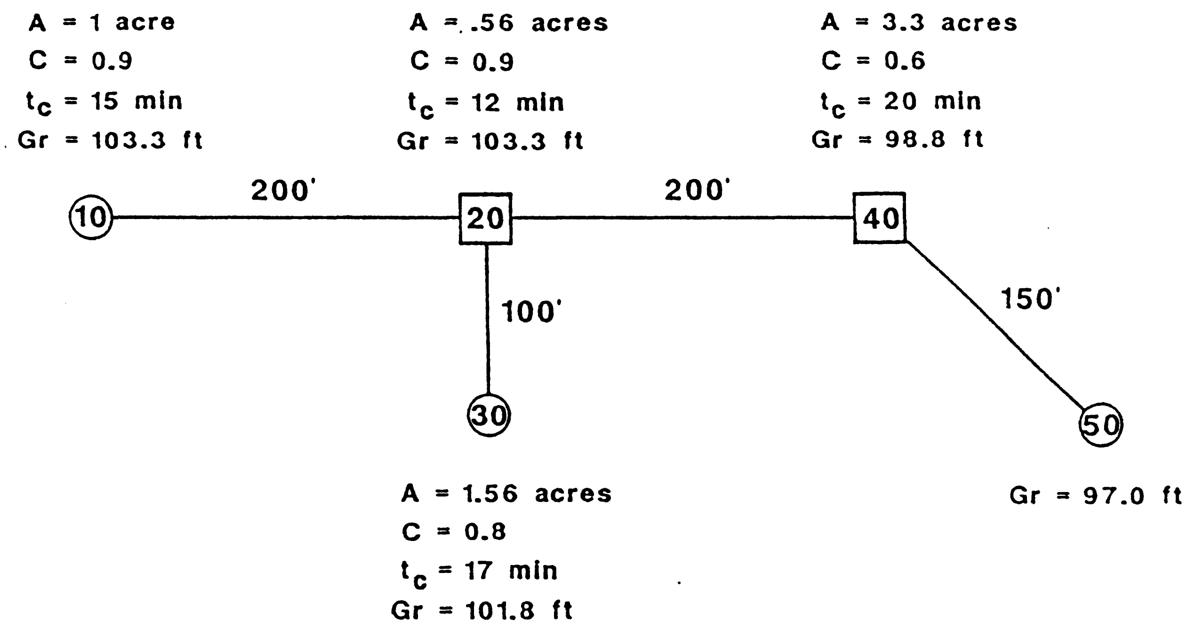

IDF time/rain

0/7.1 5/7.1 10/6 15/5.1 20/4.5 30/3.6 40/3 50/2.6 120/1.4 150/1.4
60/2.3 120/1.4 150/1.4

Figure 8. A sewer network of Example 1.

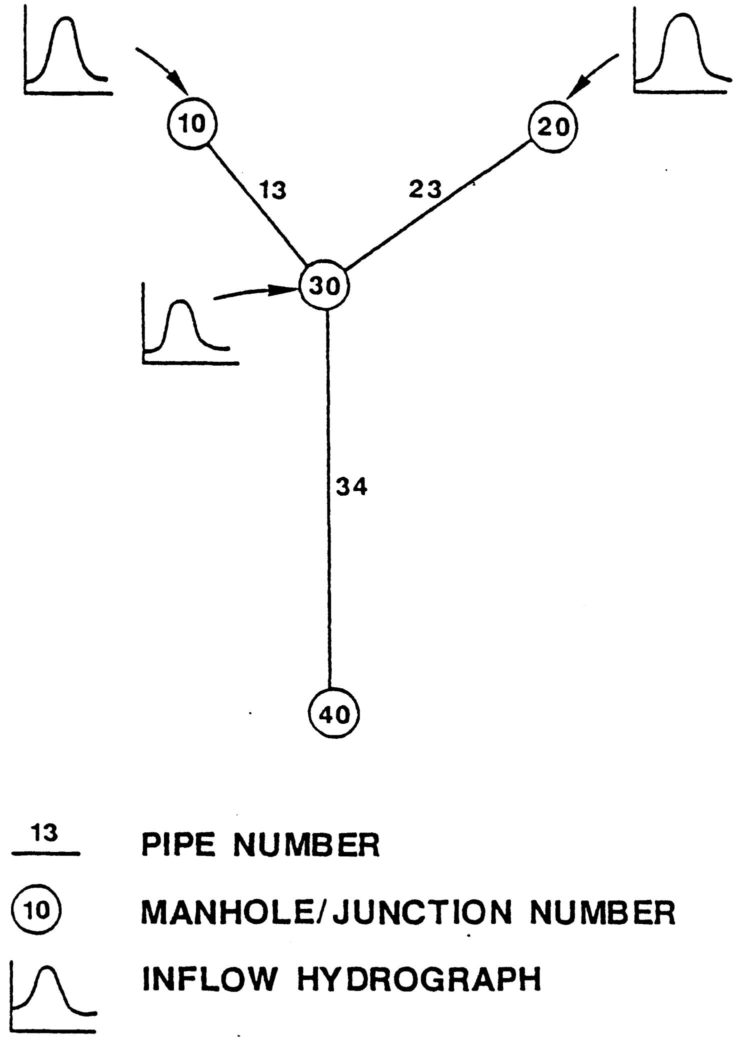

Figure 9. Three-pipe sewer system of Example 2.

Example 2

Example 2 demonstrates that HYDRA runs both open-channel and surcharged flow by calling the PFSM subroutine only when inflow hydrographs at junctions are given. Figure 9 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 for 20 min, 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 have surcharge and flooding problems and are often numerically unstable (Wood & 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 increased chances of surface flooding.

TABLE 1

Original Data Summary[1]

Pipe No.	Nodes	Numbers	Length (ft)	Diameter (in)	Roughness (ft)	M-Loss	Initial Flow Rate (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
2	3	4	500.00	30.00	0.00100	0.0	15.00

Manhole Data

Junction No.	Elevation	Height (ft)	Diameter (in)	Storage Diameter (ft)	Initial Head (ft)
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	48.00	150.00	55.490
4	52.00	This junction has fixed head of 55.00 ft			

Hydrograph Information

Junction No.	Initial Flow (cfs)	Peak Flow (cfs)	Time Lag (min)	Time to Peak min)	Time Base (min)
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

[1] The Darcy-Weisbach head loss equation is used; the kinematic viscosity = 0.00001059 ft^2/sec.

The results for this example are shown in Appendix C, and the hydraulic gradeline (head) computation at junction 10 is plotted in Figure 10. This graph shows that the head value becomes different when simulation time increases. The difference in head at junction 10 is about 10 ft at 10 minutes simulation time. This indicates the highly unstable nature of flow within the system.

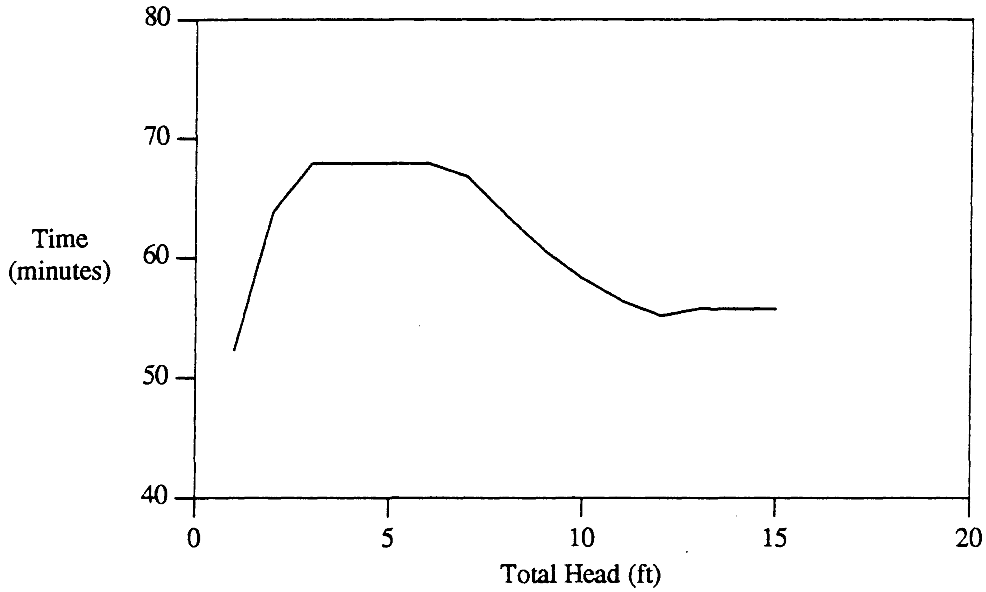

Figure 10. The hydraulic gradeline (head) computation at junction 10.

CONCLUSIONS

1. With certain modifications, the EXTRAN module of the SWMM model appears to suit the needs of VDOT best and is also part of the FHWA Pooled Integrated Drainage package.

2. The modified EXTRAN program, PFSM, follows the theoretical background and numerical algorithms of EXTRAN. Upon suggestions made by VDOT, certain unnecessary features such as tidal gates, flow reversals, etc. were deleted so that running time on a microcomputer would not be excessive.

3. Two new subroutines were developed as part of PFSM. One subroutine connects PFSM with PFP-HYDRA, and the other generates a triangular hydrograph at each junction based on the rational formula and procedures used by VDOT.

4. With the addition of PFSM, the capability of PFP-HYDRA is significantly enhanced. PFSM can predict the locations and duration of surcharge as well as flow rate, velocity, and hydraulic gradeline at selected locations in the sewer system.

RECOMMENDATIONS

1. Program PFSM, derived from the EXTRAN module of the model SWMM, should be used as a sewer analysis tool when there is a possibility that the pipes might be surcharged. PFSM is attached to the FHWA Pooled Fund's PFP-HYDRA program, but it can also be used as a stand-alone program.

2. PFSM should be further enhanced to include features such as the use of a synthetic hydrograph method instead of the rational formula as the basis of hydrograph generation. Other potential enhancements include the development of an expert system module that would help the user select pertinent parameters such as time-step and cycles for the numerical solution of the flow equations.

ACKNOWLEDGMENTS

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APPENDIX A

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APPENDIX A

PFP-HYDRA New Commands

COMMAND: Ben - Pipe BENd data

Purpose: This command is to specify the bend angle and radius for the computation of losses due to curved aligment of pipe as shown in Figure A-1. This is usually placed after the PNC statement to indicate that a bend occurs at the link specified by the previous PIP statement.

Structure:

BEN F1, F2

1) F1 - Bend radius of the link specified by the previous PIP statement (ft).

2) F2 - Bend angle of the link specified by the previous PIP statement (degree).

Note: Bend angle is usually between 0 and 120 degrees.

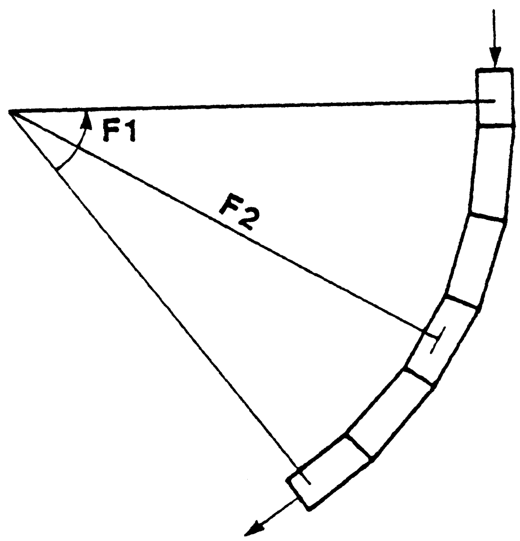


Figure A-1. Description of BEN command statement.

COMMAND: HHD - Hydrograph Data

Purpose: This command provides inflow hydrographs.

Structure:

HHD Time, $Inflow_1$, $Inflow_2$, $Inflow_{NJSW}$

1) Time - Clock time (hr)

2) $Inflow_1$, . . . - Flow rate (cfs)

Note: The hydrograph distribution is assumed to be a triangular hydrograph. Only four points are required for the time and discharge values.

COMMAND: HHJ - Hydrograph Junction Input

Purpose: Junctions have hydrographs.

Structure:

HHJ $Hyjun_1$, $Hyjun_2$, . . . $Hyjun_{NJSW}$

1) $Hyjun_1$ - Junction number for a hydrograph.

Note: This card is needed when PFP-HYDRA runs only pressurized flow or any other option except the rational formula option.

COMMAND: IDY - Initial Depth

Purpose: This command supplies the initial conditions for the initial depth in the same sequence as with the IQV described.

Structure:

IDY y_1, y_2, . . .

1) y_1, y_2, . . . Initial depth (ft)

COMMAND: IQV - Initial Discharge and Velocity

Purpose: This command supplies the initial conditions throughout the drainage system at the beginning of the simulation. The command contains initial discharge and velocity in the same order as PNC specified at upstream nodes but the outfall at downstream nodes.

Structure:

IQV Dis/Vel, Dis/Vel, . . .

1) Dis - Initial discharge (cfs)

2) Vel - Initial velocity (fps)

COMMAND: PFA - Pressurized Flow Data

Purpose: This command defines control parameters for running pressurized flow.

Structure:

PFA NTCYC, DELT, TZERO, NHPRT, NQPRT, NSTART, INTER, NJSW, ITMAX, SURTOL, LPFSM

1) NTCYC Number of integration steps or time cycles desired.

2) DELT Length of integration step (sec).

3) TZERO Start time of simulation (hr).

4) NHPRT Number of junctions for detailed printing of head output (20 nodes max.).

5) NQPRT Number of pipes for detailed printing of discharge output (20 pipes max.).

6) NSTART First time-step to begin print cycle.

7) INTER Interval between print cycles (max. number of cycles printed is $\frac{\text{NTCYC - NSTART}}{\text{(INTER)}} < 100$.

8) NJSW Number of junctions having input hydrographs.

9) ITMAX Maximum number of iterations to adjust head and flow of surcharged junctions.

10) SURTOL Segment of flow in surcharged area to be used as the tolerance for ending surcharge iterations.

11) LPFSM Run pressurized flow simulation only (option) combined with selecting SWI command as 6 (1 for PFSM only, 0 by default).

Notes:

1. The time-step, DELT, is most critical to the cost and stability of the PFSM subroutine and must be selected carefully. The time-step should be selected according to the final report described in METHOD OF COMPUTING PRESSURIZED FLOW IN A SEWER SYSTEM (see equations 10 and 11). The computer program will check each pipe for violation of the surface wave criteria and will print the warning message. Be sure to check these messages.

2. The length of the total simulation period defined as the product of NTCYC and DELT.

3. ITMAX and SURTOL control the accuracy of the solution in surcharged areas. In reality, the inflow to a surcharged area should equal the outflow from it. Therefore, the flows and heads in surcharged areas are recalculated until either the difference in inflows and outflows is less than a tolerance, which is defined as SURTOL times the average flow in the surcharged area, or the number of iterations exceeds ITMAX. It has been found that good starting values for ITMAX and SURTOL are 30 and 0.05, respectively.

COMMAND: PFP - Printed Flow Pipe

Purpose: This command contains the list of individual pipes (up to 20) for which flows and velocities are to be printed.

Structure:

PFP $Pipe_1$, $Pipe_2$, . . . $Pipe_{NQPRT}$

1) $Pipe_1$ - Pipe number printed

COMMAND: PHJ - Printed Heads Junction

Purpose: This command contains the list of individual junctions (up to 20) for which water depth and water surface elevations are to be printed continuously throughout the course of the simulation period.

Structure:

PHJ Jun_1, Jun_2, . . . Jun_{NHPRT}

1) Jun_1 - Junction number printed

COMMAND: PNC - Pipe Node Connection

Purpose: This command is to specify the connection of links and nodes for the computation of the hydraulic gradeline. Each PNC statement must immediately follows the PIP statement.

Structure:

PNC I1, I2, I3, I4, I5, F6, I7, F8, F9, (F10), (F11), (F12), (F13), (F14), (F15)

1) I1 Pipe number

2) I2 Node Number connecting the upstream end of the link specified by the previous PIP statement.

3) I3 Type of node I2 (1 for manhole; 2 for pipe junction; 3 for pump; 5 for terminal manhole; any other numbers are invalid).

4) I4 Node Number connecting the downstream end of the link specified by the previous PIP statement.

5) I5 Type of node I4 (1 for manhole; 2 for pipe junction; 3 for pump; 4 for outfall point; any other numbers are invalid).

6) I6 Identification of the link specified by the previous PIP statement as mainline link (1 for Yes; 2 for No).

7) F7 Deflection angle of the mainline link (degree).

8) I8 Identification of the link specified by the previous PIP statement as sideline link (1 for Yes; 2 for No).

9) F9 Skew angle of the sideline link (degree).

10) (F10) Loss coefficient for terminal nodes (e.g.,) terminal manhole loss coefficient; entrance loss coefficient).

11) (F11) Tailwater elevation at the point of the system's outlet (optional).

12) (F12) Minor loss coefficient. This is only required when the downstream velocity is less than the upstream velocity within a pipe.

13) (F13) Distance of pipe invert above junction invert at upstream end.

14) (F14) Distance of pipe invert above junction invert at downstream end.

15) (F15) Identification of inlet shaping (1 for inlet dropping; 0 by default). With the inlet shaping option, the value of total minor losses, H_t is reduced by 50 percent according to the <u>Drainage Manual</u> (VDOT, 1980).

COMMAND: SWI - Criteria SWItch

Purpose: This command establishes the method by which PFP-HYDRA is to analyze storm flows.

Structure:

Switch - a number 1, 2, 3, 4, 5, or 6

1) 1 Sanitary analysis only.

2) 2 Storm analysis - rational method only.

3) 3 Storm analysis - hydrographic method only.

4) 4 Sanitary and rational analysis.

5) 5 Sanitary and hydrographic analysis.

6) 6 Pressurized Flow Simulation only combined with the 11th parameter, LPFSM, of the PFA statement.

APPENDIX B

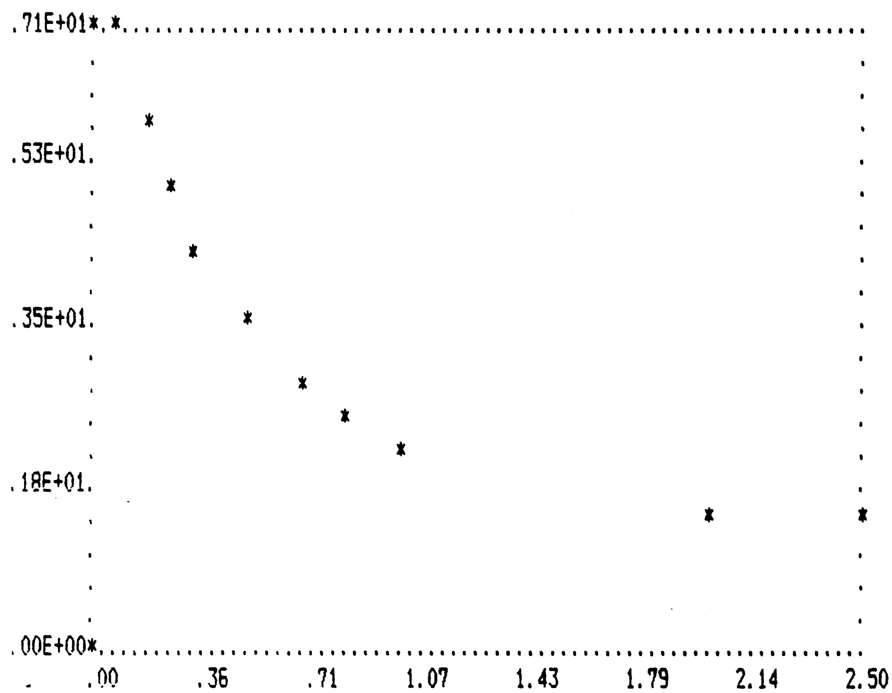
APPENDIX B

```
*** PFP-HYDRA (Version of Oct. 2, 1986) ***          DATE 03-03-89
                                                     PAGE NO   1


    PROBLEM 1            FILENAME: EX1.HDA

Commands Read From File example.hda
   10 JOB
   20 SWI 2
   30 CRI 0
   40 PDA .013 12 3 2 2 .002
   50 RAI 0 7.1 5 7.1 10 6 15 5.1 20 4.5 30 3.6 40 3 50 2.6 60 2.3 +
        120 1.4 150 1.4
```

IDF CURVE



PLOT-DATA (VALUE Vs.TIME)

.000	7.100	2.500	1.400	.000	.000	.000	.000	.000	.000
.083	7.100	.000	.000	.000	.000	.000	.000	.000	.000
.167	6.000	.000	.000	.000	.000	.000	.000	.000	.000
.250	5.100	.000	.000	.000	.000	.000	.000	.000	.000
.333	4.500	.000	.000	.000	.000	.000	.000	.000	.000
.500	3.600	.000	.000	.000	.000	.000	.000	.000	.000
.667	3.000	.000	.000	.000	.000	.000	.000	.000	.000
.833	2.600	.000	.000	.000	.000	.000	.000	.000	.000
1.000	2.300	.000	.000	.000	-99.000	.000	.000	.000	.000
2.000	1.400	.000	.000	.000	.000	.000	.000	.000	.000

```
   70 NEW LATERAL 1-2
   80 STO 1 .9 15
   90 PIP 200 103.3 101.3
```

```
*** PFP-HYDRA (Version of Oct. 2, 1986) ***          DATE 03-03-89
                                                      PAGE NO   2

      PROBLEM 1              FILENAME: EX1.HDA
 100 PNC 12 10 5 20 1 0 0 1 0 1.5
 110 HOL 1
 120 NEW T L 3-2
 130 STO 1.56 .8 17
 140 PIP 100 101.8 101.3
 150 PNC 32 30 5 20 1 1 90 0 0 1.5
 160 REM T L 2-4
 170 REC 1
 180 STO .56 .9 12
 190 PIP 200 101.3 98.8
 200 PNC 24 20 1 40 1 1 45 0 0
 210 REM T L 4-5(OF)
 220 STO 3.3 .6 20
 230 PIP 150 98.8 97.0 94.5 91.8
 240 PNC 45 40 1 50 4 1 0 0 0 0 100.0
 241 PFA 120 10. 0. 4 4 6 6 4 30 0.05
 242 PHJ 10 20 30 40
 243 PFP 12 32 24 45
 250 END
END OF RUN.
```

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PROBLEM 1　　FILENAME: EX1.HDA

*** LATERAL 1-2　　Pipe Design

Link	Length (ft)	Diam (in)	Invert Up/Dn (ft)	Slope (ft/ft)	Depth Up/Dn (ft)	Min. Cover (ft)	Velocity Act/Full (ft/sec)	--Flow-- Act/Full (cfs)	Estimated Cost ($)
1	200	18	99.7	.01000	3.6	2.0	5.8	4.59	0.
			97.7		3.6		6.0	10.53	

LENGTH = 200.　COST = 0.
TOTAL LENGTH = 200.　TOTAL COST = 0.

*** T L 3-2　　Pipe Design

Link	Length (ft)	Diam (in)	Invert Up/Dn (ft)	Slope (ft/ft)	Depth Up/Dn (ft)	Min. Cover (ft)	Velocity Act/Full (ft/sec)	--Flow-- Act/Full (cfs)	Estimated Cost ($)
2	100	18	98.2	.00500	3.6	2.0	4.7	6.07	0.
			97.7		3.6		4.2	7.45	
3	200	24	96.5	.00938	4.8	2.0	7.3	12.78	0.
			94.6		4.2		7.0	21.96	
4	150	24	94.5	.01800	4.3	2.1	10.4	20.84	0.
			91.8		5.2		9.7	30.43	

LENGTH = 450.　COST = 0.
TOTAL LENGTH = 650.　TOTAL COST = 0.

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Link	Node Number U/S	Node Number D/S	Node Type U/S	Node Type D/S	Main Line	Deflected Angle	Side Line	Skew Angle	Bend Radius [Ft]	Bend Angle
1	10	20	5	1	0	.0	1	.0	.00	.0
2	30	20	5	1	1	90.0	0	.0	.00	.0
3	20	40	1	1	1	45.0	0	.0	.00	.0
4	40	50	1	4	1	.0	0	.0	.00	.0

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Node#	Potential Water Level (Ft)	Ground Level (Ft)	Lowest Crown Elevation of Links Connecting Node Link#	Elevation (Ft)	Location	Possible Surcharging to the Link
10	105.1	103.3	1	101.2	Upstream	Yes
20	103.9	101.3	3	98.5	Upstream	Yes
30	104.8	101.8	2	99.7	Upstream	Yes
40	102.7	98.8	4	96.5	Upstream	Yes
50	100.0	97.0	4	93.8	Downstream	Yes

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***** PRESSURIZED FLOW SIMULATIONS *****

INTEGRATION CYCLES 120
LENGTH OF INTEGRATION STEP IS 10. SECONDS
PRINTING STARTS IN CYCLE 6 AND PRINTS AT INTERVALS OF 6 CYCLES
INITIAL TIME .00 HOURS
SURCHARGE VARIABLES: ITMAX... 30
SURTOL... .050
PRINTED OUTPUT AT THE FOLLOWING 4 JUNCTIONS

10 20 30 40

AND FOR THE FOLLOWING 4 CONDUITS

12 32 24 45

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	PIPE NO. NUMBER	LENGTH (FT)	AREA (SQ FT)	MANNING COEF.	MAX WIDTH (FT)	DEPTH (FT)	JUNCTIONS AT ENDS		INVERT HEIGHT ABOVE JUNCTIONS	
1	12	200.	1.77	.013	1.50	1.50	10	20	.00	1.17
2	32	100.	1.77	.013	1.50	1.50	30	20	.00	1.17
3	24	200.	3.14	.013	2.00	2.00	20	40	.00	.13
4	45	150.	3.14	.013	2.00	2.00	40	50		

	JUNCTION NUMBER	GRELEV ELEV.	CROWN ELEV.	INVERT ELEV.	QINST (CFS)	CONNECTING CONDUITS		
1	10	103.30	101.18	99.68	.00	12		
2	30	101.80	99.68	98.18	.00	32		
3	20	101.30	99.18	96.51	.00	12	32	24
4	40	98.80	96.63	94.50	.00	24	45	
5	50	97.00	93.80	91.80	.00	45		

- FREE OUTFALL DATA -

FREE OUTFLOW AT JUNCTIONS 50
OUTFLOW CONTROL WATER SURFACE ELEVATION IS 100.00 FEET
- SUMMARY OF INITIAL HEADS, FLOWS AND VELOCITIES - - - - - - - - - - - - - -

INITIAL HEADS, FLOWS AND VELOCITIES ARE ZERO

***** JUNCTION HYDROGRAPHS OBTAINED BY SIMPLIFIED RATI ONAL FORMULA *****

JUNCTION NUMBER	TRIANGLE HYDROGRAPH TIME (MIN)/INFLOW (CFS)					
10	.00/	.00	15.00/	3.44	40.05/	.00
30	.00/	.00	17.00/	4.55	45.39/	.00
20	.00/	.00	12.00/	2.13	32.04/	.00
40	.00/	.00	20.00/	6.68	53.40/	.00

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*** * * * * * * * * * * * * * * * T I M E H I S T O R Y O F H. G. L. * * * * * * * ***

(VALUES IN FEET)

TIME	JUNCTION 10 GRND 103.30		JUNCTION 20 GRND 101.30		JUNCTION 30 GRND 101.80		JUNCTION 40 GRND 98.80	
HR . MIN	ELEV	DEPTH	ELEV	DEPTH	ELEV	DEPTH	ELEV	DEPTH
0. 1	99.77	.10	98.03	1.52	98.34	.16	98.80	2.13
0. 2	99.86	.18	101.30	2.67	101.40	1.50	98.80	2.13
0. 3	99.94	.26	98.79	2.28	98.83	.65	98.80	2.13
0. 4	99.96	.29	98.81	2.30	98.64	.46	98.80	2.13
0. 5	100.00	.33	98.98	2.47	99.04	.87	98.80	2.13
0. 6	100.03	.36	98.81	2.30	98.88	.71	98.80	2.13
0. 7	100.06	.39	98.90	2.39	98.93	.76	98.80	2.13
0. 8	100.09	.42	98.94	2.43	98.97	.80	98.80	2.13
0. 9	100.12	.45	98.95	2.45	98.97	.80	98.80	2.13
0.10	100.15	.47	98.99	2.48	99.03	.85	98.80	2.13
0.11	100.17	.50	99.03	2.52	99.07	.90	98.80	2.13
0.12	100.20	.52	99.07	2.56	99.12	.94	98.80	2.13
0.13	100.22	.54	99.09	2.58	99.16	.98	98.80	2.13
0.14	100.24	.57	99.11	2.61	99.19	1.02	98.80	2.13
0.15	100.26	.59	99.14	2.63	99.23	1.06	98.80	2.13
0.16	100.26	.58	99.14	2.63	99.26	1.09	98.80	2.13
0.17	100.24	.57	99.14	2.63	99.27	1.10	98.80	2.13
0.18	100.23	.55	99.10	2.60	99.23	1.05	98.80	2.13
0.19	100.22	.54	99.08	2.57	99.18	1.01	98.80	2.13
0.20	100.20	.53	99.05	2.54	99.15	.97	98.80	2.13

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S U M M A R Y S T A T I S T I C S F O R J U N C T I O N S

JUNCTION NUMBER	GRELEV ELEVATION (FT)	UPPERMOST PIPE CROWN ELEVATION (FT)	MAXIMUM COMPUTED DEPTH (FT)	TIME OF OCCURENCE HR.	MIN.	FEET OF SURCHARGE AT MAX. DEPTH	FEET MAX. DEPTH IS BELOW GRELEV ELEVATION	LENGTH OF SURCHARGE (MIN)
10	103.30	101.18	.59	0	15	.00	3.04	.0
30	101.80	99.68	3.63	0	2	2.13	.00	.5
20	101.30	99.18	4.79	0	2	2.13	.00	.5
40	98.80	96.63	4.30	0	1	2.17	.00	19.5
50	97.00	93.80	8.20	0	0	6.20	.00	20.0

**************** T I M E H I S T O R Y O F F L O W A N D V E L O C I T Y ****************

Q(CFS), VEL(FPS)

TIME	CONDUIT 12		CONDUIT 32		CONDUIT 24		CONDUIT 45	
HR . MIN	FLOW	VEL	FLOW	VEL	FLOW	VEL	FLOW	VEL
0. 1	.08	1.0	.17	1.4	-12.27	-2.7	-23.30	-8.4
0. 2	.00	.3	.35	.0	.00	-.7	-20.29	-6.5
0. 3	.72	.9	1.43	1.4	4.87	1.6	-20.23	-6.4
0. 4	.83	1.0	.79	1.2	.76	.3	-20.23	-6.4
0. 5	1.07	1.1	2.03	1.2	3.41	.9	-20.23	-6.4
0. 6	1.30	1.4	1.92	1.5	4.12	1.4	-20.23	-6.4
0. 7	1.54	1.6	1.56	1.2	4.22	1.3	-20.23	-6.4
0. 8	1.77	1.7	2.07	1.6	5.42	1.7	-20.23	-6.4
0. 9	2.00	1.9	2.36	1.8	5.86	1.8	-20.23	-6.4
0.10	2.23	2.0	2.63	1.9	6.51	2.0	-20.23	-6.4
0.11	2.47	2.2	2.90	2.0	7.21	2.3	-20.23	-6.4
0.12	2.70	2.3	3.15	2.1	7.87	2.5	-20.23	-6.4
0.13	2.93	2.4	3.45	2.3	8.37	2.6	-20.23	-6.4
0.14	3.16	2.6	3.70	2.4	8.73	2.8	-20.23	-6.4
0.15	3.39	2.7	3.97	2.5	9.12	2.9	-20.23	-6.4
0.16	3.34	2.7	4.28	2.7	9.34	3.0	-20.23	-6.4
0.17	3.20	2.6	4.53	2.8	9.33	3.0	-20.23	-6.4
0.18	3.06	2.6	4.46	2.9	9.10	2.9	-20.23	-6.4
0.19	2.93	2.5	4.27	2.8	8.64	2.8	-20.23	-6.4
0.20	2.79	2.4	4.10	2.8	8.24	2.6	-20.23	-6.4

'''''''''''''''''' S U M M A R Y S T A T I S T I C S F O R C O N D U I T S ''''''''''''''''''

CONDUIT NUMBER	DESIGN FLOW (CFS)	DESIGN VELOCITY (FPS)	CONDUIT VERTICAL DEPTH (IN)	MAXIMUM COMPUTED FLOW (CFS)	TIME OF OCCURENCE HR.	TIME OF OCCURENCE MIN.	MAXIMUM COMPUTED VELOCITY (FPS)	TIME OF OCCURENCE HR.	TIME OF OCCURENCE MIN.	RATIO OF MAX. TO DESIGN FLOW	MAXIMUM DEPTH ABOVE INVERT AT CONDUIT ENDS UPSTREAM (FT)	MAXIMUM DEPTH ABOVE INVERT AT CONDUIT ENDS DOWNSTREAM (FT)
12	10.5	5.9	18.0	3.4	0	15	2.7	0	16	.3	.59	3.63
32	7.4	4.2	18.0	4.5	0	17	2.9	0	18	.6	3.63	3.63
24	21.9	7.0	24.0	9.3	0	16	3.0	0	16	.4	4.79	4.17
45	30.3	9.7	24.0	.0	0	0	.0	0	0	.0	4.30	8.20

* * * * * PRESURIZED FLOW SIMULATION ENDED * * * * *

APPENDIX C

APPENDIX C

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PROBLEM #5          FILENAME: EX5.HDA

Commands Read From File example.hda
   10 JOB
   20 SWI 6
   40 PDA .009 18 3 2 2 .002
   70 NEW LATERAL 1-2
   90 PIP 200 67.9 67.5 52.9 52.5
  100 PNC 13 10 5 30 1 0 0 1 0 1.5
  120 NEW T L 3-2
  121 PDA .020 24 3 2 2 .002
  140 PIP 300 68.1 67.5 53.1 52.5
  150 PNC 23 20 5 30 1 1 90 0 0 1.5
  160 REM T L 2-4
  171 PDA .011 30 3 2 2 .002
  190 PIP 500.0 67.5 67.0 52.5 52.0
  200 PNC 34 30 1 40 4 1 0 0 0 0 55.0
  210 REM T L 4-5(OF)
  241 PFA 120 10. 0. 4 3 6 6 3 30 0.05 1
  242 PHJ 10 20 30 40
  243 PFP 13 23 34
  244 HHY 10 20 30
  245 HHD 0. 5. 5. 5.
  246 HHD 0.067 30. 30. 30.
  247 HHD 0.20 5. 5. 5.
  248 HHD 0.50 5. 5. 5.
  249 IQV 5. 2.829 5. 1.592 15.0 3.056
  250 IDY 2.90 2.50 2.99 3.00
  260 END
 END OF RUN.
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***** PRESSURIZED FLOW SIMULATIONS *****

INTEGRATION CYCLES 120
LENGTH OF INTEGRATION STEP IS 10. SECONDS
PRINTING STARTS IN CYCLE 6 AND PRINTS AT INTERVALS OF 6 CYCLES
INITIAL TIME .00 HOURS
SURCHARGE VARIABLES: ITMAX... 30
SURTOL... .050
PRINTED OUTPUT AT THE FOLLOWING 4 JUNCTIONS

10 20 30 40

AND FOR THE FOLLOWING 3 CONDUITS

13 23 34

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	PIPE NO. NUMBER	LENGTH (FT)	AREA (SQ FT)	MANNING COEF.	MAX WIDTH (FT)	DEPTH (FT)	JUNCTIONS AT ENDS	
1	13	200.	1.77	.009	1.50	1.50	10	30
2	23	300.	3.14	.020	2.00	2.00	20	30
3	34	500.	4.91	.011	2.50	2.50	30	40

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	JUNCTION NUMBER	GRELEV ELEV.	CROWN ELEV.	INVERT ELEV.	QINST (CFS)	CONNECTING CONDUITS		
1	10	67.90	54.40	52.90	.00	13		
2	20	68.10	55.10	53.10	.00	23		
3	30	67.50	55.00	52.50	.00	13	23	34
4	40	67.00	54.50	52.00	.00	34		

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------------------------------ FREE OUTFALL DATA ------------------------------

FREE OUTFLOW AT JUNCTIONS 40
OUTFLOW CONTROL WATER SURFACE ELEVATION IS 55.00 FEET

---------------------- SUMMARY OF INITIAL HEADS, FLOWS AND VELOCITIES ----------------------

CONDUIT NO.	FLOW(CFS)	VELOCITY(FPS)	CONDUIT NO.	FLOW(CFS)	VELOCITY(FPS)	CONDUIT NO.	FLOW(CFS)	VELOCITY(FPS)
13	5.0	2.8	23	5.0	1.6	34	15.0	3.1
90004	.0	.0						

----------------------------- SUMMARY OF INITIAL DEPTHS -----------------------------

JUNCTION NO.	DEPTH(FT)	JUNCTION NO.	DEPTH(FT)	JUNCTION NO.	DEPTH(FT)	JUNCTION NO.	DEPTH(FT)
10	2.9	20	2.5	30	3.0	40	3.0

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***** JUNCTION HYDROGRAPHS GIVEN BY USERS *****

JUNCTION NUMBER	TRIANGLE HYDROGRAPH TIME (MIN)/INFLOW (CFS)		
10	.00/ 5.00	4.02/ 30.00	12.00/ 5.00
20	.00/ 5.00	4.02/ 30.00	12.00/ 5.00
30	.00/ 5.00	4.02/ 30.00	12.00/ 5.00

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******************* TIME HISTORY OF H. G. L. ********************

(VALUES IN FEET)

TIME	JUNCTION 10		JUNCTION 20		JUNCTION 30		JUNCTION 40	
	GRND 67.90		GRND 68.10		GRND 67.50		GRND 67.00	
HR . MIN	ELEV	DEPTH	ELEV	DEPTH	ELEV	DEPTH	ELEV	DEPTH
0. 1	59.27	1.50	59.74	2.00	57.99	2.50	55.00	2.50
0. 2	63.81	1.50	65.11	2.00	61.12	2.50	55.00	2.50
0. 3	67.90	1.50	68.10	2.00	63.69	2.50	55.00	2.50
0. 4	67.90	1.50	68.10	2.00	64.39	2.50	55.00	2.50
0. 5	67.90	1.50	68.10	2.00	64.03	2.50	55.00	2.50
0. 6	67.90	1.50	68.10	2.00	63.65	2.50	55.00	2.50
0. 7	66.76	1.50	68.10	2.00	62.95	2.50	55.00	2.50
0. 8	63.55	1.50	65.33	2.00	60.86	2.50	55.00	2.50
0. 9	60.61	1.50	61.83	2.00	58.83	2.50	55.00	2.50
0.10	58.22	1.50	59.00	2.00	57.19	2.50	55.00	2.50
0.11	56.38	1.50	56.80	2.00	55.92	2.50	55.00	2.50
0.12	55.12	1.50	55.31	2.00	55.05	2.50	55.00	2.50
0.13	55.73	1.50	55.86	2.00	55.51	2.50	55.00	2.50
0.14	55.68	1.50	55.81	2.00	55.47	2.50	55.00	2.50
0.15	55.70	1.50	55.83	2.00	55.48	2.50	55.00	2.50
0.16	55.69	1.50	55.82	2.00	55.48	2.50	55.00	2.50
0.17	55.70	1.50	55.83	2.00	55.48	2.50	55.00	2.50
0.18	55.70	1.50	55.83	2.00	55.48	2.50	55.00	2.50
0.19	55.70	1.50	55.83	2.00	55.48	2.50	55.00	2.50
0.20	55.70	1.50	55.83	2.00	55.48	2.50	55.00	2.50

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SUMMARY STATISTICS FOR JUNCTIONS

JUNCTION NUMBER	GRELEV ELEVATION (FT)	UPPERMOST PIPE CROWN ELEVATION (FT)	MAXIMUM COMPUTED DEPTH (FT)	TIME OF OCCURENCE HR.	MIN.	FEET OF SURCHARGE AT MAX. DEPTH	FEET MAX. DEPTH IS BELOW GRELEV ELEVATION	LENGTH OF SURCHARGE (MIN)
10	67.90	54.40	15.00	0	3	13.50	.00	20.0
20	68.10	55.10	15.00	0	3	13.00	.00	20.0
30	67.50	55.00	11.91	0	4	9.41	3.09	20.0
40	67.00	54.50	3.00	0	0	.50	12.00	20.0

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***************** T I M E H I S T O R Y O F F L O W A N D V E L O C I T Y ****************

Q(CFS), VEL(FPS)

TIME	CONDUIT 13		CONDUIT 23		CONDUIT 34	
HR . MIN	FLOW	VEL	FLOW	VEL	FLOW	VEL
0. 1	10.53	5.4	10.53	3.0	31.53	5.8
0. 2	16.67	8.9	16.68	5.0	49.90	9.5
0. 3	21.79	12.2	17.95	5.9	62.83	12.6
0. 4	20.33	11.7	16.43	5.3	66.06	13.4
0. 5	21.00	11.8	17.10	5.4	65.32	13.4
0. 6	22.01	12.4	17.87	5.6	63.97	13.1
0. 7	21.54	12.5	19.00	5.9	62.02	12.8
0. 8	18.11	10.5	18.11	5.9	54.45	11.4
0. 9	14.92	8.7	14.93	4.9	44.88	9.5
0.10	11.74	6.9	11.76	3.9	35.32	7.5
0.11	8.54	5.1	8.55	2.9	25.70	5.6
0.12	5.37	3.3	5.38	1.9	16.08	3.6
0.13	5.01	2.8	5.01	1.6	15.04	3.0
0.14	4.99	2.8	5.00	1.6	14.98	3.1
0.15	5.00	2.8	5.00	1.6	15.01	3.1
0.16	5.00	2.8	5.00	1.6	15.00	3.1
0.17	5.00	2.8	5.00	1.6	15.00	3.1
0.18	5.00	2.8	5.00	1.6	15.00	3.1
0.19	5.00	2.8	5.00	1.6	15.00	3.1
0.20	5.00	2.8	5.00	1.6	15.00	3.1

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' ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' S U M M A R Y S T A T I S T I C S F O R C O N D U I T S ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' ' '

CONDUIT NUMBER	DESIGN FLOW (CFS)	DESIGN VELOCITY (FPS)	CONDUIT VERTICAL DEPTH (IN)	MAXIMUM COMPUTED FLOW (CFS)	TIME OF OCCURENCE HR.	MIN.	MAXIMUM COMPUTED VELOCITY (FPS)	TIME OF OCCURENCE HR.	MIN.	RATIO OF MAX. TO DESIGN FLOW	MAXIMUM DEPTH ABOVE INVERT AT CONDUIT ENDS UPSTREAM (FT)	DOWNSTREAM (FT)
13	6.8	3.8	18.0	22.5	0	7	12.7	0	7	3.3	15.00	11.91
23	6.6	2.1	24.0	19.8	0	7	6.3	0	8	3.0	15.00	11.91
34	15.3	3.1	30.0	66.3	0	4	13.5	0	4	4.3	11.91	3.00

* * * * * PRESURIZED FLOW SIMULATION ENDED * * * * *