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Proceedings of Stormwater and Water Quality Model Users Group Meeting

March 23-24, 1987
Denver, Colorado

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August 1987

PROCEEDINGS OF STORMWATER AND WATER QUALITY MODEL USERS GROUP MEETING

March 23-24, 1987
Denver, Colorado

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DISCLAIMER

FOREWORD

A major function of research and development programs is the effective and expeditious fransfer of technology developed by those programs to the user community. A corollary function is to provide for the continuing exchange of information and ideas between researchers and users, and among the users themselves. The Stormwater and Water Quality Model Users Group, sponsored jointly by the U.S. Environmental Protection Agency and Environment Canada/Ontario Ministry of the Environment, was established to provide such a forum. The group has recently widened its interests to include models other than the Stormwater Management Model and other aspects of modeling water quality in urban and natural waters. This report, a compendium of papers presented at the users group meeting held on March 23-24, 1987, in Denver, CO, is published in the interest of disseminating to a wide audience the work of group members.

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ABSTRACT

This proceedings includes 18 papers on topics related to the development and application of computer-based mathematical models for water quality and quality management. The papers were presented at the semi-annual meeting of the joint US-Canadian Stormwater and Water Quality Model Users Group, held on March 23-24, 1987, in Denver, Colorado.

Several papers deal with recent developments and adaptations of the USEPA SWMM model itself. Its application in a variety of situations is described in a nunber of additional papers. Other models covered include UDSEWER, SEWERCADD, and RAFTS.

A number of papers provide a critical overview of hydrologic models and modeling techniques, and a prediction of future development in stormwater modeling, particularly on microcomputers. Other papers deal more specifically with such topics as tidal flooding, corrective phosphorus removal, wasteload allocations, and spreadsheet cost estimations for drainage design parameters.

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ACKNOWLEDGMENT

The Stormwater and Water Quality Model Users Group appreciates the help of interested members in making arrangements for certain of its meetings. This particular meeting was arranged and organized by Dr. William James of the Univesity of Alabama. It was the Group's first meeting in Colorado, a particularly beautiful area. The meeting was locally sponsored by the Flood and Drainage Control District 69, and local assistance was supplied by Dr. James Guo of the University of Colorado at Denver.

STORM SEWER SYSTEM DESIGN BY UDSEWER MODEL

By

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INTRODUCTION

Storm sewer system is a vital element in preserving an urban storm drainage systems; the design of storm sewers involves surface runoff hydrology and sewer hydraulics including surcharged and open channel flow. Considerable effort has been devoted to the developments of methodology and computer models for storm sewer system design. Despite of the existence of many sophisticated hydrologic techniques for the design of storm sewers, the most commonly used one is still the rational method.

Storm sewer design needs to meet the design requirements while recognizing existing physical constrains such as slopes, depth of cover, utility interferences, etc. Usually a storm sewer design is achieved by a series of trial and error calculations until flow conditions and configurations satisfy all the design requirements and site constraints. This iterative process is time consuming and manpower demanding.

In 1986, the Urban Drainage and Flood Control District in Denver, Colorado sponsored the development of the personal computer software, UDSEWER (1), for the design of storm sewer system. Although UDSEWER is primarily programmed to follow the Urban Storm Drainage Criteria Manual (2) at the District, it does provide the user options of implementing different criteria to override default values.

This paper presents background information of UDSEWER, including its capability, limitations and features. It is believed that the use of UDSEWER can improve the efficiency of storm sewer design.

GENERAL INFORMATION ON UDSEWER

UDSEWER is a computer software developed for the use with the IBM personal computer and compatibles. It is written in compiled BASIC computer language and is menu driven with graphic displays, user interaction on-screen editing and on-line help. It can be run on machines that have a floppy or hard disk drives and can be printed using standard dot matrix printer.

UDSEWER can handle a basin system having up to 100 manholes and up to 100 sewers. For larger basins, the user may input off-site runoff and in effect link basins together. EAch manhole mode can have up to four incoming sewers, but only one outgoing sewer. UDSEWER will check for consistency of input data and for proper connections of the sewer-manhole network.

The rational method is used to estimate the peak runoff and to size the downstream sewer. Although UDSEWER uses open channel flow hydraulics to size or evaluate sewer segments, it will also handle pressure flow in existing sewers that are smaller than required. UDSEWER also calculates surface water profiles throughout the sewer system to estimate the water surface elevation at each manhole. Although the program will size only circular pipes, it will also calculate pressure flow and hydraulic grade lines in existing sewers of rectangular and arch shapes.

Final printout includes hydrology and depths of cover at each manhole and flow conditions for each sewer segment. The latter including flow velocity, surcharged length, possible hydraulic jump, etc. UDSEWER will flag violations of design constraints set forth by the user.

REQUIRED INPUT DATA

There are four required groups of input data:

(1) design constraints

(2) rainfall intensity-duration-frequency

(3) manhole information and its surface hydrology and

(4) sewer hydraulic information

The programs user's manual provides input data summary tables designed for the use with the data editor. The following describe in more detail the specifies of each of the four data groups:

Design Constraints:

Design constraints includes minimum dept of coverage, minimum sewer size, and the range of permissible flow velocities in sewer (3). Design

constraints do not affect hydraulic computations, but only serve as a basis to flag any violations in criteria. The program has a pre-set default values for these constraints; however, the user has an option to override any of them.

Rainfall Intensity - Duration Frequency:

Rainfall intensity-duration-frequency data is needed for the Rational Method. UDSEWER permits the user to provide either a rainfall intensity formula or a rainfall intensity table for durations of 5, 10, 30, 40, 60, and 120 minutes.

The default rainfall intensity formula in UDSEWER takes the format developed for Denver area.

$$i = \frac{A \cdot H1}{(Td + B)^C} \tag{1}$$

in which i - rainfall intensity in inches/hour, H1 = one hour rainfall depth in inches, Td - rainfall duration in minutes, and A, B, and C are empirical constants.

For Denver area, it has been found that A = 28.5, B = 10 and C = 0.786.

When rainfall intensity formulate is not available or does not take the above format, the user may enter the design rainfall intensity-duration table. UDSEWER will use linear interpolation and extrapolation to find the design rainfall intensity for any other rainfall duration. Due to the fact that most rainfall statistics were developed from the data recorded with the shortest time interval of five minutes, UDSEWER therefore uses the intensity of five-minutes for any duration (time of concentration) that is less than five minutes.

Manhole Information and Surface Hydrology:

The input data for each manhole includes the manhole identification number, the identification numbers of incoming and outgoing sewers assigned by the user and the ground elevation of the manhole.

When the local peak runoff is known, the user can input its valuation with the contributing area and runoff coefficient for the design flood. If the user wishes UDSEWER to calculate the flow, the user provides sub-basin area, runoff coefficients for both the five-year flood and design flood, overland flow length and its slope and gutter flow length and its velocity. Program will calculate the time of concentration to the upstream manhole of the basin using the following equations.

$$Tc = to + Tf \tag{2}$$

$$Tf - Lf / (Vf \times 60) \quad (3)$$

$$T_o = \frac{1.8\ (1.1 - C5)\ Lo^{0.5}}{So^{0.33}} \quad (4)$$

in which Tc = time of concentration in minutes. Tf - gutter flow time in minutes, Lf - gutter flow length in feet, Vf = gutter velocity in feet/second, Lo - overland flow length in feet, To = overland flow time in minutes, So - overland flow slope, C5 = five-year overland runoff coefficient.

In the computation of the time of concentration, the program defaults to overland flow length of less than 500 feet for a rural area and 300 feet for an urbanized area. According to the Denver Urban Drainage Design Criteria, the time of concentration of the basin can not be shorter than 5 minutes for an urbanized area and 10 minutes for a rural area. These design criteria have been programed into UDSEWER. The demarcation of urbanization used in UDSEWER is the five-year runoff coefficient, C5. when C5 > = 0.3, it is considered urbanized.

Assuming that the time of concentration is the critical design rainfall duration, UDSEWER uses the rational method to estimate the peak runoff.

$$Qp = C\ i\ A \quad (5)$$

in which Qp = peak runoff in cfs, C - runoff coefficient for design flood, A = drainage area in acres.

As the runoff moves downstream in storm sewers, the times of concentration at each manhole from the different parts of the basin upstream are independently calculated. Often, in practice, the longest time of concentration is used for design rainfall duration. However, it is possible that a highly urbanized subbasin with a shorter time of concentration may generate higher peak runoff than a larger composite rural-urbanized subbasin with a longer time of concentration. Therefore, the program calculates every possible combination of subbasins upstream and use the highest peak runoff to size the immediate downstream sewer dimensions.

Sewer Hydraulic Information:

The user needs to provide sewer identification number, length, slope, Manning's roughness n, shape and upstream crown elevation. For an existing sewer, the user needs to identify the sewer shape and dimensions and UDSEWER will evaluate its capacity. For new sewers, round pipes will be sized for the computed or given peak runoff rates. However, the user may use the option of existing sewer to predetermine the sewer shape.

Manning's equation is employed to compute the required sewer size and UDSEWER will then suggest the next larger commercially available sewer size for design.

When calculating hydraulic grade line, Benoulli energy equation is used to balance the energy between two adjacent cross sections.

$$H1 = H2 + \text{friction loss} + \text{local loss} \tag{7}$$

$$H1 = Y1 + Z1 + (V1^2/2g) \tag{8}$$

in which H1 - Bernoulli sum at section 1, etc., Y1 - flow depth in feet at section 1, etc., Z1 - elevation in feet at section 1, etc., and V1 = cross sectional average velocity in ft/sec at section 1, etc.

The friction loss is computer by the nonuniform open channel flow equation.

$$\text{friction loss} = \left(\frac{n^2 \; v^2 \; R^{(-4/3)}}{2.22} \right) \tag{9}$$

in which n = Manning's roughness, R - hydraulic radius in feet, Ls - sewer length in feet.

The junction loss caused by the turbulence at each manhole is estimated by UDSEWER to be 50% of the difference between the incoming and outgoing velocity heads. The exit loss caused by the downstream surcharge or submergence is assumed to be the entire velocity head at the exit.

CASE STUDY

The layout for the example storm sewer system is shown in Figure 1. The user may utilize the input data forms to prepare the input data and then use the UDSEWER data editor to input and edit data. UDSEWER produces two reports: Report I, as shown in Table 1, tabulates input data and Report II, as shown in Table 2, summarizes flow conditions in each sewer and surface hydrology at each manhole.

You will note in Table 2, that for this example the overland slope is manhole 3 is so flat that the overland flow time is 174 minutes. Instead of taking the longest time for concentration as the design rainfall duration, UDSEWER checks every possible combination of upstream subbasins. As a result, the highest runoff rate at manhole 3 is determined to be the combined runoff contributed from the upstream subbasins at manhole 7 and manhole 5.

In this example, the last sewer, ID number 992, the outlet is fully submerged. As a result the entire sewer 992 and part of sewer 1499 are surcharged.

A comparison between the predictions from UDSEWER and STORM1, developed by the Construction Engineering Research Laboratory, Corp of Engineers (4), is shown in Table 3. Although STORM1 uses Kirpitch formula to compute overland flow time, for this example, both methods showed a good agreement in sizing of storm sewers.

CONCLUSIONS

A personal computer software, UDSEWER, was developed for storm sewer system design. Although UDSEWER follows the Denver Urban Storm Drainage Criteria, it has the flexibility of input of any user defined criteria. UDSEWER is capable of handling multiple basins with each basin having up to 100 manholes and up to 100 sewers. UDSEWER estimates surface hydrology at each manhole and calculates flow conditions for each sewer segment. UDSEWER is supported by the University of Colorado at Denver and Urban Drainage and Flood Control District, Denver, Colorado.

The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

ACKNOWLEDGMENT

The development of UDSEWER model, a personal computer software for storm sewer system design, was sponsored by the Urban Drainage and Flood Control District, Denver, Colorado. The practical application of this software is not, however, limited to the Denver region.

REFERENCES

1. Guo, C.Y. and Urbonas, B, "Storm Sewer System Simulator", the Fourth National Conference on Microcomputer in CIvil engineering held in Orlando, Florida, Nov. 5-7, 1986, PP 312-316

2. "Urban Storm Drainage Criteria Manual", Vol 1, Runoff Section, Urban Drainage and Flood Control District, Denver, Colorado, 1969

3. "Design and Construction of Sanitary and Storm Sewer", American Society of Civil Engineers, New York, 1979.

4. "CESTORM: Storm Sewer Distribution Model"; Department of Army, Construction Engineering Research laboratory, Champaign, Illinois.

TABLE 1 UDSEWER REPORT I: SUMMARY OF INPUT DATA

INPUT DATA FOR STORM SEWER SYSTEM DESIGN

USING UDSEWER-MODEL VERSION 1.1

DEVELOPED
BY
JAMES C. Y. GUO, PHD, PE
DEPARTMENT OF CIVIL ENGINEERING, UNIVERSITY OF COLORADO AT DENVER
IN COOPERATION WITH
URBAN DRAINAGE AND FLOOD CONTROL DISTRICT
DENVER, COLORADO

*** PROJECT TITLE :

CASE STUDY : EXAMPLE ONE

*** DESIGN CONSTRAINTS ARE:

MINIMUM BURIED DEPTH IN FT	= 2
MINIMUM SEWER SIZE IN INCH	= 18
MAXIMUM FLOW VELOCITY IN FPS	= 20
MINIMUM FLOW VELOCITY IN FPS	= 3

*** DESIGN RAINFALL RETURN PERIOD IS 5 YEAR

*** DESIGN RAINFALL DURATION (MIN) VS. RAINFALL INTENSITY (IN/HR)

DURATION	5.00	10.00	20.00	30.00	40.00	60.00	120.00
INTENSITY	4.97	4.30	3.38	2.79	2.37	1.83	1.08

*** SUMMARY OF MANHOLE AND SEWER CONNECTIONS

MANHOLE ID NO.	OUTGOING SEWER ID	INCOMING SEWER ID NUMBERS 1ST SEWER	2ND SEWER	3RD SEWER	4TH SEWER
5.0	53.0	0.0	0.0	0.0	0.0
7.0	73.0	0.0	0.0	0.0	0.0
3.0	314.0	53.0	73.0	0.0	0.0
6.0	610.0	0.0	0.0	0.0	0.0
10.0	1014.0	610.0	0.0	0.0	0.0
14.0	1499.0	1014.0	314.0	0.0	0.0
98.0	992.0	1499.0	0.0	0.0	0.0
2.0	0.0	992.0	0.0	0.0	0.0

*** SUMMARY OF SUBBASIN INFORMATION

STORM SEWER SYSTEM DESIGN: NEW SEWERS AND EXISTING SEWERS

MANHOLE ID NO.	GROUND ELEVATION FT	BASIN AREA ACRE(S)	DESIGN RUNOFF C <1.0	DESIGN RUNOFF CS <1.0	KNOWN PEAK QP CFS
5.00	98.70	3.00	0.70	0.70	0.00
7.00	100.30	3.00	0.70	0.70	0.00
3.00	99.00	3.00	0.70	0.70	0.00
6.00	99.20	3.00	0.70	0.70	0.00
10.00	97.50	3.00	0.70	0.70	0.00
14.00	96.50	3.00	0.70	0.70	0.00
98.00	97.50	3.00	0.70	0.70	0.00
2.00	93.00	3.00	0.70	0.70	0.00

MANHOLE ID NO.	OVERLAND FLOW LNTH FEET	OVERLAND SLOPE %	GUTTER FLOW LNTH FEET	GUTTER VELOCITY FPS
5.00	200.00	0.02	305.00	2.50
7.00	200.00	0.02	305.00	2.50
3.00	200.00	0.00	305.00	2.50
6.00	100.00	0.02	405.00	4.00
10.00	200.00	0.02	305.00	2.50
14.00	200.00	0.02	305.00	2.50
98.00	250.00	0.02	255.00	2.00
2.00	200.00	0.02	305.00	2.50

*** SUMMARY OF SEWER INFORMATION

SEWER ID NO.	LENGTH FT	SLOPE %	UPSTREAM CROWN ELEV (FT)	MANNING'N ROUGHNESS	SHAPE	EXISTING CONDITION DIA(HIGH) (IN) (FT)	WIDTH (IN) (FT)
53.00	450.00	0.41	94.91	0.013	ROUND	0.00	0.00
73.00	600.00	0.47	95.88	0.013	BOX	1.50	1.50
314.00	800.00	0.01	93.06	0.013	ROUND	0.00	0.00
610.00	320.00	0.80	95.68	0.014	ARCH	12.00	24.00
1014.00	700.00	0.02	93.12	0.013	ROUND	0.00	0.00
1499.00	1000.00	0.15	92.98	0.013	ROUND	0.00	0.00
992.00	1210.00	0.02	91.48	0.014	ROUND	0.00	0.00

NOTE: (1) DIMENSION UNITS FOR ROUND AND ARCH SEWER ARE INCHES
DIMENSION UNITS FOR BOX SEWER ARE FEET
(2) WHEN DIMENSION OF SEWER IS NOT GIVEN (=0), PROGRAM WILL SIZE THE SEWER BY A ROUND PIPE

TABLE 2 UDSEWER REPORT II: SUMMARY OF SEWER SYSTEM DESIGN

REPORT OF STORM SEWER SYSTEM DESIGN

USING UDSEWER-MODEL VERSION 1.1
DEVELOPED
BY
JAMES C.Y. GUO, PHD, PE
DEPARTMENT OF CIVIL ENGINEERING, UNIVERSITY OF COLORADO AT DENVER
IN COOPERATION WITH
URBAN DRAINAGE AND FLOOD CONTROL DISTRICT
DENVER, COLORADO

*** PROJECT TITLE :

CASE STUDY : EXAMPLE ONE

*** RETURN PERIOD OF RAINFALL IS 5 YEARS

RAINFALL INTENSITY TABLE IS GIVEN

*** SUMMARY OF SUBBASIN RUNOFF PREDICTIONS

MANHOLE ID NUMBER	SUM OF AREA * C	TIME OF CONCENTRATION OVERLAND To (MIN)	GUTTER Tf (MIN)	BASIN Tc (MIN)	RAIN I INCH/HR	PEAK FLOW CFS
5.00	2.10	37.51	2.03	12.81	4.04	8.49
7.00	2.10	37.51	2.03	12.81	4.04	8.49
3.00	2.10	174.11	2.03	176.14	0.38	0.79
6.00	2.10	26.52	1.69	12.81	4.04	8.49
10.00	2.10	37.51	2.03	39.54	2.39	5.02
14.00	2.10	37.51	2.03	39.54	2.39	5.02
98.00	2.10	41.94	2.13	44.06	2.26	4.75
2.00	2.10	37.51	2.03	39.54	2.39	5.02

FOR RURAL AREA, BASIN TIME OF CONCENTRATION >10 MINUTES
FOR URBAN AREA, BASIN TIME OF CONCENTRATION >5 MINUTES AND
AT THE 1ST DESIGN POINT, TC < (10+TOTAL LENGTH/180) IN MINUTES
WHEN WEIGHTED RUNOFF COEFF >0.30, THE BASIN IS CONSIDERED TO BE URBANIZED

NOTICE: WHEN TO+TF <> TC, IT INDICATES THAT THE ABOVE DESIGN CRITERIA SUPERCEDE COMPUTATIONS

*** SUMMARY OF SEWER HYDRAULICS

SEWER ID NUMBER	MANHOLE NUMBER UPSTREAM ID NO.	DNSTREAM ID NO.	SEWER SHAPE	REQUIRED DIA(HIGH) (IN)	SUGGESTED DIA(HIGH) (IN)	EXISTING DIA(HIGH) (IN)	WIDTH (FT)
53.00	5.00	3.00	ROUND	19.76	21.00	0.00	0.00
73.00	7.00	3.00	BOX	1.20	1.50	1.50	1.50
314.00	3.00	14.00	ROUND	50.17	54.00	0.00	0.00
610.00	6.00	10.00	ARCH	17.58	18.00	12.00	24.00
1014.00	10.00	14.00	ROUND	36.96	42.00	0.00	0.00
1499.00	14.00	98.00	ROUND	37.32	42.00	0.00	0.00
992.00	98.00	2.00	ROUND	59.57	60.00	0.00	0.00

DIMENSION UNITS FOR ROUND AND ARCH SEWER ARE INCHES
DIMENSION UNITS FOR BOX SEWER ARE FEET
REQUIRED DIAMETER = HYDRAULICALLY DETERMINED; SUGGESTED DIAMETER = COMMERCIALLY AVAILABLE
FOR A NEW SEWER, FLOW IS ANALYZED BY THE SUGGESTED SEWER SIZE; OTHERWISE, EXISTING SIZE IS USED

!!! CHECK THE ADEQUACY OF EXISTING SEWER SIZE !!

SEWER ID NUMBER	DESIGN Q CFS	P-FULL Q CFS	DEPTH FEET	FLOW AREA SQ FT	VELOCITY FPS	FROUDE NUMBER	COMMENTS
53.00	8.49	10.02	1.23	1.80	4.70	0.78	V-OK
73.00	8.49	9.12	1.20	1.81	4.70	0.26	V-OK
314.00	16.15	19.72	3.08	11.61	1.39	0.15	V-LOW
610.00	8.49	9.07	1.16	1.46	5.80	0.95	V-OK
1014.00	10.03	14.16	2.14	6.16	1.63	0.21	V-LOW
1499.00	27.58	37.90	2.20	6.37	4.33	0.56	V-OK
992.00	30.65	31.34	4.98	19.62	1.56	0.05	V-LOW

FROUDE NUMBER=0 INDICATES A PRESSURED FLOW OCCURS

SEWER ID NUMBER	SLOPE %	INVERT ELEVATION UPSTREAM (FT)	DNSTREAM (FT)	BURIED DEPTH UPSTREAM (FT)	DNSTREAM (FT)	COMMENTS
53.00	0.41	93.16	91.32	3.79	5.93	OK
73.00	0.47	94.38	91.56	4.42	5.94	OK
314.00	0.01	88.56	88.48	5.94	3.52	OK
610.00	0.80	94.18	91.62	3.52	4.38	OK
1014.00	0.02	89.62	89.48	4.38	3.52	OK
1499.00	0.15	89.48	87.98	3.52	6.02	OK
992.00	0.02	86.48	86.29	6.02	1.71	NO

COMMENTS ARE OK WHEN THE BURIED DEPTH IS GREATER THAN THE REQUIRED BURIED DEPTH OF 2 FEET

*** SUMMARY OF COMBINED TIMES OF CONCENTRATION AT MANHOLES

MANHOLE ID NUMBER	LOCAL BASIN Tc MINUTES	INCOMING SEWER ID	Tc AT UPST MNHL MINUTES	TRAVEL TIME MINUTES	AREA*C	TIME OF CNCNTRATION (MIN)
5.00	12.81				2.10	12.81
7.00	12.81				2.10	12.81
3.00	176.14				2.10	176.14
		53.00	12.81	1.59	2.10	14.40
		73.00	12.81	2.13	2.10	14.93
6.00	12.81				2.10	12.81
10.00	39.54				2.10	39.54
		610.00	12.81	0.92	2.10	13.7[illegible]
14.00	39.54				2.10	39.54
		1014.00	39.54	7.16	4.20	46.70
		314.00	14.93	9.58	6.30	24.51
98.00	44.06				2.10	44.06
		1499.00	46.70	3.85	12.60	50.55
2.00	39.54				2.10	39.54
		992.00	50.55	12.91	14.70	63.46

*** SUMMARY OF HYDRAULICS AT MANHOLES

MANHOLE ID NUMBER	SUM OF AREA * C	RAINFALL DURATION MINUTES	RAINFALL INTENSITY INCH/HR	PREDICTED PEAK FLOW CFS	GROUND ELEVATION FEET	WATER ELEVATION FEET	COMMENTS
5.00	2.10	12.81	4.04	8.49	98.70	94.76	OK
7.00	2.10	12.81	4.04	8.49	100.30	98.73	OK
3.00	6.30	14.93	3.85	16.15	99.00	92.92	OK
6.00	2.10	12.81	4.04	8.49	99.20	95.51	OK
10.00	4.20	39.54	2.39	10.03	97.50	92.95	OK
14.00	12.60	46.70	2.19	27.58	96.50	92.85	OK
98.00	14.70	50.55	2.09	30.65	97.50	92.22	OK
2.00	0.00	0.00	0.00	0.00	93.00	92.00	OK

COMMENTS ARE OK WHEN WATER ELEVATION IS LOWER THAN THE GROUND ELEVATION AT MANHOLE

*** SUMMARY OF HYDRAULIC GRADIENT LINE ALONG SEWERS

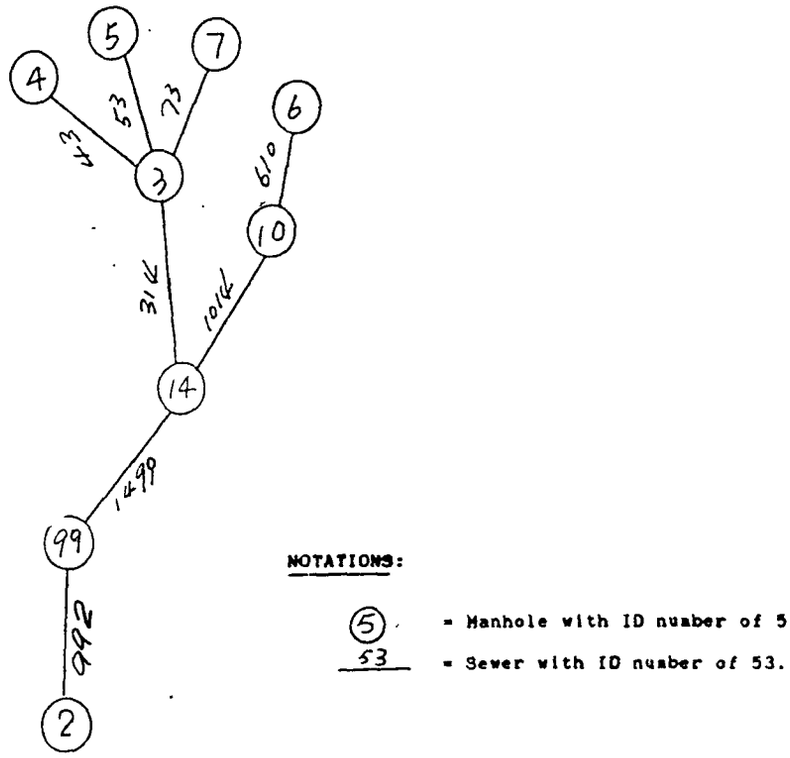
SEWER ID NUMBER	SEWER LENGTH FT	SURCHARGED LENGTH FT	CROWN ELEVATION UPSTREAM FT	DNSTREAM FT	WATER ELEVATION UPSTREAM FT	DNSTREAM FT	FLOW CONDITION
53.00	450.00	0.00	94.91	93.07	94.76	92.92	SUBCR
73.00	600.00	0.00	95.88	93.06	95.73	92.92	SUBCR
314.00	800.00	0.00	93.06	92.98	92.92	92.85	SUBCR
610.00	320.00	0.00	95.68	93.12	95.51	92.95	SUBCR
1014.00	700.00	0.00	93.12	92.98	92.95	92.85	SUBCR
1499.00	1000.00	495.31	92.98	91.48	92.85	92.22	SUBCR
992.00	1210.00	1210.00	91.48	91.29	92.22	92.00	PRES'ED

SUBCR=SUBCRITICAL FLOW; PRSS'ED=PRESSURED FLOW; JUMP=POSSIBLE OCCURENCE OF HYDRAULIC JUMP

TABLE 3 COMPARISON BETWEEN PREDICTIONS FROM UDSEWER AND STORM 1.

	Predicted Peak Runoff in CFS		Pipe Size in Inches			
Sewer Segment	UDSEWER	STORM1	UDSEWER Round	UDSEWER Arch	UDSEWER Box	STORM1 Round
53	8.49	6.42	21			21
73	8.49	6.42			18x18	18
314	16.21	18.28	54			54
610	8.49	6.42		18x24		21
1014	10.03	12.41	42			42
1499	27.58	30.76	42			42
992	32.65	33.89	63			66

FIGURE 1 LAYOUT OF STORM SEWER SYSTEM IN CASE STUDY



MICROCOMPUTERS - THE STORMWATER MODELLING FUTURE

by

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ABSTRACT

The rapid development in recent years of increasingly powerful microcomputers is radically changing the availibility and application of mathematical models. By 1985, former mainframe computer stormwater modes including HEC-1, HEC-2, SWMM, DAMBRK, DAMS2, ILLUDAS, RAFTS, RATHGL and CELLS were available on the IBM PC family of microcomputers. In 1987 an increasing number of stormwater models for the Apple Macintosh family, including HEC-2, SWMM, DAMBRK, RAFTS, RATHGL and CELLS are being released. Clearly, the future of stormwater modelling, its widespread use and the manner in which the profession responds to the challenge of responsibily implementing the power of microcomputers is being governed by the rapid development of new generations of powerful and inexpensive microcomputers.

The historical development of microcomputers is briefly reviewed and the processing power of current microcomputers is presented. The results of three benchmark tests of the SWMM, HEC-2 and RAFTS models are presented and conclusions are drawn. Future developments in the microcomputer field are speculated upon and the implications for stormwater modelling are discussed. It is concluded that the current implementation of stormwater models on microcomputers heralds the future direction of stormwater modelling.

The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not neccessarily reflect the views of the Agency and no official endorsement should be inferred.

INTRODUCTION

Mathematical models are becoming increasingly important and more frequently used in the analysis of complex hydrological processes. With the increase in microprocessor power former mainframe computer stormwater models including HEC-1, HEC-2, SWMM, DAMBRK, DAMS2, ILLUDAS, RAFTS, RATHGL and CELLS are now being routinely executed on microcomputers.

It is no coincidence that the upsurge in interest and usage of mathematical models has been directly linked to the development of mainframe computers and more recently to the development of powerful and inexpensive microcomputers. Such a linkage is demonstrated in the history of the SWMM model. This model was first developed in 1971 to analyse water quantity and quality problems resulting from urban storm water runoff and combined sewer overflows and was available solely as a mainframe computer model. Until recently, it was still perceived that SWMM could only be executed on large computers. This perception is evidenced by the following statement, issued as recently as 1984, on computer needs for SWMM (Huber et al (1)):

> "A large high speed computer is required for operation of the SWMM, such as an IBM370, Amdahl 470, UNIVAC 1108 or CDC 6600..... Through considerable efforts, users have been able to adapt different blocks of the program to various mini computers, but only with extensive use of off-line storage and increase in execution time."

The rapid development of powerful and inexpensive microcomputers from the mid 1970's onwards culminated in the release of a microcomputer version of SWMM3 for IBM PC compatibles (PCSWMM3) in early 1984. The implementaion of SWMM3 on microcomputers has recently been further enhanced by the release of a version of SWMM3 for the Apple Macintosh computers (MACSWMM) which utilizes recent advances in microcomputers including the window environment, mouse control and expanded memory capabilities.

The implementation of the SWMM model on microcomputers is but a single example of the ability of today's microcomputer to implement mathematical models which until recent years were considered to be solely in the domain of mainframe computers. Clearly, the future of stormwater modelling, its widespread use and the manner in which the profession responds to the challenge of responsibily implementing the power of microcomputers is being governed by the rapid development of new generations of powerful and inexpensive microcomputers.

MICROCOMPUTER DEVELOPMENT

The last decade has seen the turbulent growth of the fledgling microcomputer industry into an industry which is impacting on everybody's lives. This growth has been led sometimes by small innovative companies, sometimes by international giants. In 1977 the first true home computers were released by Apple, Commodore, Radio Shack and other companies. Initial sales growth were considered acceptable at the time with, for example, Apple taking 2 1/2 years to sell 50 000 Apple II computers with 4 kbytes (4 kb) of memory. The following year the 5 1/4" floppy disk drive was announced, paving the way for future software development. This was followed

in 1980 by the introduction of the 5 1/4" Winchester hard disk drive, the first large affordable mass storage device.

In August 1981, IBM released the IBM PC and proceeded to sell 50,000 units in 7 months exceeding all expectations of IBM. Within a year seven companies announced IBM "compatible" computers. In 1983 the HP150 Touchscreen Computer and IBM PC-XT were announced to be followed in 1984 by the release of the Apple Macintosh and the IBM PC-AT. In January of 1984 Apple took 74 days to sell 50,000 Macintoshes and in April 1984, in a remarkable marketing drive, sold 50,000 Macintoshes in only 7 1/2 hours. This increase in the rate of sales clearly shows the rapid acceptance of microcomputers at both the personal and corporate levels.

Nearing the end of 1985, networking and multi tasking were begining to redefine the use of microcomputers, a trend which is expected to continue into the 1990's.

Prior to 1983 the microcomputer market was seen to be 75% recreation and 25% business. By 1985 this ratio had been reversed to 80% business and 20% recreation. This trend can be attributed to an intensive campaign originally by IBM and subsequentlyby IBM "compatible" manufacturers to establish the IBM PC as the defacto industry standard for business personal computers. What therefore will prevent IBM from continuing its domination in this field?

There appear to be two factors impeding the growth of the IBM PC market, its hardware and its operating system limitations. The IBM PC computer is based on the Intel 8088 chip which was originally developed from the 8 bit 8080 chip released in 1973. Since the Intel 8088 chip has only an 8 bit bus it must store a 16 bit value in memory in two parts causing it to operate at half the speed of a chip with a 16 bit bus. The adoption by IBM of the 8088 chip in preference to its predecessor the true 16 bit 8086 chip is a decision only IBM can explain. This decision has, however, laid some ground rules for software development. A program cannot be contained within data segments unless both the program and the data segments are in the same 64 kb of memory. Nor can the stack be inserted in a data segment. Hence, at any one time the computer can access only 64 kb of memory; a further limitation is that there can only be 1 Mbyte (1 Mb) of memory on a chip. This memory is in turn further reduced by the 360 kb required when wiring the board, leaving a maximum of 640 kb of user memory.

At first glance a 640 kb memory limitation does not appear to be a hinderance, however, it is just this limitation which is preventing the IBM expansion into the graphics field. A 1200 x 1000 pixel bit-mapped display requires 150 kb of memory, and a grey-scale or colour display can easily require 1.2 Mb or 2.4 Mb. This snares a large slice from an available 640 kb and makes realistic CAD applications impractical. A large display also requires a more powerful processor to refresh the screen, imposing a further limitation on *real* CAD for IBM PC family.

The IBM PC family is also linked integrally with the development of the MS DOS operating system. In late 1980 Microsoft won the contract to supply an operating system for the IBM PC. With the release of the IBM PC only approximately 6 months away and with insufficient time available to develop a proprietry operating system Microsoft purchased SCP-DOS from Seattle Computer Products. This low-powered operating system, which is seemingly based on CP/M, the 8 bit industry standard and which showed no great advantages over Digital Research's CP/M-86, was destined to become MS DOS Version 1.0. The release of MS DOS Version 2.0 offered great improvements including new file access, memory management and a hierachical directory structure. It seemed to draw heavily on UNIX ideas using the same file structure; the development of "pipes" and "redirection" of both input and output and the power of the "shell" iterative system level commands in the form of batch files. However, the original rushed development of MS DOS Version 1.0 and the decision to maintain the compatibility of

Version 2.0 with its predecessor has meant that the MS DOS system has never offered the facilities required to exploit the full capabilities of the screen and serial and parallel ports.

It is now seen that computers based on the Intel 8086 chip are being superseded by computers based on the Intel 80286 chip which may in turn be superseded before operating systems are available to make use of this increased power. Alternatively an extremely powerful and advanced computer based on the Intel 80386 chip may be the answer but the current non-availibility of both application software and an operating system means that its power can not be currently exploited.

In contrast, computers based on the Motorola 68000 chip family have a natural expansion path. This path begins with the Motorola 68000 chip which runs at 0.6 MIPS (8 MHz) and continues on to the to the Motorola 68020 and 68030 chips and the Motorola 68881 and 68882 floating point co-processor chips. By the end of 1987 the RISC (Reduced Instruction Set Computer) technology Motorola 78000 chip, which will run at 20 MIPS (25+ MHz), will be available to further enhance the power of this chip family. Currently there are more than 200 computers available which are based on the Motorola 68000 chip family including Sun, Apollo, Domain, Hewlett Packard, Amiga and the Apple Macintosh.

The Apple Macintosh is an excellent example of the growth of computers based on the Motorola 68000 chip. Its innovative WIMP interface (windows, icons, mouse and pull-down menu), originally developed by Xerox but successfully implemented first by Apple, is now being emulated by workstations costing $100 000 and more, and is even being emulated by IBM with Microsoft Windows. To the scientific user it means that the former 128 kb home computer has expanded into a microcomputer that is ideally suited to the implementation of mainframe software.

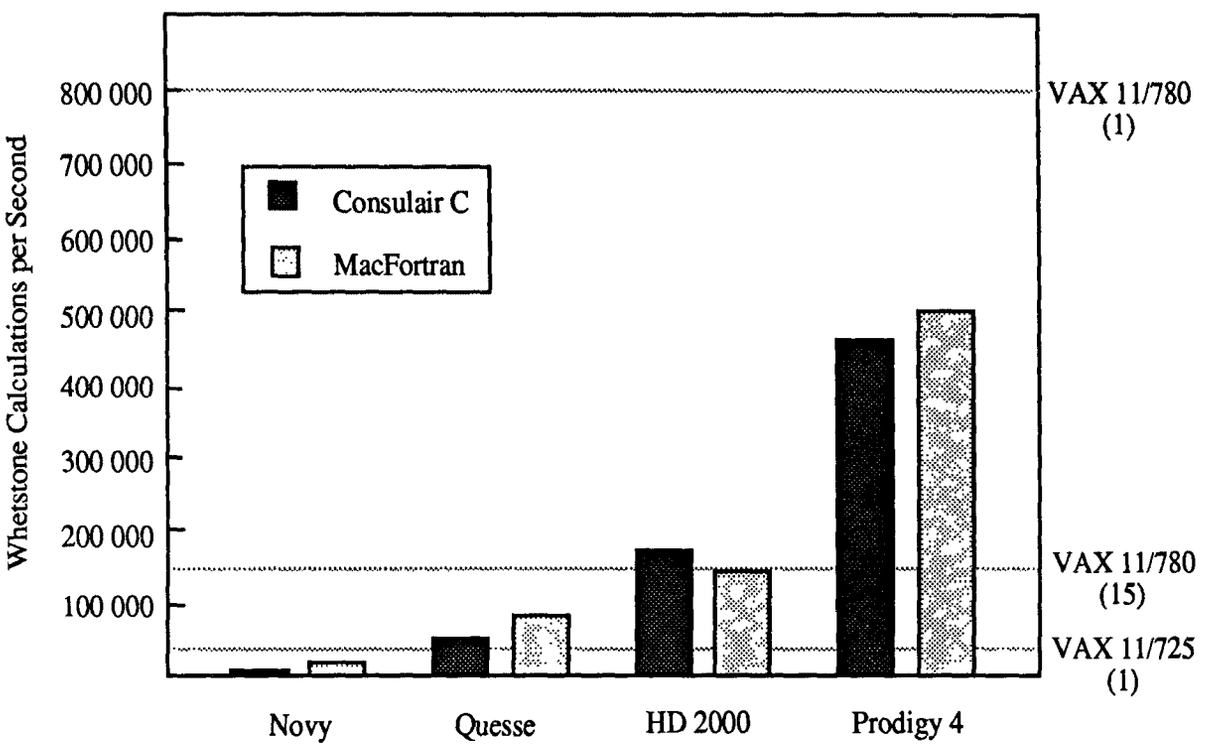

Figure 1 Apple Macintosh Whetstone Benchmark (After Ushijima and Foster (2))

The performance of the microcomputers can be also improved by installing third party upgrades to the point where, for example, the Apple Macintosh can compete with mainframe computers of the power of a DEC VAX 11/780. Ushijima and Foster (2) recently reported in detail on four upgrades currently available for the Apple Macintosh. Of particular interest, though, was the reported comparative performance of the four upgrades with the performance of the VAX 11/780 and the VAX 11/725 computers.

A Whetstone program that employs double precision floating point calculations was one of the tests employed to rate the performance of the various upgrades. The results of the Whetstone benchmark are presented in Figure 1.

It is readily concluded from the results presented in Figure 1 that awesome power is currently available to the Macintosh user and that the availibility of such power will increase as new generations of microcomputers which are based on advanced Motorola chips are released.

STORMWATER MODELLING ON MICROCOMPUTERS

STORMWATER MODELS

The upsurge in interest and usage of mathematical models has been directly linked to the development of mainframe computers and in recent years to the development of powerful microcomputers. By 1985, former mainframe computer stormwater modes including HEC-1, HEC-2, SWMM3, DAMBRK, DAMS2, ILLUDAS, RAFTS, RATHGL and CELLS were available on the IBM PC family of microcomputers. In 1987 an increasing number of stormwater models for the Apple Macintosh family, including HEC-2, SWMM3, DAMBRK, RAFTS, RATHGL and CELLS are being released.

The development of the SWMM model is just one example of this process. The last major revision of the EPA SWMM, SWMM3, was released in 1983 and coincided with the release of the IBM PC. The burgeoning interest in microcomputers and their scientific applications was reflected in the release in 1984 of an adaption of SWMM3 for the IBM PC and compatibles,namely PC SWMM3 (CHI (3)). This version of SWMM represented both a dramatic reduction of the complexity of SWMM3 data entry and a dramatic increase in the availability of the SWMM3 model to users who previously did not have access to mainframe computers. In 1987 PCSWMM3 has been joined by the recently released Apple Macintosh version, namely MACSWMM (CHI (4)).

Software for computers based on the Motorola 68000 chip family is gaining in popularity due to its ability to utilize the innovative features of this chip family to provide features including:

- windows / mouse control
- menu driven applications
- graphics capabilities
- simplicity of operation

A typical example of the graphics capabilities available, for example, to Macintosh users is the graphics output presented in Figure 2. This output presented in Figure 2 is output produced by the Macintosh implementation of the RUNOFF module of SWMM3 (CHI (4)).

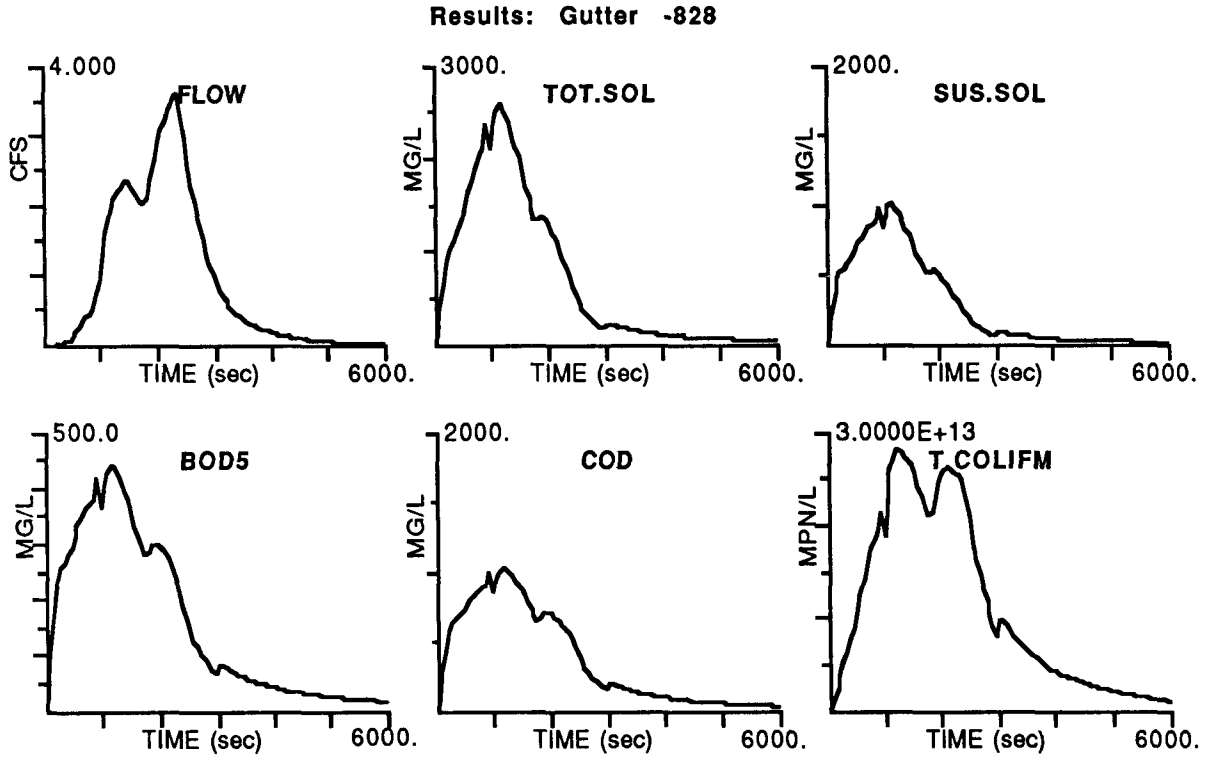

Figure 2 Sample Graphics Output from MACSWMM RUNOFF Module - INSTL3A Data Set

STORMWATER MODEL BENCHMARKS

The performance of three stormwater models, namely SWWM3, HEC-2 and RAFTS has also been investigated on three families of microcomputer. The microcomputer configurations and upgrades tested are listed in Table 1.

TABLE 1 MICROCOMPUTER CONFIGURATIONS AND UPGRADES

CODE	COMPUTER	UPGRADE	PROCESSOR
A1	Apple Mac Plus		Motorola 68000 (8 MHz)
A2	Apple Mac Plus	HD2000	Motorola 68000 (12 MHz)
A3	Apple Mac Plus	HD2000	Motorola 68000 (12 MHz) + Motorola 68881*
O1	Olivetti M24		Intel 8088
O2	Olivetti M24		Intel 8088 + Intel 8087-2*
I1	IBM PC-XT		Intel 8088 + Intel 8087-3*

* The Motorola 68881, Intel 8087-2 and Intel 8087-3 chips are all co-processor chips

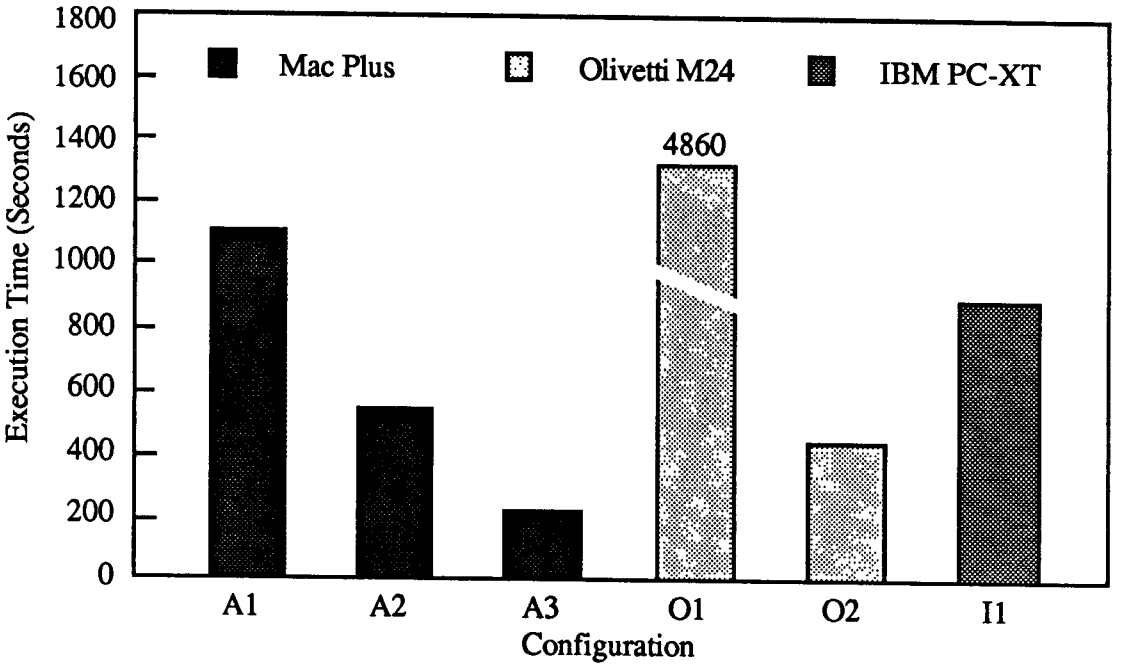

Figure 3 SWMM Benchmark - INSTAL4A Data Set

The first performance test conducted was the execution of verification test data set INSTL4A supplied with PCSWMM3 (CHI (3)). This data set simulates a single event (11/28/73) for Lancaster, Pennsylvania drainage area. The only module executed is the RUNOFF module. The results of the INSTL4A benchmark test is presented in Figure 3.

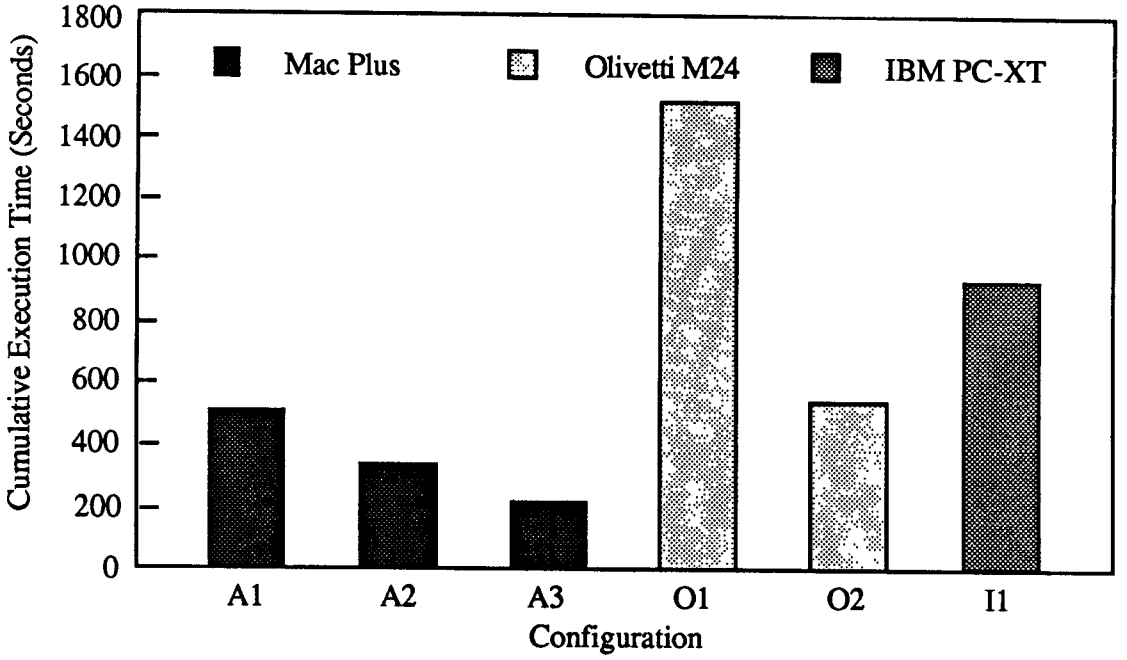

Figure 4 HEC-2 Benchmarks - TEST1, TEST5 and TEST16 Data Sets

The second, third and fourth performance tests conducted were the execution of verification test data set TEST1, TEST5 and TEST16 supplied with HEC-2 (U.S.Army Corps of Engineers (5), (6)). These data set simulate a subcritical flow profile, special and normal bridge with tributary flow and a split flow simulation respectively. The cumulative execution times of the TEST1, TEST5 and TEST16 benchmark tests are presented in Figure 4.

The fifth and sixth performance tesst conducted were the execution of the Wrights Basin and Mogo test data sets supplied with RAFTS (Willing & Partners (7)). These single event data sets simulate a 1000 Yr ARI, 1 hour duration storm for southern Canberra and a 5 Yr ARI, 12 hour duration storm for the southern New South Wales coast respectively. The cumulative execution times of the Wrights Basin and Mogo benchmark tests are presented in Figure 5.

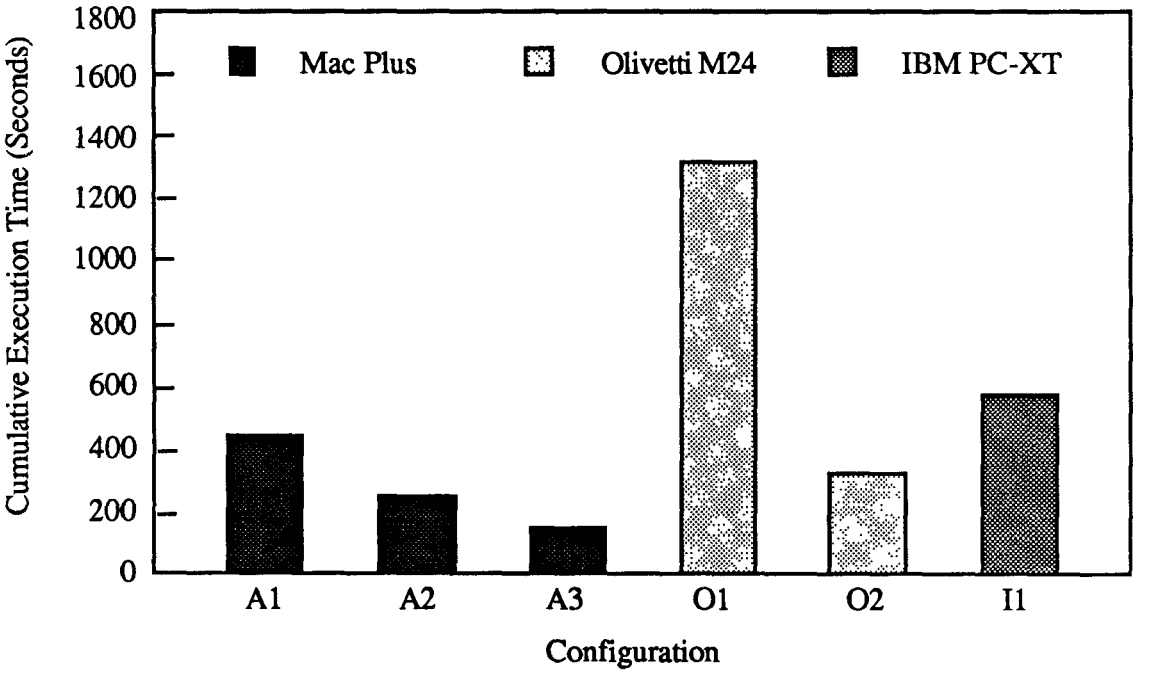

Figure 5 RAFTS Benchmarks - Wrights Basin and Mogo Test Data Sets

The results of the benchmark tests highlight both the practicality of conducting stormwater model simulations on microcomputers and the processing power currently available. The benchmark tests also highlighted the advisability of running stormwater models on a microcomputer fitted with a floating point co-processor. The Apple Macintosh benchmark tests also indicate the dramatic reduction in execution time to be expected as new generations of microcomputers based on the the Motorola 68000 chip family are released in the future.

THE STORMWATER MODELLING FUTURE

The only certainty in the computer industry over the next five or more years is that the breakneck pace of development of the last decade will not slow down. All else is speculation.

It is speculated that desktop computers in the next few years will have a minimum 1 Mb of random access memory (RAM) but more likely will support 8 Mb to 16 Mb of RAM. They will operate at speeds in excess of 10 MIPS and will have *much* more mass storage. They will have much better graphics capabilities with high resolution full page screens and will dump output onto high resolution non impact *quiet* printers. When CD mass storage, voice driven input and optical circuitry can be expected however is a matter for true conjecture.

With the ever increasing cost of human resources and the decreasing cost and increasing power of computers we are already witnessing a shift in emphasis from hardware to software and software support. It is likely that this trend will continue and it will therefore be found that a large investment in software will control the upgrade path followed by computer users.

This upgrade path may be controlled by either maintaining continuously upgradeable chips, which would require all computers to be based on the same chip family, or by establishing a common operating system which could be overlayed on the hardware; such an operating system would need to minimize the number of system dependant calls.

The former alternative is clearly impossible due to the wide range of proprietry hardware currently in existence, however, a likely contender for the latter alternative is the impressive UNIX operating system. Developed in 1969 by A T & T Bell Laboratories there are now at least 74 vendors including Sun, DEC, Hewlett Packard, Apollo, Microsoft (Xenix), Amdahl, Data General, Burroughs/Sperry, NCR, IBM (AIX) and Apple with UNIX operating systems for machines ranging from microcomputers to supercomputers. Of these, more than 50 vendors are complying with the System V Interface Definition (SVID).

The independence of a particular hardware set or vendor means that the investment of a software developer is protected since a multi-vendor standard minimizes risks and ensures the continuity of software sales.

While the carefully nurtured growth of UNIX may not be evident when tracing the chronological release of the software; Version 6 followed by Version 7, then 3.0BSD through 4.2BSD, System III, System V and now System V Release 2, it is well structured, with a consistent and powerful philosophy of control and structure. Originally written for programmers, UNIX utilities for debugging (eg. make, SCCS) provide a productive environment for developing software. For developers, applications written for a standard operating system (e.g. SVID) and ported onto a compliant host only requires the software to be re-compiled and re-linked.

The number of vendors already involved,the special interest groups, trade expositions, journals and most importantly university curricula are encouraging the development of UNIX into the standard operating system for technical computing.

Having established a standard operating system which will ensure a protected investment for software developers what results can be expected?

The electronic paperless office obstinately refuses to arrive and the reams of output accompanying computer programs is largely responsible. Graphics will undoubtedly play a key role in output for the future in both presentation of information for the decision makers and as an aid to the designer. An example of the first step in such a direction is the graphic output from MACSWMM presented in Figure 2. A further step will be the integration of the results of complex numerical models into CAD packages for plotting of construction details It suffices to

say that the full impact of graphics on numerical modelling in the future is yet to be realized either physically or conceptually.

The advent of the microcomputer has already seen the development of the "user friendly" operating system with the Macintosh WIMP (windows, icons, mouse and pull-down menus) interface often being declared to be an industry standard. But how far should this "user friendliness" be transposed into the development of a numerical model? Should the model become an "expert system" capable of being used by anybody ,ie. professional and non-professional alike, or should the software retain some of the intricacies of parameter selection, for example, and require the engineer or modeller to understand the model ?

Ideally, the expert system is the path to follow in the future. However, the cost of resources necessary to develop true artificial intelligence in software is immense and anything less runs the risk of still obeying the axiom - "garbage in / garbage out".

CONCLUSIONS

The advent of increasingly powerful microcomputers in the 1980's is inexorably changing the method in which scientific calculations and mathematical modelling are undertaken. The SWMM3, HEC-2, DAMBRK, RAFTS and CELLS models are just a few examples of the downloading of former mainframe models into microcomputers in recent years. The implementation of these and other models on microcomputers heralds the start of a new future for stormwater modelling.

It is also evident that the impressive power of current microcomputers will be dwarfed by the power of tomorrow's microcomputers. It is expected that furure developments in the stormwater modelling field will utilize the increasing power of microcomputers to improve the scientific basis of the models, improve the user friendliness for both input and output and will integrate current models with sophisticated support packages.

ACKNOWLEDGEMENT

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The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not neccessarily reflect the views of the Agency and no official endorsement should be inferred.

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ENHANCING SWMM3 FOR COMBINED SANITARY SEWERS

by

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ABSTRACT

Operating authorities responsible for the design and maintenance of sewer systems with problems such as a large percentage of storm water, or frequent recurrence of basement flooding during major storm events, or by-passes of excess flows into lakes and rivers, have identified a need for a forecasting method. The method should compute the various components of flow entering combined sanitary sewers.

Most computational models account for flow sources entering a sanitary sewer by making coarse approximations of several flow sources in one aggregate flow calculation. This paper examines procedures for forecasting the major components of inflow and infiltration from surface and groundwater sources as well as sanitary flows from residential, commerical and industrial sources.

The USEPA Stormwater Management Model (SWMM), in particular the version of SWMM adapted for microcomputers (PCSWMM), was evaluated for forecasting combined sewer flows. Two sewersheds in Oakville and Burlington in Ontario were used for this assessment. New algorithms for the additional forecasting procedures are suggested.

INTRODUCTION

The Water Pollution Control Federation 1970 (1) defines wastewater as:

> "A combination of liquid and water-carried wastes from residences, commercial buildings, industrial plants and institutions together with any groundwater, surface water, and storm water that may be present."

Sewers carrying this type of flow are generally categorized as combined storm sewers. The use of a single sewer to carry both storm and sanitary flows is no longer allowed, for environmental and health reasons, although it is by far the least expensive of all the urban servicing methods. Analysis of this type of sewer can be easily accommodated through the use of hydrologic models such as the USEPA Stormwater Management Model (SWMM3) (Huber et al. 1981 (2)).

It was a common method in many municipalities during the post war era of the 1950's and 1960's to allow builders to service new subdivisions with sanitary sewers only. The sewers were typically 200 mm to 300 mm diameter and designed to carry domestic wastes and a small amount of groundwater infiltration.

Builders were allowed to connect foundation weeper pipes and, in some cases, roof drainage pipes directly into the sanitary sewer. Storm drainage was provided by gutters, roadside ditches, storm sewers and drainage swales. In the interests of minimizing construction costs, the storm sewers were generally constructed at a minimum frost protection depth and too shallow to intercept the basement weeper flows, which thus continued to outlet into the sanitary sewers. Pipe deterioration further aggravated the problem by allowing groundwater to enter the sanitary sewer system through pipe joints and broken or cracked pipes. Figure 1 illustrates a sewer system with house connections typical of this servicing method.

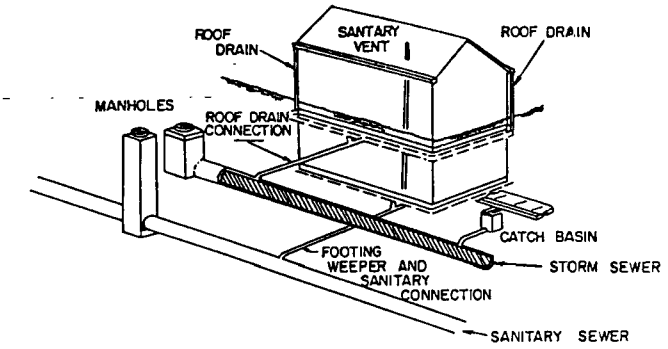
Metcalf and Eddy (1979 (3) p.24) define the sources of stormwater which enter a sanitary sewer as inflow/infiltration:

> "*Infiltration*:: Water entering a sewer system, including sewer service connections from the ground, through such means as, but not limited to, defective pipes, pipe joints, connections, or manhole walls. Infiltration does not include and is distinguished from inflow."

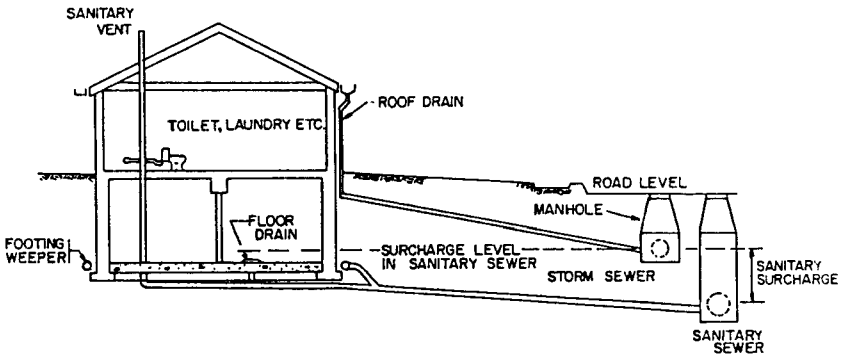
> "*Inflow:* Water discharged into a sanitary sewer system, including service connections, from such sources as, but not limited to, roof leaders, cellars, yards, and area drains, foundation drains, cooling water discharges, drains from springs and swampy areas, manhole covers, cross connections from storm sewers and combined sewers, catchbasins, storm waters, surface runoff, street wash waters, or drainage. Inflow does not include, and is distinguished from, infiltration."

The type of sanitary sewer which is experiencing a high inflow/infiltration component can be defined as a *combined sanitary sewer,* as distinct from a *combined storm sewer*. Figure 1

Figure 1



TYPICAL CONNECTIONS TO STORM AND SANITARY SEWERS



BASEMENT FLOODING FROM INFLOW/INFILTRATION INTO SANITARY SEWERS

illustrates a basement flooding condition which is one consequence of a combined sanitary sewer.

In recent years, with the passing of the U.S. Water Pollution Control Act amendments (1972) and the Fisheries Act and Environmental Contaminants Act in Canada, municipalities are
required to separate the wastewater generated from urban development into two separate systems:
one system to carry sanitary waste and a second system to carry storm runoff. The Water Pollution Control Federation (1972 (4)) defines sanitary sewers and storm sewers as follows:

> "*Sanitary sewer* - a sewer that carries liquid and water-carried wastes from residences, commercial buildings, industrial plants and institutions together with minor quantities of storm, surface and groundwaters that are not admitted intentionally."

> "*Storm sewer* - a sewer that carries storm water and surface water, street wash and other wash waters or drainage, but excludes domestic wastewater and industrial wastes."

Virtually all new sewer construction carried out in municipalities today requires sanitary and storm flows to be collected in separate systems.

Thus we now distinguish between combined sewer systems designed primarily for stormwater (combined storm sewers) and combined sanitary systems designed primarily for sanitary flows (combined sanitary sewers).

Combined storm sewers can be readily analyzed using readily available storm sewer design or analysis techniques. There is, however, a need for a method to forecast flows in combined sanitary sewers with high inflow and infiltration. This paper reviews methods for accounting for the various flow components which enter a combined sanitary sewer.

SCOPE OF THE PROBLEM

A recent court decision on a pump station discharge in British Columbia highlights the concern over combined sewer discharges in lakes or rivers. The events surrounding the discharge related to a pump station malfunction for a period of approximately 20 minutes during which sewage was discharged into an adjacent creek. The District of North Vancouver was charged *and fined* under The Fisheries Act for depositing a deleterious substance into an adjacent water course through an emergency overflow. The overflow had been designed and approved as part of the drainage system (Consulting Engineers of British Columbia, 1983 (5)).

Most municipalities have overflows designed into their systems at various low points to prevent basement flooding during rainfall events. The Court decision to penalize the operating authority, in this case the District of North Vancouver, may have wider implications for approving authorities and the consulting engineering industry.

Environmental concerns relating to combined sewer overflows have been well documented by various Government authorities and court decisions (Patinskas 1983 (6)). The impact which excessive combined sewer flows have on homeowners and operating authorities has not been as well recorded. In 1984 a review of treatment costs relating to combined sewer flows was carried out for six treatment plants in southern Ontario serving approximately 2,000 hectares of urban development with a population of approximately 270,000 (Regional

Municipality of Halton Annual Report, 1984 (7)). Using plant flow information, an approximate annual volume of inflow and infiltration treated at 6 plants was determined. Groundwater inflow and infiltration was estimated by comparing plant flow records for a low groundwater season (such as mid January) to that of a high groundwater season (such as mid March). To reduce the effect of contributions from homeowners and industries, the plant flow record used in the comparison was the lowest point of recording, which occurs at approximately 2:30 a.m. This comparison provides an estimate of infiltration volumes in the sanitary sewer system from groundwater conditions.

To estimate inflow from downspouts, window wells, and catchbasins connected directly
to the sanitary sewer, a comparison was made between a dry summer daily flow rate and a storm event flow rate for the same time of year. The comparison between the instantaneous flow rates occurring immediately after the storm event gives an indication of the amount of direct inflow from impervious areas to the sanitary system.

Estimating inflow and infiltration in this way for each of six treatment plants, it was determined that an annual cost of approximately $400,000 per year is being expended on treatment of groundwater and surface water inflow (approximately $1.50 per person per year). In addition to the direct treatment cost , there are other capital cost losses which also affect the true cost of groundwater and stormwater inflow and infiltration. These relate to the loss of treatment plant capacity and sewer system capacity. The loss in capacity is more difficult to assess and no doubt would exceed the operating loss estimated above.

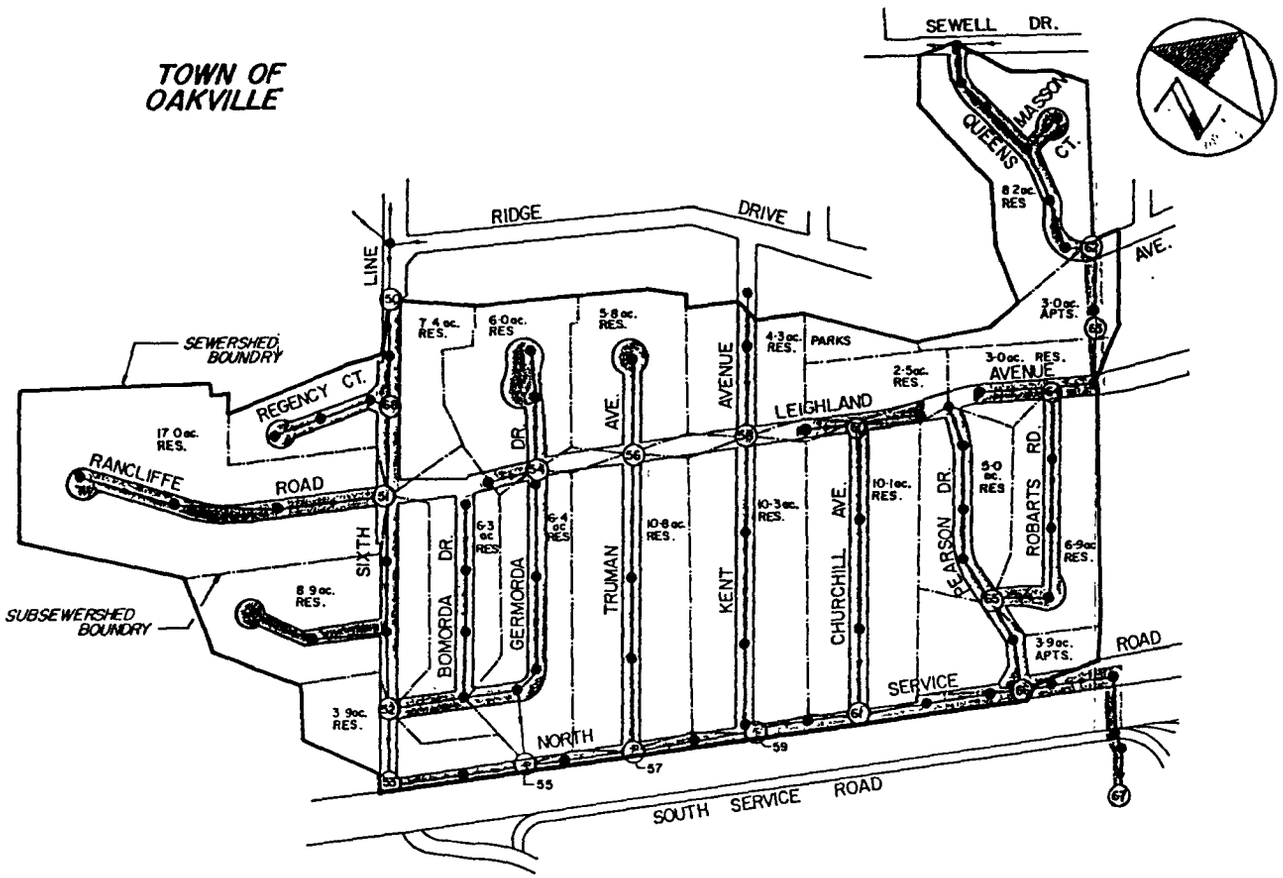
As stated earlier the homeowner (or user of the system) also suffers through frequent surcharging of the sanitary line which often results in flooded basement floor drains, also illustrated in Figure 1. The resulting damage to household furnishings and, on occasion, structural damage to floor slabs, results in numerous insurance claims. These costs are reflected in litigation costs and higher premiums.

APPLICATION OF PCSWMM3

PCSWMM was applied to two sewersheds, A and B, which are considered typical of most sewersheds with combined sanitary sewer systems. The computed results were then compared to flows recorded by American Digital Systems (ADS, 1985 (8)). Eighteen subdrainage areas were monitored in the City of Burlington Maple Drainage Area and 32 subdrainage areas were monitored in the Town of Oakville, South West Drainage Area. In addition to flow data, American Digital Systems Inc. collected rainfall data. The purpose for the data collection was to establish comparative flow results between the various drainage areas and to pinpoint those areas which were displaying high inflow during rainfall events. The two typical sites were selected from the data to represent the high goundwater infiltration condition (Site A) and the high pipe inflow condition (Site B).

DESCRIPTION OF SEWERSHED A

Sewershed A comprises 126 acres of residential development including single family homes and a small percentage of low density apartments as shown in Figure 2. The area was serviced in the mid 1950's with vitrified clay pipe material. The condition of the sewer pipes is known from photographs taken on in-line camera . The photos indicate severe cracking around



SITE 'A'
LANDUSE AND SEWERSHED BOUNDRIES

Figure 2

the crown of pipe and frequent displacement of joints between pipes. Groundwater infiltration has been occurring for some time as evidenced by calcium build-up at the joints, and root infiltration.

The as-constructed drawings contained in the Halton Region record system indicate that most of the sewers throughout Sewershed A were excavated into a rock trench. At the time of the initial servicing, only a sanitary sewer was installed; storm sewers were not constructed and storm drainage was provided by roadside ditches in most areas. Limited storm sewers were constructed as a means of draining the roadside ditches. Comments from the residents in this area indicate a high groundwater table, particularly in the spring season. The soil conditions in the area indicate a 3-6 foot layer of clay soil over limestone rock. A very early topographic map of this area (1920) designates Sewershed A area a swamp or marsh. The high rock profile and poor drainage of the subsoil material results in a high groundwater condition around most basements and sewers.

DESCRIPTION OF SEWERSHED B

Site B comprises 97 acres of residential and semi-detached homes as shown in Figure 3. The area was serviced between 1972 and 1975. Asbestos cement and clay pipe material were used for the main sewer and house service connections. At the time of servicing, both storm sewers and sanitary sewers were installed along with a curb and gutter urban class roadway. The soil type typical of this area is a clay till classification. The water table elevation particularly in the spring season rises to the elevation of the sanitary sewer and surrounding basements. This high watertable condition is of short duration and remains below the sewer and basement elevation for the remainder of the year.

Site B is subjected to a high sanitary inflow condition during rainfall events. A more detailed investigation was undertaken by the Halton Region engineering staff to indentify the source of the stormwater inflows. With the co-operation of adjacent residents, a program of smoke testing and dye testing was undertaken. The testing program revealed 13 homes that had at least one roof downspout connected directly to the sanitary sewer. In addition, one double street catchbasin, a driveway catchbasin, and a hydro transformer vault were also found to be directly connected.

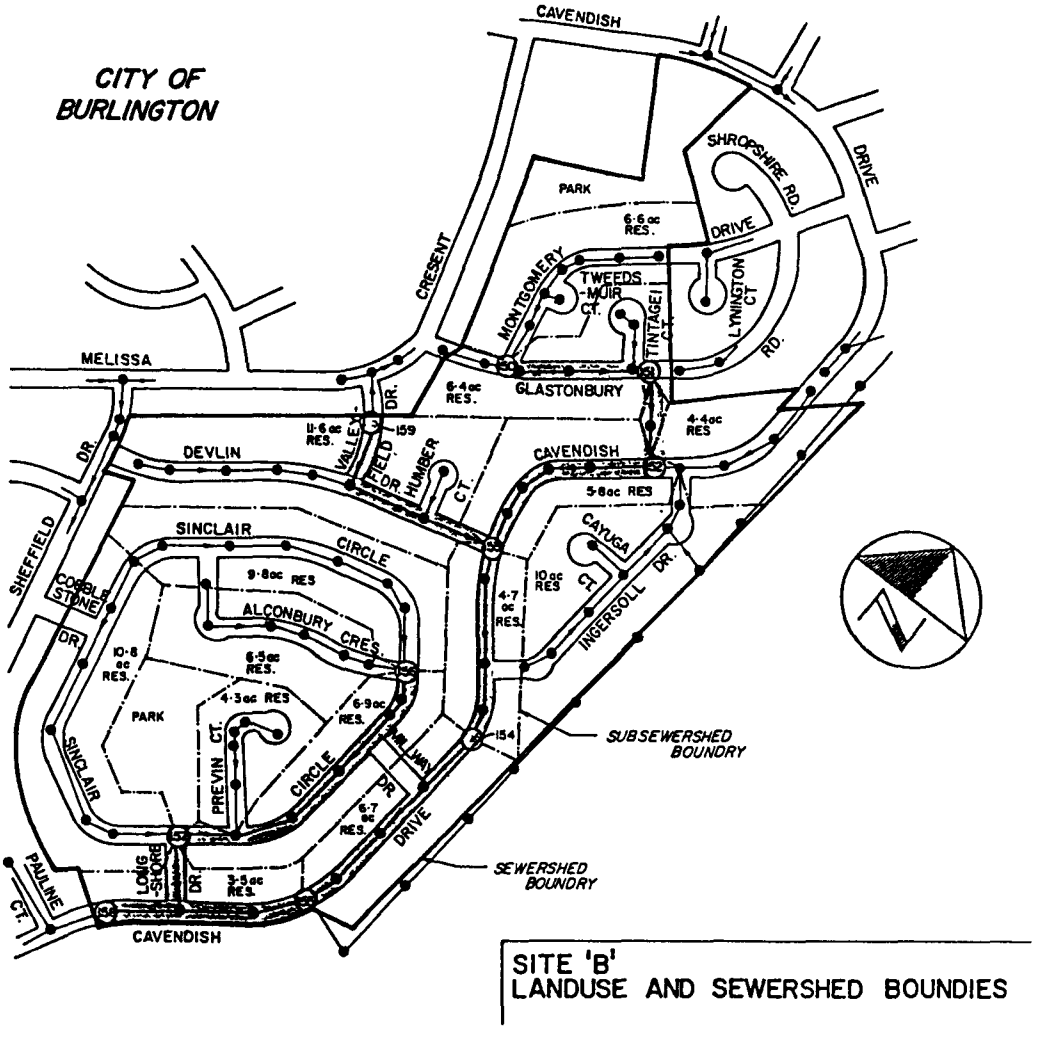
DATA COLLECTION FOR SITE A AND SITE B

Pipe length, size and manhole data for Site A and Site B was collected from as-built records provided by the Regional Municipality of Halton. Since Site A was constructed much earlier than Site B, the accuracy of records and data is not as complete. Much of the data for Site A was taken from a sewer network or operating map which has been kept up to date, and the accuracy of these maps is adequate for the input requirements of PCSWMM.

The as-constructed plan and profile drawings for Site B were used to confirm pipe lengths, pipe sizes and manhole information. In addition, the as-built drawings for Site B were used for determining runoff areas for roadway and driveway catchbasin areas and transformer vault drainage.

Information on the number of homes within each sub-sewershed was obtained from street index maps provided by the local area municipalities, Oakville and Burlington. Data on house values, population information and statistics relating to the market value of homes, percentage garbage grinders and average family income was obtained from the Planning Department staff at the Regional Municipality of Halton. Information on water billing rates and

Figure 3


CITY OF BURLINGTON
CAVENDISH
SHROPSHIRE RD.
DRIVE
PARK
6·6ac RES.
DRIVE
MONTGOMERY
TWEEDS-MUIR CT.
TINTAGEL CT.
LYNNINGTON CT
RD.
CRESENT
MELISSA
GLASTONBURY
6·4ac RES.
4·4ac RES
11·6ac RES.
159
VALLEY FIELD DR.
DEVLIN
HUMBER CT.
CAVENDISH
5·8ac RES
SHEFFIELD DR.
SINCLAIR CIRCLE
9·8ac RES
ALCONBURY CRES.
CAYUGA CT.
10ac RES
4·7 ac RES.
INGERSOLL DR.
COBBLE STONE DR.
10·8 ac RES.
6·5ac RES.
4·3ac RES
6·9ac RES.
PARK
SINCLAIR
PREVIN CT.
CIRCLE
154
SUBSEWERSHED BOUNDRY
DR
DRIVE
6·7 ac RES.
SEWERSHED BOUNDRY
PAULINE CT.
Lodg-Store
DR
3·5ac RES.
154
CAVENDISH
SITE 'B'
LANDUSE AND SEWERSHED BOUNDIES

water consumption rates was obtained from the Finance Department for the Region of Halton. Data relating to sewer infiltration for the sewershed was also input to the PSWMM model.

The computed flows from PCSWMM were compared with flows recorded by on-site monitoring equipment located at the outlet manhole locations for Sites A and B. The monitoring equipment installed in the manholes comprised a pressure transducer located inside the first length of pipe upstream from a pre-selected manhole site. The manholes selected for the monitoring equipment were field-checked to confirm that the sensing equipment would be free from flow turbulence due to a poorly benched manhole. The sensor detected the depth of water flowing within the pipe and transmitted an analogue voltage signal to a small micro-computer monitor hung under the manhole cover. The analogue signal was converted to a digital signal and stored in a data collection system.

An ultra-sound velocity sensor was also provided at each monitoring site. The depth and measured velocity were recorded at 15 minute time steps over the study period. Tipping bucket rainfall information was also recorded at 15 minute intervals. The flow recording equipment was closely monitored by field personnel. Each monitoring site was calibrated and velocity checks were carried out to confirm the accuracy of the data being recorded.

COMPUTATION METHOD USING PCSWMM

Site A was chosen to represent a high groundwater infiltration condition. A three day simulation period was selected, extending from 0:00 hour on September 18, to 0:00 hour on September 21. This was a mid week time period and the observed flows display a uniform diurnal pattern. Because the simulation involved infiltration due to groundwater, surface water inflow from rainfall was not monitored. Submodels FILTH and INFIL within the Transport Module were used in the simulation. The computed sewage flow was routed through the pipe system to outlet manhole 67. An input hydrograph accounting for all upstream flows was input at manhole 50. The dry weather sewage flow was computed in the FILTH submodel and input at various manholes throughout the system.

A value for infiltration was selected for the INFIL submodel and a proportional amount was entered at the upstream manholes of each subsewershed by the INFIL routine. Based on an "old sewer" criteria, the infiltration rate for the entire Site A sewershed was calculated to be 0.85 cfs.

For computing dry weather flows the FILTH model allows the use of water consumption rates for computing dry weather flows or population statistics and land use information. Simulations were carried out using both alternative methods and the results are discussed below.

Site B was chosen as an example of a sanitary sewershed with surface inflow and sub-surface infiltration. Input hydrographs were computed for each of the surface runoff areas within the Runoff Module. These hydrographs were input at four manhole locations, 159, 153, 152 and 157. The storm hydrograph which was input at manhole 159 was combined with a sewage hydrograph from the upstream sewage drainage areas. These hydrographs as well as the dry weather flow computed for the various sub-drainage areas were routed through the pipe system to the outlet point at manhole 158. A "new sewer" condition infiltration of 0.32 cfs was input based on an average infiltration rate of 20 cubic metres per hectare per day, typical of a new sewer infiltration rate.

An average sewage flow rate of 90 gallons per capita per day was used for predicting the dry weather flows for both Sites A and B. The computed and observed flows are presented and discussed below.

SENSITIVITY ANALYSIS

To improve on the predictions, infiltration values were optimized, based on observed flows during a 2:00 a.m. to 4:00 a.m. time period for each site. Flow rates in the early morning hours have a minimum domestic sewage component and therefore provide a more accurate value for groundwater infiltration.With the new infiltration values (Site A =0 .31 cfs and Site B = 0 .13 cfs), a significant improvement in the values computed by PCSWMM was obtained.

Additional computational runs were made for Site A using water meter records to forecast the dry weather flow. Also, the population data for Site A was used as a third means of predicting the sewage flows. As shown in Figure 4, the actual water meter record provides a slightly more accurate prediction when compared to observed flows than do the other two methods available to the user.

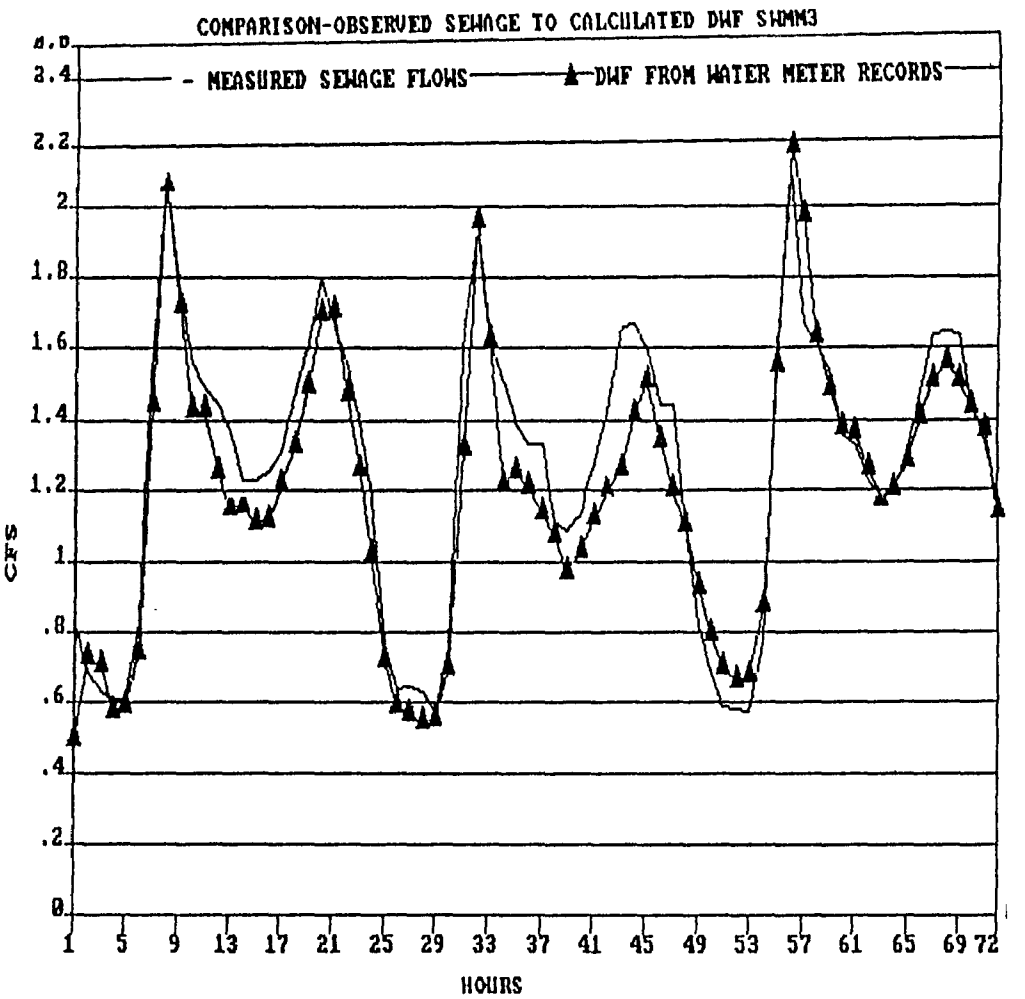
Additional predictions were made for Site A using varying input parameters . The objective was to examine the sensitivity of each variable and thereby determine which parameters must be selected with care and accuracy. The results are summarized in Table 1.

Table 1
SITE A - Sensitivity Comparison
Input variables INFIL and FILTH - PCSWMM

Variable	Output Sensitivy to Variable Change		
	High	Medium	Low
Infiltration	*		
House Prices			*
Population Density		*	
Persons Per Dwelling			*
% Garbage Grinders			*
Family Income			*
Diurnal Variation			*

From the computed results and sensitivity analysis , it can be concluded that the largest single variable in the prediction of combined sanitary sewer flow is infiltration. In the present version of PCSWMM, the user must select this value using his best judgment. There is no method to predict the groundwater flow infiltration component for a combined sewer model. All other variables contained in the FILTH algorithm are predictable and reasonable estimates can be obtained through the various data sources within municipalities or operating authority records.

Figure 4



SITE A

PROPOSED IMPROVEMENTS

GENERAL

The various sources of flow entering a combined sanitary sewer have been categorized as follows:

a) sanitary sewage from domestic, commercial and industrial sources,
b) subsurface inflow and infiltration,
c) surface inflow.

Stormwater models such as PCSWMM3 are designed to compute surface runoff and can be used to predict the storm water component of a combined sanitary sewer (referred to under (c) above). Several changes and additions are now suggested for SWMM such that flows can be determined for the sanitary sewer component described in (a) and inflow and infiltration flow component described in (b). The sources of flow can be separated as shown in Figure 5.

For stormwater modelling, the design engineer usually first discretizes the drainage area into various sub-catchment areas whose surface characteristics can be meaningfully estimated. A similar approach should be taken to discretizing the sanitary flow catchment areas and subsurface infiltration/inflow sewersheds. All significant sources of inflow and infiltration must be accounted for. The three primary categories are:

a) Inflow from direct surface runoff sources,
b) Infiltration from existing groundwater sources,
c) Infiltration from surface water which percolates through the soil to the pipe system.

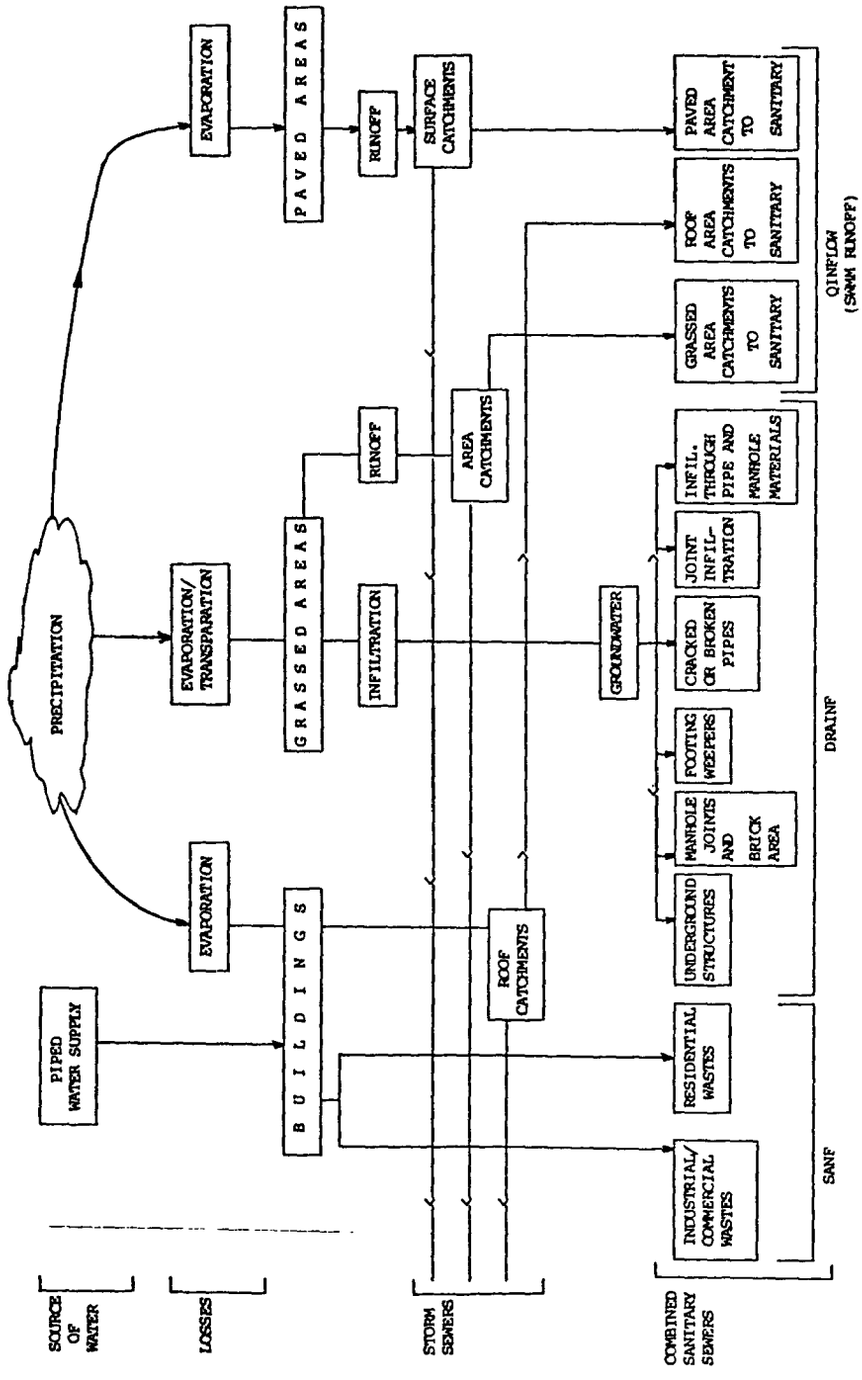
All significant sources of flow in a combined sanitary sewer should be disaggregated and predicted separately. This will allow the user to predict each flow component and further to examine alternatives for a sanitary sewer with high inflow and infiltration. The user will have a greater ability to manage combined sanitary sewer flows and the varied complex solutions which may be required.

The model must emphasize the need to account for all sources of flow entering the sanitary sewer. In our approach here, no attempt is made to forecast these flows through the use of the full St. Venant equations. Instead, simple mathematical expressions are used to account individually for each source.

A continuous computation of groundwater storage is necessary to compute variations in the depth of groundwater above or below the pipe system. The model should include site-specific characteristics such as soil conditions, bedrock elevation, and surface infiltration. The determination of surface inflow and subsurface inflow/infiltration should be based on different discretized drainage areas. The sanitary flow calculation should be based on discretization of residential, commercial and industrial land use areas. The modeler will, therefore, be required to discretize three sets of areas:

1) sanitary sewershed areas,
2) surface runoff catchment areas directly entering the sanitary sewer (inflow),
3) subsurface catchment areas entering the sanitary sewer (infiltration).

Figure 5
Flows Included in Prediction Modules SANF, DRAINF, Qinflow



The user must therefore collect data on surface and subsurface characteristics as well as land use information. All sources of flow in the upper sewershed should be determined in the RUNOFF Module of any storm water model. The flows and solution alternatives in the larger diameter sewer network can be analyzed in the TRANSPORT Module.

The prediction methods for sanitary flow and inflow/infiltration suggested here use variables for which field data is available or readily obtained.

SANITARY COMPONENT (SANF)

A computational algorithm SANF is presented in Figure 6. The required data and steps to be followed when using SANF are presented in Figure 7.

The land-use area (a_i) used as the basis for computing sewage flow is readily available from aerial photography or land use maps. The land use areas are multiplied by a population density (Pi) in the case of residential area, and a population density equivalent in the case of commerical and industrial areas, to determine the net population (P) for the total study area. The net population figure is multiplied by an average daily per capita flow value (q), to arrive at the average daily sewage flow for the drainage area. Because of fluctuations in daily and weekly flow patterns, a peaking factor (M) must be applied to the average daily flow to estimate a peak sewage flow (Q_s).

Typical factors for population densities for various land use types are given in Table 2 (Halton 1985 (8)).

TABLE 2
POPULATION DENSITIES FOR VARIOUS TYPES OF LAND USE

	Land Use Type	Persons Per ha
1	Single Family	55
2	Semi-Detached	100
3	Multi-Family (row housing)	135
4	Apartments (over 6 stories)	285
5	Light Commerical	90
6	Light Industrial	125

The average daily per capita sewage flow (q) exclusive of infiltration and inflow, ranges from 225 to 450 litres per capita per day (M.O.E. 1984 (9)). A typical value used for a southern Ontario community is 275 litres per capita per day (Moore 1985 (10)). This value, determined from water use records, provides a best estimate for water entering the sewage system, exclusive of water used for fire fighting, lawn watering and pipe loss.

If peak flow information from industrial and commerical sources is readily available, the equivalent area calculation for these land uses may be omitted and the flows added directly to the residential flow component to arrive at the peak flow for the drainage area.

The peaking factor (M) is the ratio of maximum to average daily sewage flow rates (Babbitt and Baumann 1958 (11)). Sanitary sewage flow will typically follow a diurnal pattern. Minimum flow occurs during the early morning hours when water consumption is lowest. The first flow peak generally occurs in the morning when the peak morning water use reaches the

Figure 6
Algorithm Schematic for Sanitary Flow (SANF)

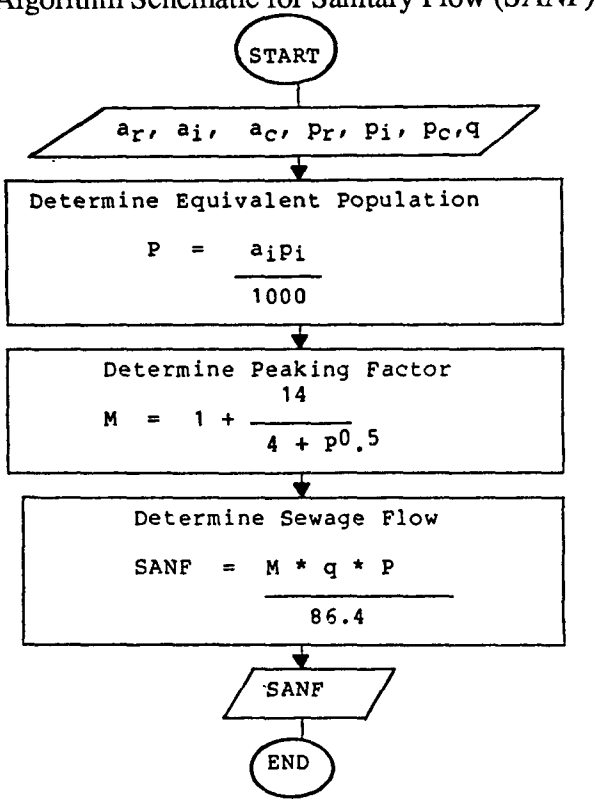
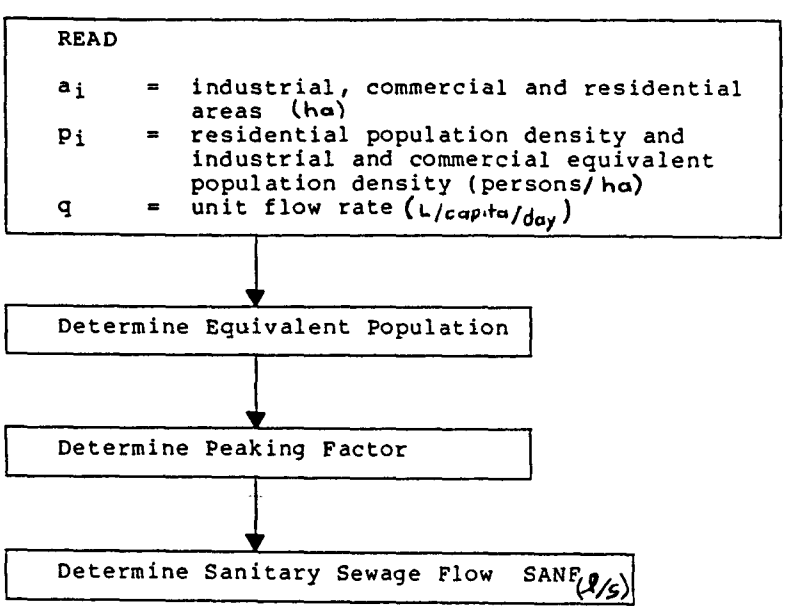


Figure 7
Logic Schematic for Sanitary Flow



treatment plant. A second peak flow generally occurs in the early evening hours between 7:00 p.m. and 9:00 p.m. This pattern is normally constant throughout the year, however, weekly or seasonal variations may also occur. For example in areas which have a high industrial flow component and definite shift times, or in areas which have a high percentage of seasonal users e.g. in cottage or trailer park areas, such variations may occur. In all cases, for a sanitary sewer with high inflow and infiltration, the diurnal variations are insignificant in comparison with the seasonal variations experienced from high groundwater and rainwater inflow. For this reason, diurnal, weekly or seasonal variations are not considered further here; the sanitary flow component is determined on the basis of a daily peak flow.

To obtain the daily peak flow, a peaking factor (M) is applied to the average daily flow. Various peaking factors have been developed from empirical relationships, for example Harman (1918 (12)).

Alternatively, the user may prefer to develop a site-specific peaking factor, if sufficient average daily flow and peak daily flow information is available.

INFLOW (QINFLOW) AND INFILTRATION (DRAINF)

To compute the inflow from direct surface runoff, existing storm runoff algorithms in PCSWMM can be used. This flow determination will be referred to hereafter as QINFLOW. The user must consider drainage area from roofs, pavement areas, grassed areas, etc. which are directly connected to the sanitary sewer. A separate area determination must be made for each sewershed. Surface water percolation and groundwater contributions are more difficult to compute.

An algorithm for computing the groundwater elevation over a continuous time period is presented in Figure 8. Also presented is a method for computing an infiltration flow (DRAINF) into a sewer system given the groundwater elevation above the sewer. Figure 9 defines the input variables and provides the user with a series of steps to follow when using DRAINF.

To calculate infiltration into the sanitary sewer a water balance must be carried out for each integration interval - this gives the amount of groundwater present in the soil and its head relative to the sanitary sewer. The terms which make up the groundwater accounting model are as follows: The surface infiltration source (INF) as defined by El-Kadi and Heije (1983 (13)) is the entry of water from the air side of the air/soil interface into the soil profile. The amount of water which infiltrates may not directly contribute to the groundwater accounting model. A percentage of the surface infiltration may be lost to evapotranspiration in the upper soil zone (EVAPFR) or lost in deep percolation in the lower soil zone (DEEPFR). The combined loss of surface infiltration is LOSSFR.

In the RUNOFF Module surface infiltration is computed by either the Horton or Green-Ampt infiltration algorithms. Both of these algorithms subtract evaporation from rainfall depths prior to calculating infiltration. In DRAINF, the computed infiltration from the Horton or Green-Ampt algorithms is further reduced to reflect the loss of available groundwater due to DEEPFR and EVAPFR. Both of these losses continue to occur after rainfall has stopped. The losses of groundwater to deep percolation and evapotranspiration are treated as calibration parameters.

Flow from external sources is difficult to compute without extensive knowledge of the groundwater movement. For this reason, in this study, the external source flow is taken to be a

Figure 8
Algorithm Subsurface Infiltration Flow (DRAINF)

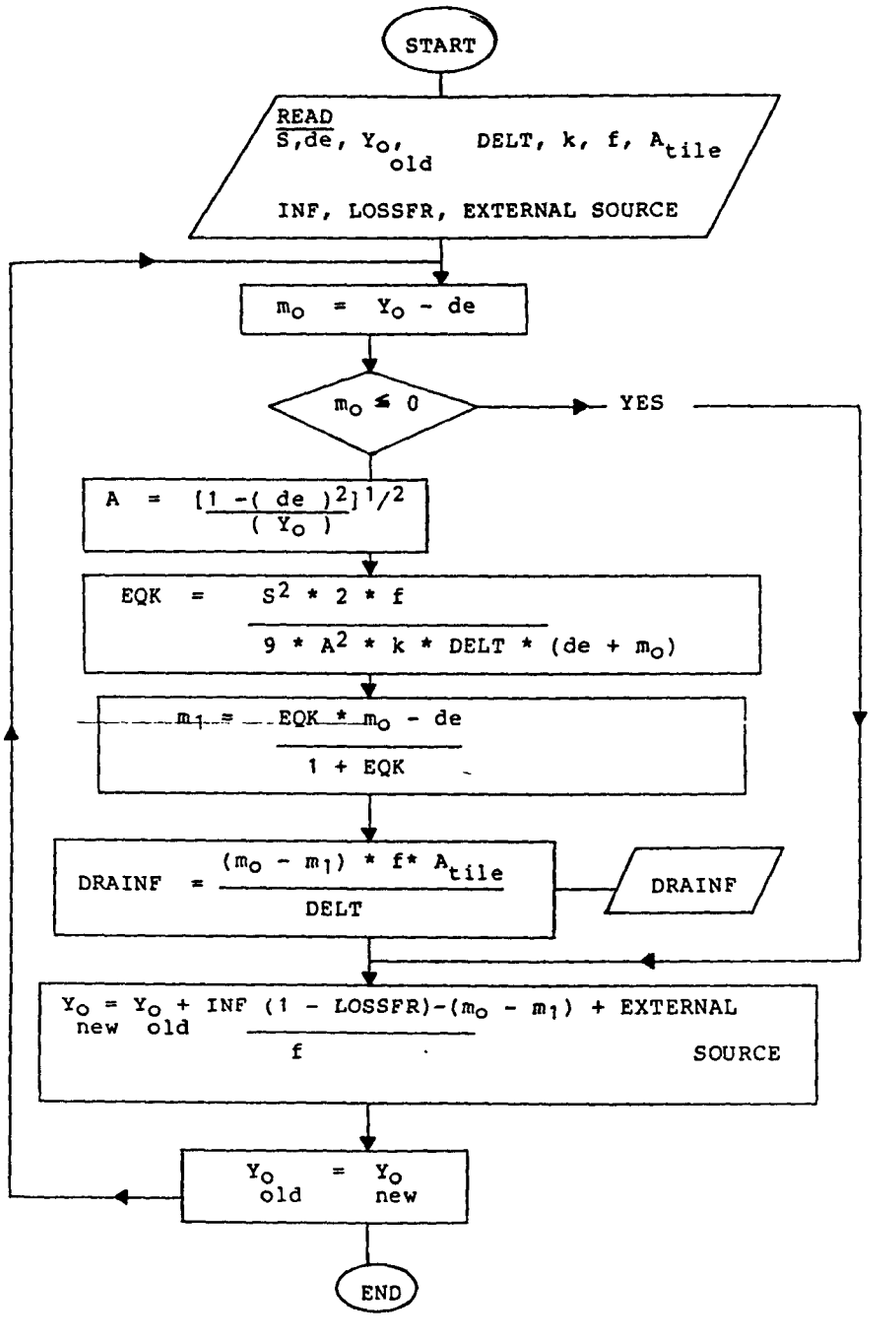
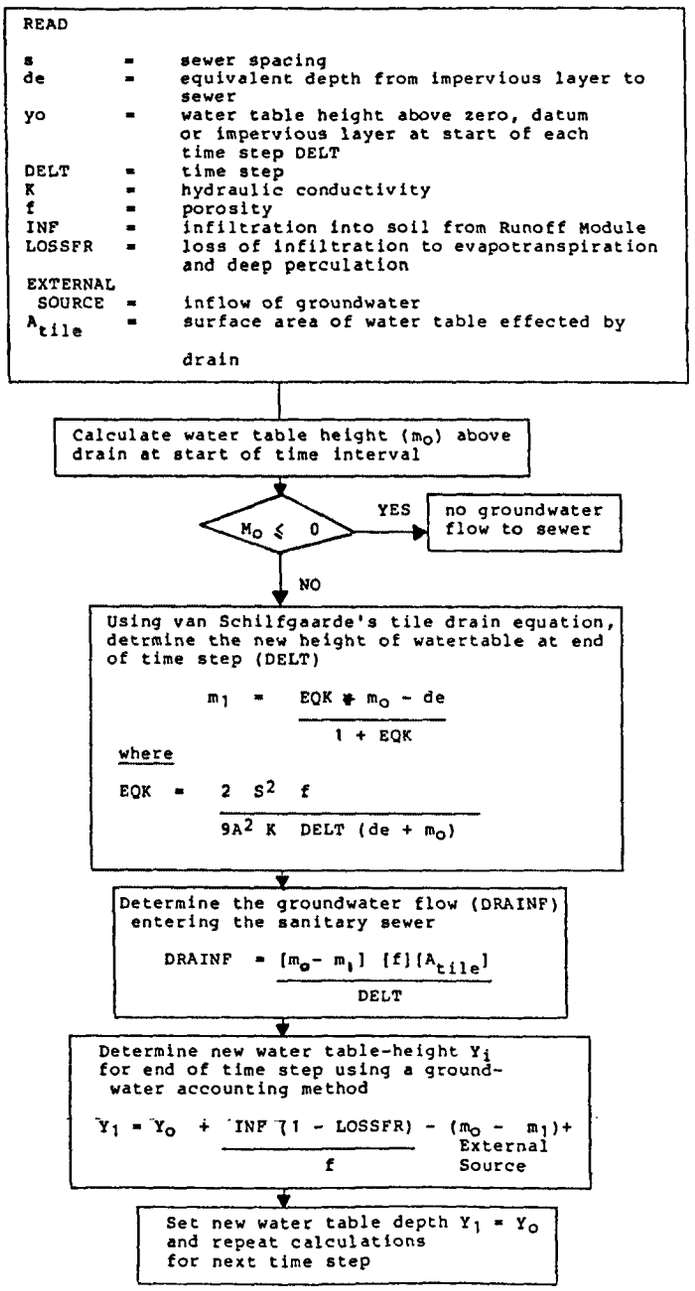


Figure 9
Logic Schematic or Subsurface Infiltration to Sanitary Sewer (DRAINF)



calibration parameter. The external source parameter may also include lateral inflow from other sewersheds. The external source is entered in units of inches of groundwater depth.

The final variable to be defined in groundwater accounting is DRAINF. This variable represents pipe infiltration into a sanitary sewer through cracked pipes, joints and porous pipe materials.

The hydraulic theory underlying the computation of DRAINF is considered to be the same as a tile drain subjected to a groundwater head. An empirical relationship for tile drain spacing with a falling water table condition was developed at North Carolina State College by van Schilfgaarde (1963 (14)). It should be noted that the water table in van Schilfgaards's equation refers to a water table condition in the immediate area of the tile drain or sewer being considered. The equation may be rearranged so that the water table measured above the top of pipe (m_1) can be determined at the end of each time step (DELT).

The groundwater flow into the sanitary sewer (DRAINF) can then be determined from the following relationship (Thompson 1979 (15)):

$$DRAINF = \frac{(m_0 - m_1)(f)(A_{tile})}{DELT}$$

where:

m_0	=	watertable height above drain at start of DELT (m)
m_1	=	watertable height above drain at end of DELT m)
f	=	soil porosity, (% of volume)
A_{tile}	=	surface area of water table affected by DRAIN (m^2)
DELT	=	time step (sec)

The infiltration flow DRAINF is determined from the computed drawdown multiplied by the drawdown area. The area is determined by the product of the length of sewer within the drainage area and the width of drawdown on each side of the sewer. The width will depend upon the amount of infiltration which is occurring into the sanitary sewer. This in turn will depend upon the age or condition of the sewer and the amount of open joints, cracked pipes, and porous materials.

It should be noted that the drawdown, m_0-m_1 is multiplied by the soil porosity. The drawdown relates to the soilwater depth within the ground profile. The soilwater depth may comprise 80% soil material and 20% water for a soil having a porosity of 20% by volume. To arrive at a cubic metre volume amount of water, the drawdown [$m_0 m_1$] must be multiplied by the soil porosity.

With all the variables defined, an accounting or water budget expression can be developed as follows:

$$Y_{new} = Y_{old} + \Delta y$$

where:

$$\Delta y = \frac{INF\,(1 - LOSSFR)}{f} - (m_0 - m_1) + \text{External Source}$$

The reader should recognize that Δy refers to a water depth in the soil profile. For this reason, the infiltration value of the water budget equation is divided by the soil porosity value.

Infiltration within the equation is measured in inches of rainwater and to convert this value to inches of soil water, the infiltration value is divided by soil porosity (f). The amount of drawdown which was computed in DRAINF will reflect the amount of soil water lost to pipe infiltration. The final variable in the water budget equation reflects the increase in soil water due to inflow from an external source, which should be input in units of soil water depth.

CONCLUSIONS

The loss of sanitary capacity to groundwater infiltration has become a problem for most municipalities with sewer systems installed in the early 1900 era. For a variety of reasons, e.g. construction methods, materials and quality control, many of the once separated sanitary sewer systems must now be analyzed, maintained and managed as combined sanitary sewers.

The sources of flows in combined sanitary sewers are complex and varied. A broad range of solution alternatives are available. Many of the solutions can be disruptive and expensive to implement in a built-up urban environment. Sewer operating authorities and sewer design engineers have identified a need for a method of analyzing and managing flows which are occurring in combined sanitary sewers.

Most hydraulic models, SWMM being one of the most widely accepted and used, do not provide a groundwater accounting procedure or a pipe infiltration prediction method. In the case of SWMM, the user is required to input a "lumped" value for all pipe infiltration from the various sources. The user's prediction is based on his personal experience or from suggested textbook values or values used by other operating authorities. The combined sewer prediction results for Site A and Site B demonstrate the sensitivity which this variable has on obtaining a satisfactory output hydrograph. An error in judgment in selecting a proper infiltration parameter will provide unsatisfactory results.

The groundwater accounting procedure and pipe infiltration procedure proposed in DRAINF will allow the user to predict pipe infiltration and to compute groundwater fluctuations on a continuous basis. The user will be able to disaggregate and compute the various flow components which make up a combined sanitary sewer flow. The code, testing and verifying of these new algorithms have not been carried out.

The review of SWMM has also indicated certain limitations in the present FILTH algorithm contained in the SWMM Transport Block. The data required for this module is <u>not</u> readily available from most municipal data bases. Further, the sensivivity of certain variables contained within the FILTH algorithms do not provide any significant change to the output results, e.g.diurnal variation parameter, % garbage grinders, house value, household income value. An alternative sewer prediction method (SANF) which will generate only peak sewage values using available data from a municipal data base source has been suggested as an alternative computational algorithm for combined sanitary sewers. The diurnal or weekly variation in sewage flow is insignificant in comparison to the total combined sewer flow. For this reason, the new method predicts only peak sewage flow.

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SEWERCADD

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ABSTRACT

Sewercadd, developed by Bovay Northwest Inc., is a product of the relatively recent availability of powerful and flexible micro computer programs. Simply stated Sewercadd is a practical application of a micro computer-based design model which has been expanded to include drafting and database functions. Sewercadd includes enhancements to the design model to assist the design engineer both in design and construction of a sewer system. These enhancements were developed around a database management system. A high priority in the development of Sewercadd was to provide consistency between the documents required to design, bid and build a sewer system such as plans, cost estimates, bid schedules and design calculations.

This paper describes the comprehensive system, Sewercadd, that has been developed from these programs and used by the authors to design and monitor the construction of several gravity sewer systems in the Spokane area.

The purpose of this paper is twofold. The first is to encourage, by example, computer design model users to take advantage of the power and flexibility of the micro computer-based design models available today. The second is to suggest that design model developers provide flexibility in their models without sacrificing the power and features now available.

SEWERCADD

INTRODUCTION

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THE SEWER DESIGN PROCESS

The design of a sewer system is an iterative process. The preliminary system layout is done with limited field data and approximate design flows. This information is then entered into a computer model for calculation of the required size and slope of the entire system. The model's results are then manually drafted. As field information becomes available it is also plotted, and conflicts with existing utilities are resolved by the design engineer. The input to the model is then edited based on this additional information about conflicts and any revised design flows. The model is then rerun to refine the design. The process is repeated until the design is finally accepted. This is a time-consuming iterative process which

requires a great deal of labor. The manual drafting results in a significant time delay between the introduction of new information and the ability of the engineer to analyze it in the context of the design.

The Sewercadd design process (as shown in figure 1.) has the same features and flow as traditional methods. Sewercadd, however, simplifies the design process by integrating hydraulic and hydrologic sewer design calculations with drafting and data management. Sewercadd manages all other project related data as well, resulting in accurate and consistent production of material takeoffs, estimates and bid schedules. The accuracy and consistency of these documents helps create a bidding environment that results in better and lower construction bids.

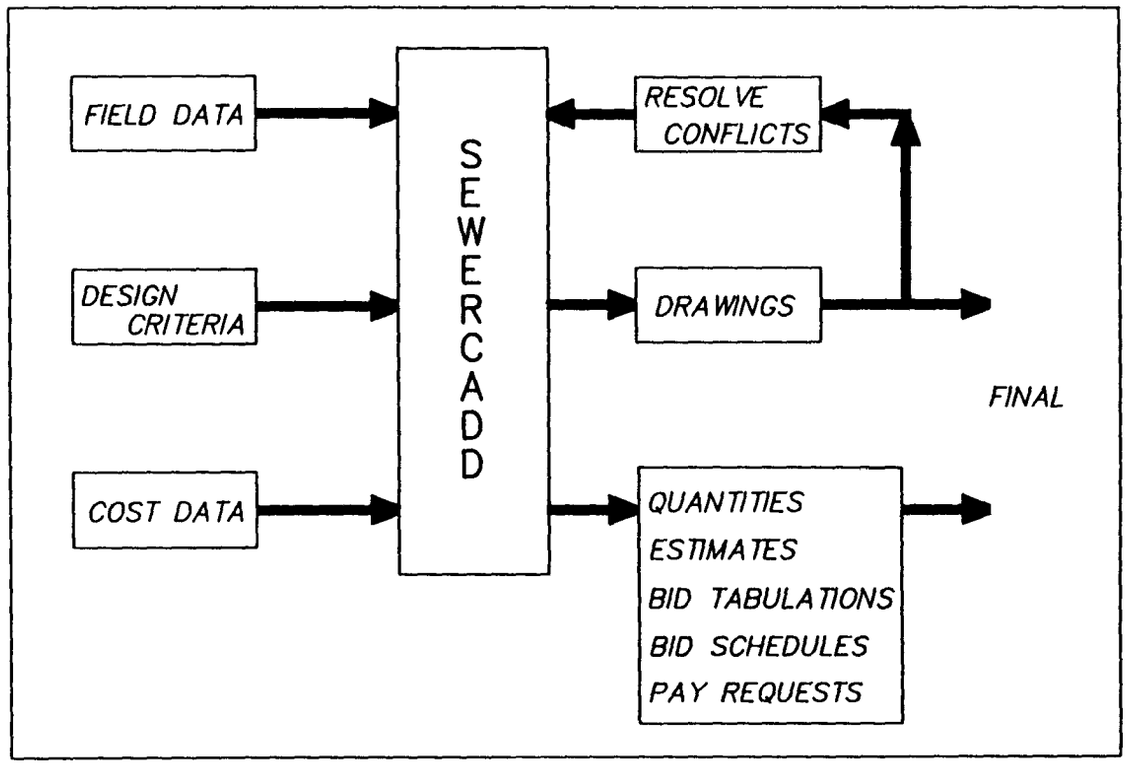

Figure 1. Sewercadd design process

SEWERCADD MODULES

The Sewercadd system is composed of three modules; the HYDRA design module, the Database Manager, and the Drafter. The modules accept input from a variety of sources and process then pass along the data to the other modules of sewercadd to produce all the required documents for a sewer design project as shown in figure 2.

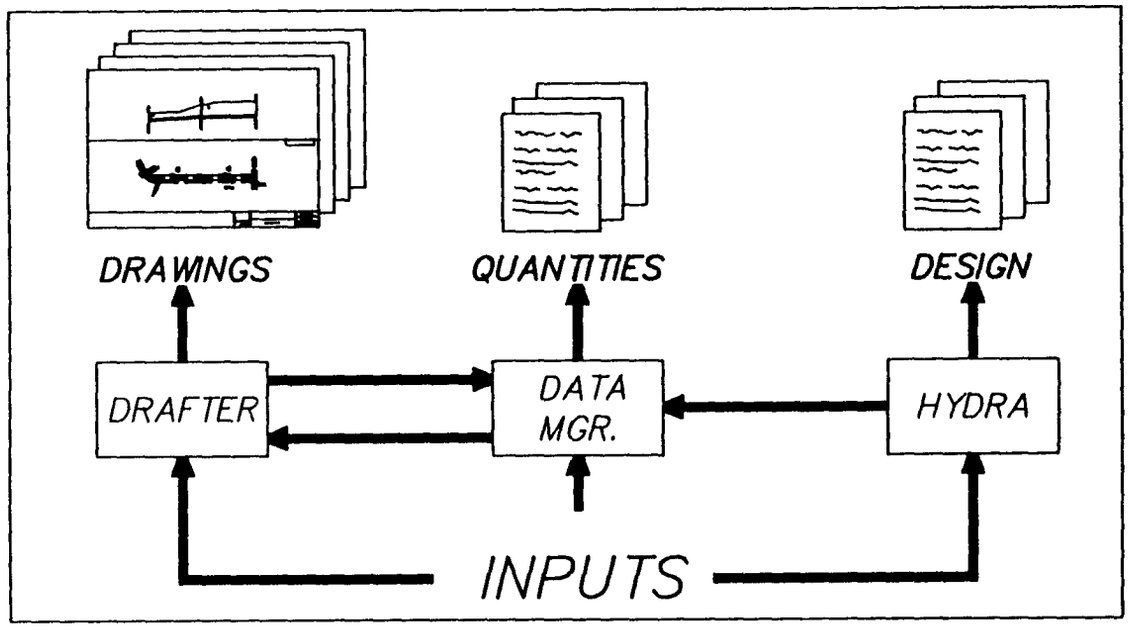

Figure 2. Module interaction

The building blocks for each of the Sewercadd modules are popular micro computer programs;

HYDRA[1],dBase III[2] and AutoCAD[3]

The flexibility and programmability of both dBase III and AutoCAD along with HYDRA's output data, made the complex and sophisticated integration of the Sewercadd modules easier to develop.

The Sewer Design Module, HYDRA

The sewer design module, HYDRA, is a flexible storm and sanitary sewer system analysis and design program. Given design criteria, a ground profile, and service area information, HYDRA can provide a preliminary design of pipe size and vertical alignment. HYDRA uses the traditional "peaking factor" concept to generate sanitary flows but gives the user the option of calculating storm flows by either the Rational Formula or by using hydrological simulation techniques. HYDRA also has functions to prepare cost estimates and financial analyses of the sewer systems. An extensive set of data is output in a ASCII text file.

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[1]HYDRA is a registered trade mark of Pizer Assoc.
[2]dBase III is a registered trade mark of Ashton-Tate.
[3]AutoCAD is a registered trade mark of Autodesk Inc.

The basic unit in the HYDRA model is a link of pipe which is the length of pipe between two manholes. HYDRA, after optimizing the system design, outputs 75 pieces of information on each link. This information includes all design information (pipe size, flow, velocity, etc.) as well as cost information for that link. Sewercadd moves this information from HYDRA into the Sewercadd Data Manager were it serves as only part of the project database.

Data Manager

The Data Manager manages all pertinent design information. dBase III was used as the base to develop a collection of routines to input, edit, create and manage information from a variety of sources. The HYDRA model, as described above, provides an extensive set of data on each link. In order for Sewercadd to complete the design, other information is required from the engineer. This information can be input through standard dBase features and includes:

- Existing utilities that cross the proposed design
- Additional ground line information
- Manhole numbering
- Information used to combine the links output from Hydra into drawings
- Stationing

The Data Manager is the heart of Sewercadd. The Data Manager stores all the project design data and can output this information to the other modules of Sewercadd. The Data Manager also can output reports for cost estimating, material takeoffs, bid schedules and construction pay requests. This module helps the engineer edit as well as enter data into the computer quickly and efficiently and then combine it with the HYDRA results.

While the Data Manager is key in the development of sewer profiles, it also is used in the collection of additional information to develop a complete cost estimate of the proposed project. Although Hydra can develop quantities for pipe and the associated excavation and paving, there are many more items that will make up the entire estimate. These items such as mobilization, tree removal, and drainage structures are entered into the Data Manager to develop a complete cost estimate. The list of items, when edited and refined, can then be used to output a bid schedule that becomes part of the bid documents. The Data Manager also has routines to prepare bid tabulations to check each contractor's bid, as well as other routines to help with construction management and calculate the payments due the contractor through the construction phase.

The use of a consistent data base from design to project closeout is a strong advantage of Sewercadd. For example, the

database is automatically updated as the project design proceeds, therefore any material takeoffs and cost estimates are output from up to date information. Since the Sewercadd Drafter uses the database as it's source of data, the drawings will be consistent with the design.

The Drafter

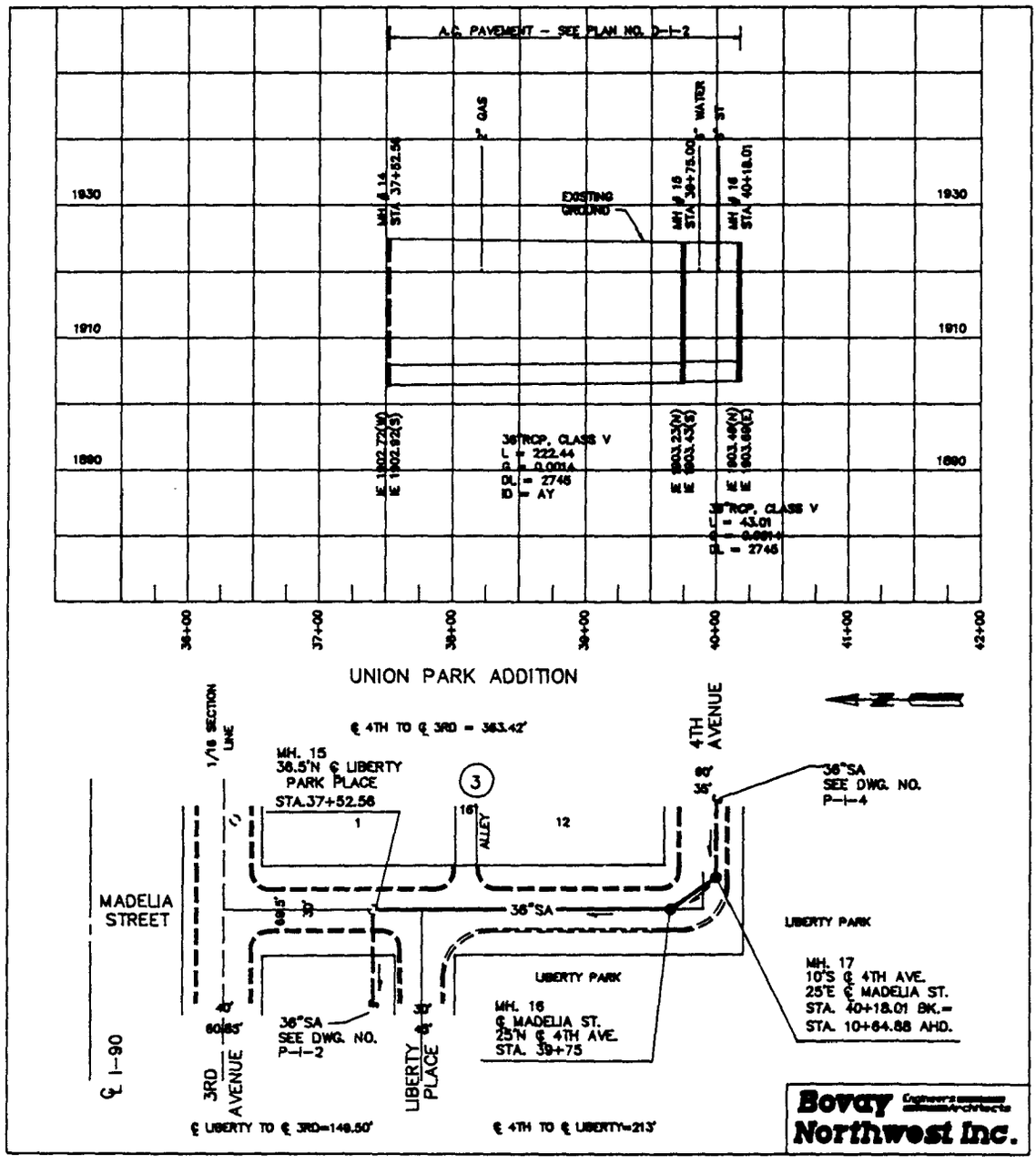

Figure 3. Sample plan and profile drawing

The Drafter is the module that assists in the creation of the drawings of the sewer system (see figure 3). It is a collection of routines written in Autocad's Autolisp language.

The Drafter extracts information from the Data Manager to draw the sewer profile including: groundline, pipe links, manholes and existing utilities.

The Drafter accurately locates this information on the drawing's grid and adds all required labels and notations. There are routines to avoid text writing over other items already in the drawing. The Drafter has a set of parametric drafting tools to assist in the creation of the plan view of the proposed sewer and the plan background showing all property lines, existing utilities, and roadway curbs an sidewalks. The drawings (plan and profile) can then be reviewed and the data edited in Sewercadd's Data Manager and then the profiles redrawn until a final solution is reached.

The Drafter also has a sophisticated method to automatically organize the HYDRA output links into drawings. These routines are based around a system diagram (see figure 4) drawn using standard AutoCAD techniques. The sewer system diagram is created with approximate manhole locations. Manhole numbers and link identifiers are added during this process. The information from the system diagram is passed to the Data Manager to assist in determining drawing layout and connectivity of the links.

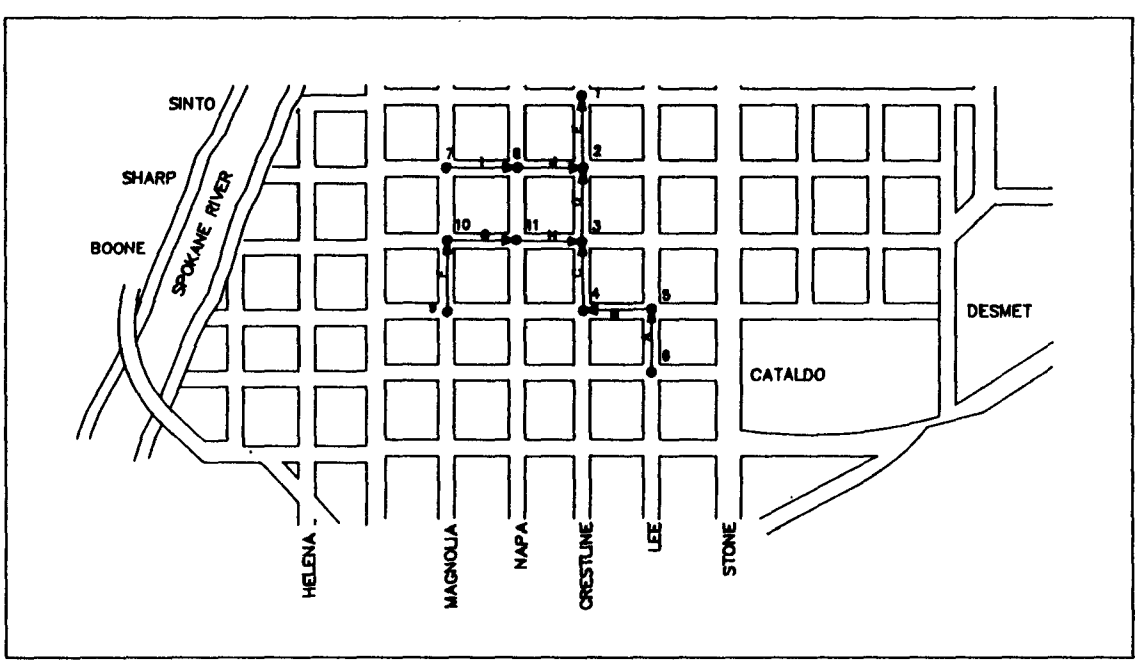

Figure 4. System Diagram

ADVANTAGES OF THE SEWERCADD SYSTEM

The Sewercadd system enhances the use of computer design modeling techniques to ease the effort for the design engineer not only in designing a sewer system but also in the other tasks included in the engineer's list of responsibilities. As in any computer program, a big advantage is the quick iterations which can be run. However, in this case the iterations include not only design calculations but also the production of graphical output. This allows the engineer to review the proposed alignment relative to utility interferences or other existing or proposed improvements more quickly on either a hard copy plot or on the computer monitor.

Using computer aided drafting methods allows all the intermediate plan and profile plots to be color coded. These plots make ideal check plots as often times there are complex networks of existing utilities to be checked.

Because Sewercadd is an integrated system there is consistency between the database functions, such as cost estimates and bid schedules, and the graphical products (the plan and profile drawings and details) produced by the system. Sewercadd's comprehensive database insures consistent plans, cost estimates, specifications, bid quantities, bid schedules and design calculations. Retaining consistency is a persistent problem for the practicing engineer as the design changes during the design phase. Inconsistency between plans and other bid documents can lead to expensive construction change orders.

In general, Sewercadd allows more time to be spent improving the design and less on drafting, material takeoff and corrections. The drafting quality is very high. Sewercadd is simple, flexible and effective in improving the design and reducing the construction cost. Additionally the computerized graphical data is a valuable resource for the owner of the Project.

CONCLUSIONS

The advent of exceptionally powerful and fast micro computers allows the design engineer to use powerful computer design models, Cadd systems and database managers in a hands on environment. These computer tools are tending to be flexible with programming languages for customizing each application. Because each engineer and design situation is unique no one system can provide for all the needs. The answer for the sophisticated user is to create his own system which meets the specific needs that the engineer has identified.

There is a trend, by design model and computer programs in general, toward allowing the user to customize as appropriate for his own needs. To take full advantage of this power and

flexibility will require two critical skills in the user. The First required is a vision about what is possible and what will work in their environment. The Second skill necessary the complete understanding of the actual requirements of the task to be automated. These skills enable users to turn powerful programs into effective tools to serve their needs.

For the model writer, then, it may be more important not to try to anticipate user needs but to allow the data to be manipulated in a way to provide for them. The computer model HYDRA did not provide the cost estimating functions suitable for our unique situation, nor did it generate useful graphical output. The model did, however, provide a data dump with every possible bit of information included. Likewise, AutoCAD provided a programming language, Autolisp, which could manipulate input data into very complex output.

Finally, Sewercadd's Database Manager was developed around dBase III and allowed input from several different sources and in different formats which provided the nexus for design and drafting functions.

It is anticipated the Sewercadd system will be further enhanced. Currently Sewercadd requires a relatively sophisticated computer user to interface between the design engineer and the programs. The next step will be to provide a graphical method of input which will interpret the user's requirements for sewer location and capacity from a schematic basemap. This basemap would be the basis for creating input data for the design module. The engineer would be provided with a graphical method to check this input data. The resulting improvement will allow an engineer, who is a relative computer novice, to utilize the entire system for design and drafting without the assistance of a computer specialist. The engineer would then be left with a powerful tool to deal with the drudgery of both design and drafting while leaving all of the creative problems to be solved with the imagination of the engineer.

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CURRENT TRENDS IN AUSTRALIAN STORMWATER MANAGEMENT

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ABSTRACT

Despite Australia's small population, it still suffers the same urban sprawl as other densely populated countries. Over the last decade significant advances in urban stormwater management have taken place.

Future directions in modelling techniques will revolve integrally with the very rapid advances in computer technology; this in turn will open up the ability for far more professionals to be involved in complex analysis. The profession will need to address the problems associated with black box analysis by persons with insufficient experience to question the results.

INTRODUCTION

Australia, with a land mass of 7,682,300 km^2 experiences climatic extremes and has a varied topography. There are rain forests and vast plains in the north, snowfields in the south-east, desert in the centre and fertile croplands in the east, south and south-west.

In hydrologic terms it is a land of contradictions. While the average annual rainfall of only 465 mm represents the driest continent in the world, mean peak annual floods, relative to mean annual runoff, are about an order of magnitude larger than world figures, McMahon, 1982[1].

Additionally despite Australia's relatively low population density of two persons per square kilometre representing a population of 16 million in a continent 82% the size of North America, it is also one of the most urbanised countries in the world with 70% of the population living in the 10 largest cities and 83% of the population classified urban. More than six million people live in the country's two largest cities of Sydney and Melbourne. See Figure 1.

It is not surprising therefore that parts of Australia suffer the same stormwater quality and quantity problems generally associated with far more populated countries around the world.

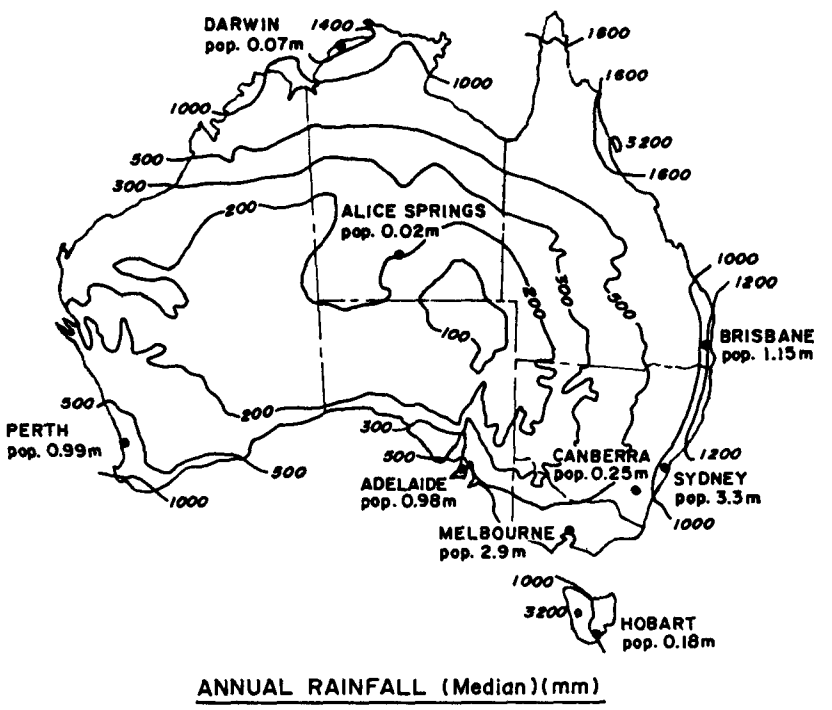

Figure 1. Australian representative rainfall and population statistics

COMPARATIVE STATISTICS

METEOROLOGY INPUTS

Stormwater management in Australia has to respond to the extreme variations in hydrologic regime occurring across the country. Table 1 details typical meteorological inputs that directly affect urban stormwater management.

As can be seen from Table 1, in a number of areas the combined affects of relatively low design rainfall intensities, very intermittent rainfall events and high evapotranspiration rates, make the estimation of design loss rates and the consequential runoff extremely difficult.

WATER QUALITY

Water pollution emanating from non-point source urban stormwater represents a growing problem in a number of Australian cities. Pollution

TABLE 1. COMPARATIVE METEOROLOGICAL STATISTICS

Location	Annual Rainfall (mm)	Annual Evaporation (mm)	1 hr Rainfall (mm)	
			1 in 1-yr AEP	1 in 100-yr AEP
Adelaide	531	1700	12.0	40.0
Alice Springs	200	3600	13.6	46.4
Brisbane	1157	1900	36.0	90.0
Canberra	639	1550	17.0	50.0
Darwin	1536	2800	50.0	95.0
Hobart	633	1100	11.0	30.0
Melbourne	661	1550	15.0	45.0
Perth	879	1900	15.0	35.0
Sydney	1215	1600	31.0	85.0

loadings are in the same order as North American data for a range of constituents, including total solids, suspended solids, nutrients and bacteria. Table 2 indicates typical Australian annual constituent loads in relation to some North American data.

Although nutrient washoff in urban stormwater at first sight would appear to equate with North American data, total phosphorus for example appears in Australia in far more particulate form. Australian soils are generally much lower in organic content than northern hemisphere soils. Based on local data, Lawrence, 1986[5], available phosphorus to algae was found to be only 30% urban runoff content plus 10% rural runoff content. This has particular ramifications when examining the behaviour of local lakes and problems of eutrophication.

RUNOFF

Recent research by McMahon, 1982[1] and Finalyson et al, 1986[6] have shown that Australian catchment runoff exhibit significant variations to both North America and world data. Australian streams are more variable than world rivers. Additionally, McMahon, 1982[1] states that relative to mean annual runoff, mean peak annual floods are about an order of magnitude larger in Australian streams than world figures. However, when catchment area is taken as the independent variable world streams produce larger mean annual floods.

TABLE 2. COMPARATIVE WATER QUALITY CONSTITUENT DATA

	Australia (Canberra)* kg/ha/yr		USA (Urban) kg/ha/yr	
Constituent	Rural	Urban	Durham, Nth Carolina†	Washington DC‡
Sediment	293	2153	5954	-
Suspended Solids	15	332	-	74
Total Kjeldahl N	.17	4.7	5.4	3.60
Total Phosphorus	.12	.61	4.2	.66

* Willing & Partners Pty Ltd, 1986[2]
† Colston, 1974[3]
‡ Randal, 1982[4]

Analysis of 100-year flood data by McMahon suggests that Australian catchments yield per unit area peak discharges that are about 60% more than world values. Figures 2 and 3 indicate the typical findings of McMahon, 1982 and Finlayson et al, 1986.

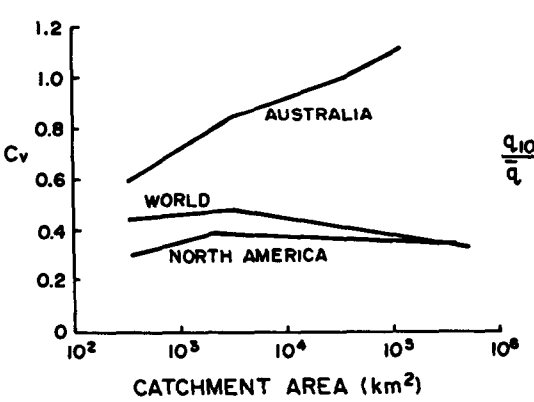

Figure 2. Coefficient of Variation of Annual Flows versus Catchment Area

After Finlayson et al, 1986

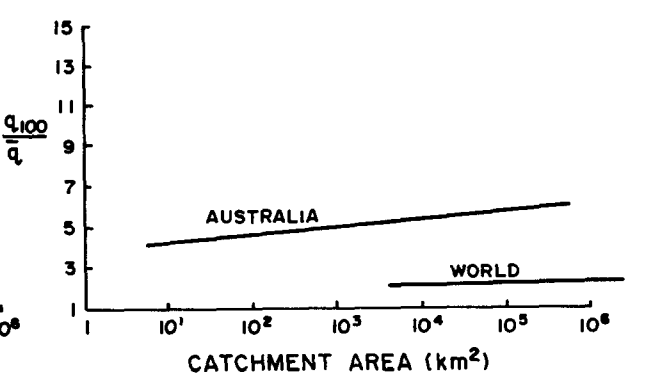

Figure 3. 100-year Floods Expressed as Ratios of Mean Annual Floods and Area

After McMahon, 1982

It is likely therefore that traditional flood frequency extrapolation techniques based and tested on northern hemisphere data may be questionable without detailed examination using local data.

URBAN STORMWATER MANAGEMENT CRITERIA

WATER QUANTITY

Over the last decade there has been a significant shift in Australia to catchment wide stormwater management and in recent years to the integration of this with wider urban planning inputs. Based on a number of judicial findings the legal responsibilities for flood damages associated with a "duty of care" has forced local councils and state government authorities to think out the wider issues of stormwater planning and advice.

In general a minor/major stormwater system has developed with a piped urban system to take flow peaks between 1 and 10-year AEP and a surcharge system via roads and formal floodways to take rarer events up to and including the 1 in 100-year AEP flood. Where retention basins of significant size have been included into a drainage system it has been common to size emergency spillways and embankment protection to frequencies of between 1 in 1000-year AEP and the MPF.

In some instances the setting of such prescriptive flood frequency levels has led to over protective measures excluding the development of otherwise valuable land. Additionally the adherence to, say, the 1 in 100-year AEP as a guide to flood protection in older areas under redevelopment has led to adverse social reaction.

In the state of New South Wales a recent Floodplain Development Manual (NSW Government, 1986)[7] has been issued to assist councils in developing plans for the management of their floodplains.

The policy takes into account that "flood liable land is a valuable resource and should not be sterilised by unncessarily precluding its development". Central to the policy is the requirement that all development proposals be treated on their merits.

This policy places considerable responsibility on individual councils to carry out adequate catchment wide flood studies to base sound management principles on flood hazard, economic factors, environmental planning and development control. This departure from a standard flood frequency requirement such as the 1 in 100-year AEP is likely to, in the shorter term, involve difficult decisions. In the longer term it is expected that the preparation and implementation of overall management plans will incorporate the merit approach to its fullest extent.

At an individual development level a range of acceptance criteria is usually applied to minimise both nuisance flooding and major hazard from flooding of roadways and buildings. Table 3 indicates a typical set of acceptance criteria being applied to urban areas within Canberra in the Australian Capital Territory.

TABLE 3. TYPICAL DESIGN ACCEPTANCE CRITERIA - CANBERRA*, A.C.T.

Surface Flow Regime				Subsurface Flow Regime	
Situation	Limiting Criteria†			Situation	Return Period
	d	v.d	at Return Period	Roads - 1-lane clear‡	
Roads - general	< .2	< .4	100	Minor	2
Access to emergency facilities	< .2	< .4	100+	Collector	5
Pedestrian trafficable floodway	< .75	<1.0	100	Distributor	10
Other floodways	<1.0	<1.0	100	Ordinary arterial	20
Other areas and§	Neg	Neg	5	Inter urban arterial	50
Hospital and defence facilities	< .1	< .1	100	Access to emergency facilities	100
				Urban development#	
				Buildings and trafficable areas to be drained to prevent damages to return period specified	
				Residential -	
				- Low density	5
				- Medium density	10
				- High density	20
				Shopping and commercial -	
				- Local	10
				- Regional	20
				Industrial -	
				- Light	20
				- Heavy	50
				Hospitals and emergency service areas	100+

Notation

v - velocity of flow (m/sec)
d - depth of flow (m)

Return Period - expressed in years

Notes:

* Limiting criteria set for Canberra region only. In other areas these would need to be adjusted to local rainfall regime.

§ Limit set to restrict surface flows being routed through private property.

‡ Criteria directly related to traffic density. Should be adjusted where situation warrants.

\# The surface flow regime should be sized to take into account partial pipe failure through blockage wherever this could possibly occur.

† In all cases the affects from flows in excess of the proposed limiting criteria should be minimised wherever possible.

The stated criteria places as much importance on the control of surface flows resulting from infrequent storm events as the removal of frequent flows from on and about the urban pedestrian and vehicle transport network.

The table sets three basic limits, being:

(a) the velocity-depth limit that has been found in association with the depth of flow to govern the stability of vehicles and the ability of pedestrians to "walk out" of flood flows,

(b) the depth limit, and

(c) the return period limit which is the economic criteria, for which damages should not be occasioned.

Resulting from the approach described in Table 3 a constant pipe design frequency is not followed. To maintain acceptable road surface flows for example it may well be possible and sometimes necessary to either decrease or increase the design frequency of the pipe system over particular reaches of the network.

WATER QUALITY

Since the enactment and implementation of environmental protection legislation in Australia, in the early 1970s, by the six states and by the Federal Government there has been growing concern in Australia about the actual and potential impacts upon receiving water bodies of polluted urban runoff. Noteworthy studies have been carried out for the two principal inland cities in Australia, ie, Canberra and Albury-Wodonga, by Cullen, Rosich & Bek, 1978[8] and by Gutteridge, Haskins & Davey, 1974[9].

It has been found that phosphorus is the limiting nutrient for a range of Australian freshwater lakes. Bliss, Brown & Perry, 1979[10] reported on investigations of the pollution potential of urban runoff in Sydney. They reached the conclusion that the pollution potential of urban runoff in Sydney was high and that both the more commonly determined pollutants such as non-filterable residue, bio-chemical oxygen demand and nitrogen and phosphorus forms, oils and polycyclic aromatic hydrocarbons may cause severe degradation of certain Sydney receiving water bodies.

In recent years investigation, planning, design and implementation of water quality control schemes have been carried out in Canberra, Goyen, et al, 1985[11] and Lawrence & Goyen, 1987[12], to combat future water quality degradation due to continued urbanisation. Up until recently however point source control and the treatment of wastewater has taken up the majority of the country's resources in this field. Monitoring within the A.C.T. has shown that a change from rural to urban land use has entrained a seven to tenfold increase in the level of export of a range of runoff constituents.

Receiving water quality objectives adopted for planning in Canberra, Lawrence 1986[5] have been based on the protection of designated uses of the waters of lakes and streams and aquatic ecology. Ecological criteria determined by bio-assay techniques have been found to have little relevance

to local fauna and the quality of lakes and streams in the region as to date they have generally been well within broad water quality objectives. It has been general policy to closely monitor changes in lake and stream ecology as the primary basis to reviewing pollution control strategies.

Rather than restricting absolute constituent concentrations from new developments to downstream receiving waters, acceptance criterion in the A.C.T. has relied more on maintaining rural water quality levels, or as near as possible, after urban development has taken place.

CURRENT MODELLING TECHNIQUES

WATER QUANTITY

As with overseas practice there has been a progressive trend over the last 10 years towards computerised analysis and design. This has occurred at both the minor stormwater reticulation level as well as on the flood mitigation level.

Minor Systems

For piped drainage systems the main analysis techniques still revolve around the Rational Formula, Messner & Goyen, 1985[13] although ILLUDAS, O'Loughlin & Mein, 1983[14] and SWMM, Carleton, 1983[15], Attwater & Vale, 1986[16] have recently been applied to a limited number of Australian catchments.

Table 4 indicates a range of urban models that have gained at least rudimentary use in Australia.

ILLUDAS developed from the TRRL Method by Terstriep & Stall, 1974[17] has been recently further developed in Australia by O'Loughlin, 1986[18]. O'Loughlin & Mein, 1983[14] stated that ILLUDAS even with recent Australian improvements would not make it suitable for detailed pipe design, including such considerations as pit energy losses and cover depths.

WASSP is a program suite developed by the UK National Water Council, 1981[19] which offers similar capabilities to ILLUDAS although to date has not been widely used or tested on Australian systems.

SWMM is a comprehensive program suite supported by the US EPA, Huber et al, 1981[21] that concentrates on urban piped systems. It provides full unsteady flow and backwater effects within the system through the use of the EXTRAN block. •Overflow rerouting and limited inlet capacity consideration is not presently covered. Carleton, 1983[15] when attempting to model an existing catchment in Sydney for a range of severe storm events was only partly successful, since the model could not take into account the extensive blocking and restrictions associated with inlet pits which were a major cause of the flooding.

PIPENET is a propriety drainage design model developed by Bloomfield, 1981[21] based around the Rational Formula that is offered as an interactive tool to design new piped systems. The model does not directly address variable pressure change coefficients, surcharges or surface flow rerouting.

TABLE 4. CHARACTERISTICS OF AUSTRALIAN USED URBAN STORMWATER MODELS

Description	Models					
	ILLUDAS H	WASSP H/H	SWMM H/H	PIPENET H/H	RAFTS/RSWM H	RATHGL H/H
Uses						
Design of New Systems	x	x	x	x	x	x
Analysis of Existing System	x	x	x		x	x
Water Quality Analysis			x			
Hydrology						
Rational Method				x†		x‡
Modified Rational Method		x				
Simple Hydrograph Routing	x	x	x			
Complex Hydrograph Routing			x		x	
Hydraulics						
Simple	x#	x			x#	
Complex		x	x	x		x
Empirical HGL Analysis				x		x
Solution of St Venant Eqns		x	x			
Energy Loss Estimates						
Colebrook-White		x		x		x
Mannings Equation	x		x	x	x	
Static Pit P.C. Coeff.		x		x		
Dynamic Pit P.C. Coeff.						x
Other Features						
Surcharge Allowed		x*	x*		x	x
Overflow Rerouting					x	x
Limited Inlet Capacity				x		x

P.C. - Pressure Change
H/H - Both hydrologic/hydraulic model
H - Primarily hydrological model

Notes: † total area only
‡ total and critical area
# pipe flow based on bedslope as the friction slope
* surcharge pools or is lost from the system

RAFTS, Goyen[22] while providing detailed hydrologic input to complex stormwater pipe and channel systems, covers only limited pipe hydraulics roughly equating to the TRANSPORT block in SWMM.

RATHGL is a Rational Formula based hydrologic model with extensive pipe hydraulic routines, Messner & Goyen, 1985[13]. The main features of the model include: network outlet (backwater) control, pit and channel surcharge facilities, surface flow rerouting, limited inlet capacity to pits, pipe and pit energy losses. Figures 5 and 6 indicate the general arrangement of RATHGL.

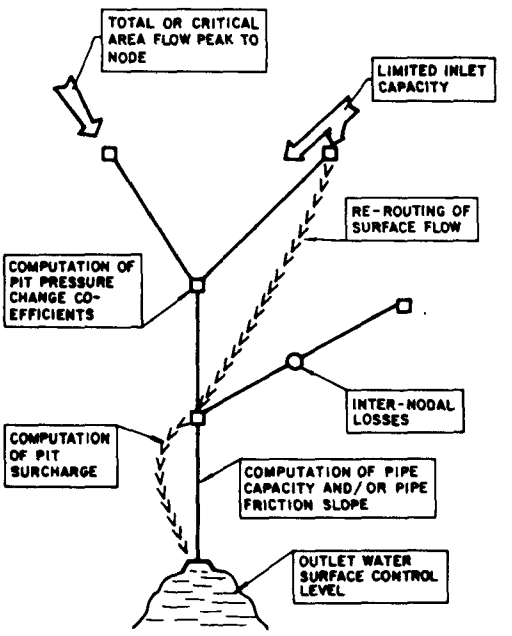

Figure 5. Typical network for RATHGL

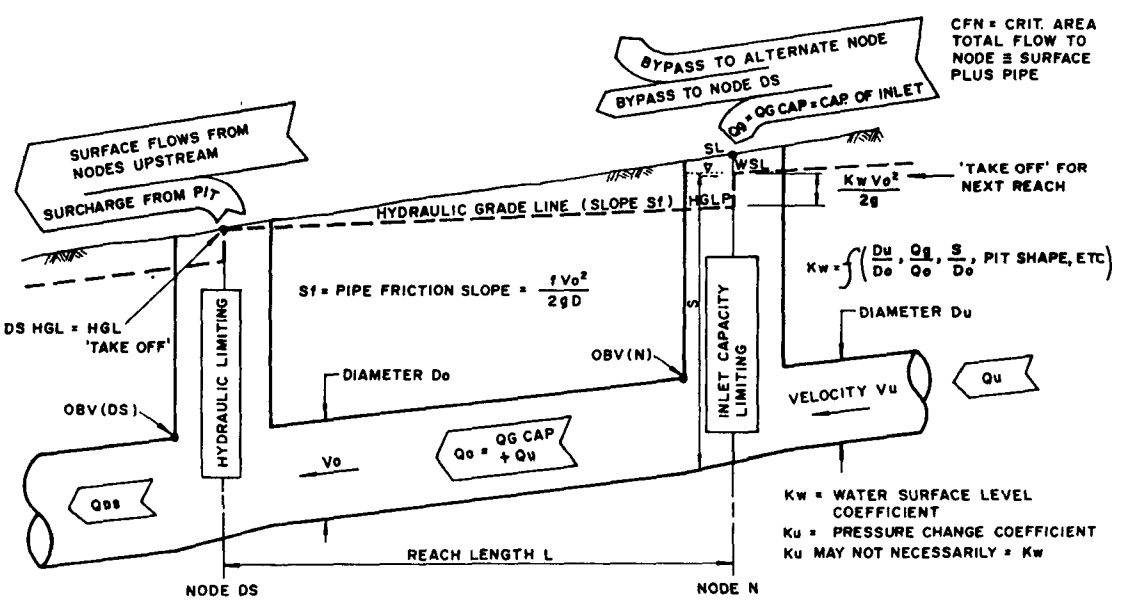

Figure 6. Typical RATHGL single reach from pipe network showing two flow limit states

To date the RATHGL type of model has gained acceptance over more complex models such as ILLUDAS and SWMM in routine pipe design and analysis. This is thought to be primarily due to their relationship with the familiar rational formula approach and their flexibility in handling a wide range of real like situations. Additionally, the hydraulic algorithms, although numerically complex, follow similar techniques to that previously carried out by hand, however with the advantage of accelerating the process of examining alternatives many fold.

MAJOR SYSTEMS

Once urban systems leave the piped network to join the major channel/floodway/river systems the use in Australia of rainfall/runoff routing models has held sway.

The most widely used models in Australia are RORB, Laurenson & Mein, 1983[23] and RAFTS, Goyen and Aitken, 1976[24], Goyen, 1983[22]. Both models consider watershed wide analysis involving streams and reservoirs or retention basins and allow the analysis and design of a wide range of flood mitigation options.

In RORB the model considers the whole catchment as a unit and describes internal concentrated storages related to a minimum of 5 to 20 internal subcatchments subdivided on watershed lines plus concentrated special storages to represent retention basins and additional stream routing effects.

All storage elements within the catchment are represented via the equation

$$S = 3600k\ Q^m$$

where k = represents a storage delay parameter and m represents a measure of the catchment's non-linearity.

When m is set equal to unity the catchment's routing is linear.

The storage parameter "k" within the general storage equation is modified to reflect not only the catchment storage but also the reach storage by the form:

$$k = kc.kr$$

where m is a measure of the catchment's non-linearity, and

kc is an empirical coefficient applicable to the entire catchment and stream network, and

kr is a dimensionless ratio called the relative delay time, applicable to an individual reach storage. kr thereby is modified to reflect the nature of the channel reach.

RORB has been used extensively throughout Australia on a range of rural and urban catchments. Calibrated values for kc and m for a large

number of regions have been developed throughout Australia which have been used to estimate flows on relatively ungauged catchments.

RSWM was originally developed in the early-1970s, Goyen & Aitken, 1976[24] jointly by Willing & Partners Pty Ltd and the Snowy Mountains Engineering Corporation. The Runoff Analysis and Flow Training Simulation (RAFTS) is a proprietry model developed from RSWM by Willing & Partners Pty Ltd, Goyen, 1983[22] to include separate routing of impervious and pervious areas, continuous loss modelling, pipe/channel analysis, detailed retention basin analysis including hydraulically interconnected schemes. The model is flexible in that, as well as handling small urban catchments, is equally comfortable with very large rural river basin analysis. RAFTS is further described diagrammatically in Figure 7.

Each subcatchment model is represented by a series of ten non-linear concentrated cascading storages based on the works of Laurenson, 1964[25].

Within RAFTS each of the subareas in a subcatchment is treated as a concentrated storage with a storage/discharge relation:

$$S = k(Q).Q$$

with $$K(Q) = B.Q^n$$

n and B represent the catchment non-linearity and subcatchment storage delay coefficient respectively and roughly equate in relative terms to RORB's m and k parameters.

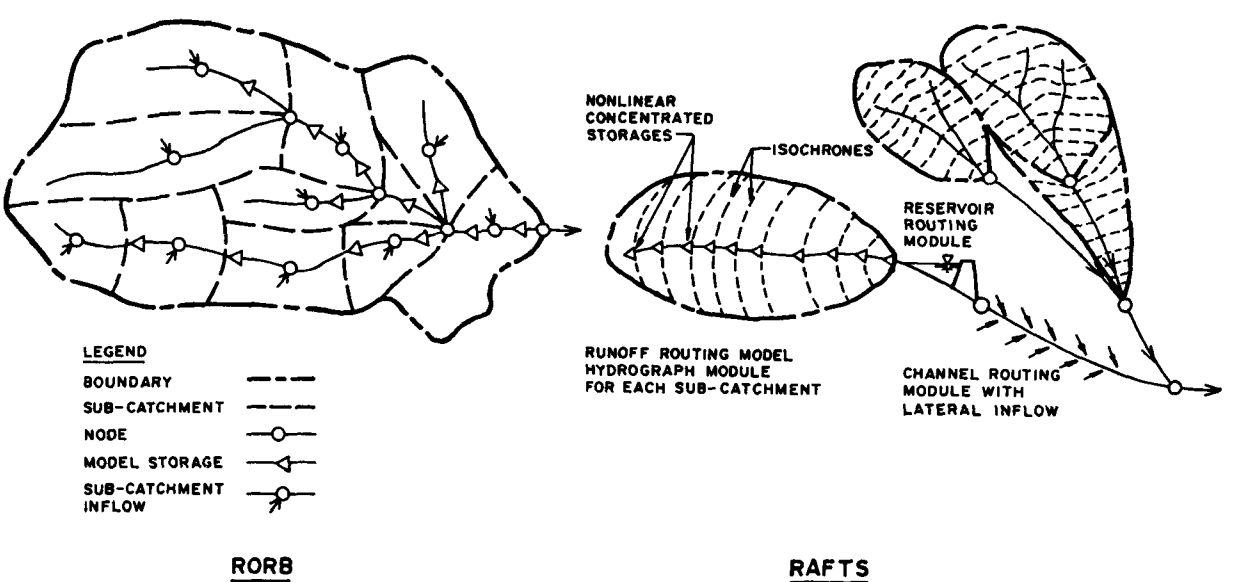

Figure 7. Diagrammatic Representation of RORB and RAFTS

The number of subcatchments used in RAFTS is not important as each individual subcatchment is represented by a complete model. RORB however requires a minimum of 5 to 20 subcatchments to represent a valid catchment model.

The RAFTS model incorporates more sophisticated loss routines than other Australian models. In addition to an initial loss/continuing loss rate option the model allows the use of the infiltration, wetting and redistribution algorithms of the Australian Representative Basins Model, Black & Aitken, 1977[26] and Goyen, 1983[22].

A further option that is provided with RAFTS is the SDLM Module which is a stochastic/deterministic loss model that links the probabilities of rainfall and soil moisture to estimate rainfall excess and runoff frequency curves without the need to use traditional loss modelling techniques, Goyen, 1983[22]. Figure 8 describes this option diagrammatically.

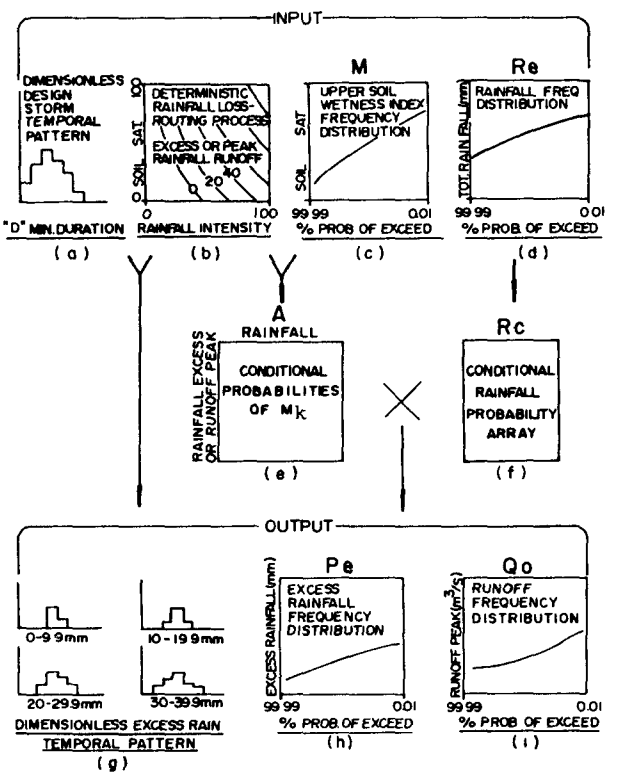

Figure 8. Diagrammatic Representation of the RAFTS/SDLM Module

RAFTS, unlike RORB, is more closely related to SWMM being a true network routing model with its basic element being the subcatchment providing input into the channel network system.

Subcatchment outflows from RAFTS are further routed through the channel network and retention basins by separate Muskingum-Cunge, Price, 1973[28] and level pool routing modules respectively. Separate routing of pipe flows under channels and retention basins is also provided.

In RAFTS n is set at a subcatchment level where m in RORB relates to the total catchment.

Similarly B in RAFTS is defined separately for each subcatchment to represent other than area considerations such as urbanisation, slope, catchment roughness, etc. k in RORB is defined on a catchment wide basis.

In RORB m is varied together with k to calibrate the catchment outflows to gauged data which means that the effects of both channel/stage and catchment storage over the entire catchment are reflected in the adopted k and m values.

RAFTS in contrast to RORB, which uses regionally derived k and m values for ungauged catchments, usually sets n equal to a constant (0.715). B is varied for each subcatchment based on measured characteristics and developed regression relationships where B is a function of (total area, slope, impervious area and surface roughness).

Over recent years research into catchment non-linearity, particularly in relation to large runoff events, has indicated that response becomes more linear with larger events, Bates & Pilgrim, 1983[27].

To allow for the rare event modelling, RAFTS allows a variable n relationship relative to subcatchment discharge/stage.

Varying linearity problems is likely to be of less a problem with RAFTS than RORB as only subcatchment routing is effected. In RORB the effects of linearising channel/storage with increasing flows is a significant factor in overall catchment routing as the combined effects have to be absorbed in the m value.

Provided subcatchments and consequential storages are kept relatively small compared to the effects of overall channel storage routing and reservoir storage the n value selected with RAFTS should be relatively insensitive to changes in flow regime.

WATER QUALITY

Modelling techniques to predict pollutant build up and wash off such as STORM, and SWMM, have only been used to a limited degree in Australia. The major modelling techniques have to date revolved around regression type algorithms to relate pollutant exports to daily runoff. The relationships shown in Table 5 have been based primarily on correlations with individual storm analysis for a range of monitored urban and rural catchments in Canberra.

In stream transfer models have, to date, been based around relatively simple gradually varying conservations of mass flow type techniques incorporating decay functions to account for loss in constituent mass flow with flow downstream.

Lake Response Models have mainly revolved around an adaptation of the Vollenweider Lake loading model, Lawrence & Goyen, 1987 to estimate the effects of eutrophication abatement programs.

TABLE 5. CORRELATION OF POLLUTANT EXPORTS WITH RUNOFF R (mm/day) (AFTER LAWRENCE & GOYEN, 1987)

Catchment Land Use	Coarse Sediment (kg/km^2)	Suspended Solids (kg/km^2)	Total Phosphorous (kg/km^2)	Total Nitrogen (kg/km^2)	E-Coli ($Count/km^2$)
Urban	$1\ 000\ R^{1.4}$	200 R	$0.39\ R^{0.8}$	$3.0\ R^{0.84}$	$30\ 000\ R^{0.9}$
Rural	$400\ R^{1.1}$	20 R	$0.115\ R^{0.57}$	$0.3\ R^{1.6}$	$500\ R^{0.9}$

TRENDS IN URBAN STORMWATER MANAGEMENT

WATER QUANTITY

In the last ten years there has been a significant improvement in catchment wide management techniques in Australia. New urban stormwater systems are now generally designed for the minor piped system together with a major surface flow floodway system. Additionally, future catchment development is now taken into account when planning and sizing downstream systems.

A number of regions now incorporate retention basins to maintain flow peaks at or below predevelopment levels. Unlike North American practice, however, retention basins are usually sized to optimise the attenuation of major flow peaks in the order of 50 to 100-year return period events. In general the size of basins are relatively large, typically 20 000 m^3 plus Mein, 1982[29] and few in number per watershed.

Predominately retention basins have only been implemented to reduce major flow peaks and water quality has not been a consideration.

The city of Canberra in the Australian Capital Territory representing a model city for trends in urban stormwater management has recently begun to incorporate wet basins to combine water quality and quantity aspects.

In recent years there has been an increasing trend to re-analyse older areas on a catchment basis to retrofit these to new area standards or as near as economically practical, Henkel & Goyen, 1980[30].

Management techniques have included the inclusion of retention basins in existing parks, mid catchment diversions, upgrading of pipe and pit systems and augmentation to channels and floodways.

In general the analysis techniques including models such as RATHGL, RORB and RAFTS have been applied to isolate particular weaknesses in the stormwater system. The degree of augmentation and the management options

selected have in recent times been based primarily on the acceptance criteria described previously.

WATER QUALITY

Possibly the most significant recent trend in a number of parts of Australia has been the inclusion of water quality elements into urban stormwater systems.

In Canberra, Australia's largest inland city, an integrated water quality/quantity approach is now in progress with seven water quality control ponds and three larger lakes providing both water quality and quantity control plus gross pollutant traps upstream of ponds and lakes either constructed or planned to trap urban litter, debris and sediment.

Similar water quality strategies are presently expanding to other areas in New South Wales in particular in areas showing specific stress from stormwater pollution.

Urban stormwater quality in Canberra has been approached with a four pronged strategy, Lawrence & Goyen, 1987[12], namely:

(a) the establishment of urban lakes, primarily as biological treatment systems,

(b) the utilisation of shallow ponds (water quality control ponds) and wetlands, as physical and biological treatment systems, upstream of urban lakes,

(c) the incorporation of gross pollutant traps on inlets to lakes or water quality control ponds to intercept trash and debris and the coarser fractions of sediment plus associated nutrient and other toxic constituents, and

(d) the incorporation of "off-stream" and "on-stream" sediment retention ponds into land development works to intercept and chemically treat runoff prior to its discharge to the stormwater system.

Additionally the Australian Capital Territory Water Pollution Ordinance was enacted in 1984 to control discharges to lakes, streams or stormwater systems. This Ordinance has provided an important enforcement mechanism during the construction phase of land development in particular.

DISCUSSION ON STORMWATER MANAGEMENT

Stormwater management in Australia as in other developed countries, is becoming extremely complex and now incorporates a wide range of constraints and social objectives not previously included. In a significant sense this has been made possible with the rapid advance in computing power available to professional engineers. It is the author's opinion that in the forthcoming decade one of the greatest challenges facing thre profession will be the marriage of this ever accelerating computational power with the knowledge and experience of those engineers having to make the complex engineering/social decisions based on computer predictions.

CONCLUSIONS

Urban stormwater management in Australia has progressed significantly since the late 1970s with integrated strategies generally now being enforced by state and local government authorities to ensure more controlled development on a catchment wide basis.

Modelling techniques, using modern computers, are now in widespread use in both government and private bodies allowing stormwater master planning to be carried out prior to development approvals plus detailed analysis of various drainage options prior to construction. This has allowed for more engineering and social issues to be investigated at the planning and design stage providing input to complex decision making processes which have significantly influenced stormwater management.

Detailed hydrologic and hydraulic simulation of existing urban pipe and channel networks has allowed the accurate isolation of system shortcomings and the formulation of appropriate management strategies for upgrading.

Water quality control has now taken on serious proportions in Australia with Canberra generally leading the way in control strategies and management techniques. It is expected that other sensitive regions in Australia will follow in Canberra's vein over the next few years.

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A NEW GROUNDWATER SUBROUTINE FOR THE STORM WATER MANAGEMENT MODEL

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ABSTRACT

Due to the importance of groundwater in the prediction of runoff, SWMM has been equipped with a groundwater subroutine to model the underlying water table. The subroutine models two zones -- an upper (unsaturated) zone and a lower (saturated) zone. Outflow from the upper zone to the lower zone is controlled by a percolation equation whose parameters can be calibrated or estimated from soils data. Loss from the unsaturated zone occurs through upper zone evapotranspiration; loss from the lower zone comes from both evapotranspiration and deep percolation. Groundwater flow from the lower zone is determined by a user-defined power function of water table stage and tailwater depth, and it can be routed to any previously defined inlet, trapezoidal channel, or pipe. (Pipes may be used to simulate under-drains.) Inflow to the subroutine is the infiltration calculated in subroutine WSHED. In the cases where the water table approaches the surface, infiltration that cannot be accepted by the soil is added back to the surface water component by means of a reduction in the variable RLOSS, the sum of infiltration and evaporation. Both printed and graphical output can be obtained for groundwater flow, water table stage, and moisture content in the unsaturated zone.

This paper has resulted from a project partially funded by the EPA, and it has been reviewed in accordance with the U.S. Environmental Protection Agency's peer and administrative review policies and approved for presentation and publication.

INTRODUCTION

Because the EPA Storm Water Management Model, SWMM (Huber et al., 1981) was originally developed to simulate combined sewer overflows in urban catchments, the fate of infiltrated water was considered insignificant. Since its development, however, SWMM has been used on areas ranging from highly urban to relatively undeveloped. Many of the undeveloped and even some of the developed areas, especially in areas like South Florida, are very flat with high water tables, and their primary drainage pathway is through the surficial groundwater aquifer and the unsaturated zone above it, rather than by overland flow. In these areas a storm will cause a rise in the water table and subsequent slow release of groundwater back to the receiving water (Capece et al., 1984). For this case, the fate of the infiltrated water is highly significant. By assuming that the infiltration is lost from the system, an important part of the high-water-table system is not being properly described (Gagliardo, 1986).

It is known that groundwater discharge accounts for the time-delayed recession curve that is prevalent in certain watersheds (Fetter, 1980). This process has not, however, been satisfactorily modeled by surface runoff methods alone. By modifying infiltration parameters to account for subsurface storage, attempts have been made to overcome the fact that SWMM assumes infiltration is lost from the system (Downs et al., 1986). Although the modeled and measured peak flows matched well, the volumes did not match well, and the values of the infiltration parameters were unrealistic. Some research on the nature of the soil storage capacity has been done in South Florida (SFWMD, 1984). However, it was directed towards determining an initial storage capacity for the start of a storm. There remains no standard, widely-used method for combining the groundwater discharge hydrograph with the surface runoff hydrograph and determining when the water table will rise to the surface. For instance, HSPF (Johansen et al., 1980) performs extensive subsurface moisture accounting and works well during average conditions. However, the model never permits the soil to become saturated so that no more infiltration is permitted, limiting its usefulness during times of surface saturation and flooding. Another difficulty with HSPF occurs during drought conditions, since there is no threshold saturated zone water storage (corresponding to the bottom of a stream channel) below which no saturated zone outflow will occur. These difficulties have limited HSPF usefulness for application to extreme hydrologic conditions in Florida (Heaney et al., 1986).

In order to incorporate subsurface processes into the simulation of a watershed and overcome previously mentioned shortcomings, SWMM has been equipped with a simple groundwater subroutine. The remainder of this paper will describe the theory, use, and some limitations of the subroutine.

THEORY

INTRODUCTION

An effort was made to utilize existing theoretical formulations for as many processes as possible. The purpose was to maintain semblance to the real

world while enabling the user to determine parameter values that have meaning to the soil scientist. Also, in the following discussion the term "flow" will refer to water that is passed on to another part of the system, and the term "loss" will refer to water that is passed out of the system. In addition, in the groundwater subroutines, flows and losses have internal units of velocity (flow per unit area).

The groundwater subroutine, GROUND, simulates two zones -- an upper (unsaturated) zone and a lower (saturated) zone. This configuration is similar to the work done by Dawdy and O'Donnell (1965) for the USGS. The flow from the unsaturated to the saturated zone is controlled by a percolation equation for which parameters may either be estimated or calibrated, depending on the availability of the necessary soil data. Upper zone evapotranspiration is the only loss from the unsaturated zone. The only inflow to subroutine GROUND is the calculated infiltration from subroutine WSHED. Losses and outflow from the lower zone can be via deep percolation, saturated zone evapotranspiration, and groundwater flow. Groundwater flow is a user-defined power function of water table stage and, if chosen, depth of water in the discharge channel.

The physical processes occurring within each zone are accounted for by individual mass balances in order to determine end-of-time-step stage, groundwater flow, deep percolation, and upper zone moisture. Parameters are shown in Figure 1 and defined below. Mass balance in the upper (unsaturated) zone is given by,

$$TH2 = \{[(ENFIL-ETU)*PAREA-PERC]*DELT+(D1-D2)*TH2+TH*DWT1\}/(DTOT-D2) \quad (1)$$

In the lower (saturated) zone, for rising water tables,

$$D2 = \{[PERC-ETD*PAREA-.5*(GWFLW+A1*(D2-BO)^{B1}+A3*D2*TA+DEPPRC+DP*D2/DTOT) -TWFLW]*DELT+(D2-D1)*(TH-TH2)\}/(PR-TH2) + D1 \quad (2)$$

and for falling water tables,

$$D2 = \{[PERC-ETD*PAREA-.5*(GWFLW+A1*(D2-BO)^{B1}+A3*D2*TA+DEPPRC+DP*D2/DTOT) -TWFLW]*DELT\}/(PR-TH2) + D1 \quad (3)$$

where
- TH2 = end-of-time-step upper zone moisture content (fraction),
- ENFIL = infiltration rate calculated in subroutine WSHED,
- ETU = upper zone evapotranspiration rate,
- PERC = percolation rate,
- PAREA = pervious area divided by total area,
- DELT = time step value,
- D1 = beginning-of-time-step lower zone depth (elevation above a datum),
- D2 = end-of-time-step lower zone depth,
- TH = beginning-of-time-step upper zone moisture content,
- DWT1 = beginning-of-time-step upper zone depth,
- DTOT = total depth of upper and lower zone = D1+DWT1,
- ETD = lower zone evapotranspiration rate,
- GWFLW = beginning-of-time-step groundwater flow rate,

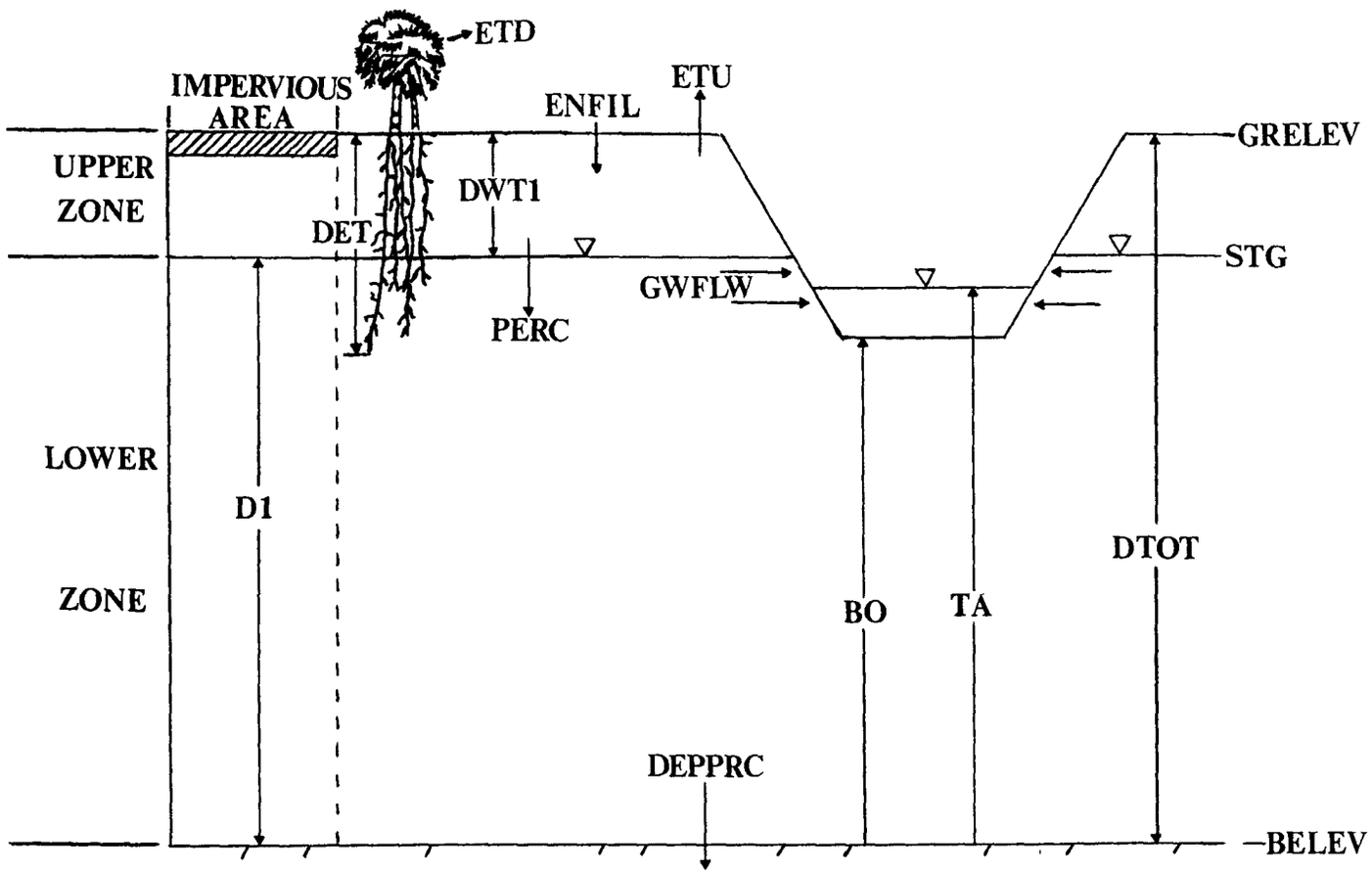


Figure 1. GROUND parameters and conceptualization.

A1 = groundwater flow coefficient,
BO = bottom of channel depth (elevation above datum),
B1 = groundwater flow exponent,
DEPPRC = beginning-of-time-step deep percolation rate,
DP = a recession coefficient derived from interevent declines in the water table,
PR = porosity, and
TWFLW = channel water influence rate,
A3 = groundwater flow coefficient, and
TA = depth of water in channel (elevation above datum).

Solving equation 1 for TH2 and using DWT1 = DTOT-D1, yields a much simpler form which is not a function of the unknown D2,

$$TH2 = [(ENFIL-ETU)*PAREA-PERC]*DELT/DWT1 + TH \quad (4)$$

Equation 4 is solved first, followed by a Newton-Raphson solution of equation 2 or 3. The sequencing will be described in more detail in a subsequent section, following a description of the various simulated processes.

UPPER ZONE ET

Evapotranspiration from the upper zone (ETU) represents soil moisture lost via cover vegetation and by direct evaporation from the pervious area of the subcatchment. No effort was made to derive a complex formulation of this process. The hierarchy of losses by evapotranspiration is as follows: 1) surface evaporation, 2) upper zone evapotranspiration, and 3) lower zone transpiration. Upper zone evapotranspiration is represented by the following equations,

$$ETMAX = VAP(MONTH) \quad (5)$$
$$ETAVLB = ETMAX-EVAPO \quad (6)$$
$$ETU = CET*ETMAX \quad (7)$$
$$IF(TH.LT.WL.OR.ENFIL.GT.0.)\ ETU = 0. \quad (8)$$
$$IF(ETU.GT.ETAVLB)\ ETU = ETAVLB \quad (9)$$

where ETMAX = maximum total evapotranspiration rate (input on card F1),
VAP(MONTH) = input maximum evapotranspiration rate for month MONTH,
ETAVLB = maximum upper zone evapotranspiration rate,
EVAPO = portion of ETMAX used by surface water evaporation,
CET = fraction of evapotranspiration apportioned to upper zone, and
WL = wilting point of soil.

The two conditions that make ETU equal to zero in equation 8 are believed to simulate the processes actually occurring in the natural system. The first condition (moisture content less than wilting point) relates to the soil science interpretation of wilting point -- the point at which plants can no longer extract moisture from the soil. The second condition (infiltration greater than zero) assumes that vapor pressure will be high enough to prevent additional evapotranspiration from the unsaturated zone.

INFILTRATION

Infiltration enters subroutine GROUND as the calculated infiltration from subroutine WSHED. As before in SWMM, either the Horton or Green-Ampt equation can be used to describe infiltration. For time steps where the water table has risen to the surface, the amount of infiltration that cannot be accepted is subtracted from RLOSS (infiltration plus surface evaporation) in subroutine WSHED. In the event that the infiltrated water is greater than the amount of storage available for that time step, the following equation is used to calculate the amount of infiltration that is not able to be accepted by the soil.

XSINFL = ENFIL*DELT-AVLVOL/PAREA (10)

where XSINFL = excess infiltration over pervious area, and
AVLVOL = initial void volume in the upper zone plus total losses and outflows from the system for the time step.

The second condition exists because of the algebra in equations 2, 3 and 4. As the water table approaches the surface, the end-of-time-step moisture value, TH2, approaches the value of porosity, which makes the denominator in equations 2 and 3 go towards zero. Since a denominator close to zero could result in an unrealistic value of D2, a different way of handling the calculations had to be implemented. When the initial available volume in the upper zone plus the volume of total outflows and losses from the system minus the infiltration volume is between zero and an arbitrary value of 0.0001 ft, several assumptions are made. First, end-of-time-step groundwater flow and deep percolation, which are normally found by iteration, are assumed to be equal to their respective beginning-of-time-step values. This step is taken to ensure that the final available volume remains in the previously mentioned range. Second, TH2 is set equal to an arbitrary value of 90% of porosity. It is believed that this will allow the TH2 value in this special case to be reasonably consistent with the TH2 values juxtaposed to it in the time series. Third, D2 is set close to the total depth -- the actual value of D2 depends on the value of porosity. Fourth, the amount of infiltration that causes the final available volume to exceed 0.0001 ft is calculated in the following equation and sent back to the surface in the form of a reduction in the term RLOSS in subroutine WSHED.

XSINFL = ENFIL*DELT+(.0001-AVLVOL)/PAREA (11)

Because of the way this special case is handled, it is possible for a falling water table to have the calculated excess infiltration be greater than the actual amount of infiltration. It is not desirable for the ground to pump water back onto the surface! Hence, the difference between the calculated excess infiltration and the actual infiltration is added to the infiltration value of the next time step. The number of occurrences of this situation in a typical run is very small, as is the computed difference that is passed to the next time step, so no problems should occur because of this solution.

LOWER ZONE EVAPOTRANSPIRATION

Lower zone evapotranspiration, ETD, represents evapotranspiration from the saturated zone over the pervious area. ETD is the last evapotranspiration removed, and is determined by the following depth-dependent equation and conditions.

```
ETD = (DET-DWT1)*ETMAX*(1-CET)/DET                          (12)
IF(ETD.GT.(ETAVLB-ETU)) ETD = ETAVLB-ETU                    (13)
IF(ETD.LT.0.) ETD = 0.                                      (14)
```

where ETD = lower zone evapotranspiration rate, and
DET = depth over which evapotranspiration can occur.

Since ETD is typically very small compared to other terms and has to be checked for certain conditions, it is assumed constant over the time step and not solved for in the iterative process.

PERCOLATION

Percolation (PERC) represents the flow of water from the unsaturated zone to the saturated zone, and is the only inflow for the saturated zone. The percolation equation in the subroutine was formulated from Darcy's Law for unsaturated flow, in which the hydraulic conductivity, K, is a function of the moisture content, TH. For one-dimensional, vertical flow, Darcy's Law may be written

$$v = -K(TH)\ dh/dz \tag{15}$$

where v = velocity (specific discharge) in the direction of z,
z = vertical coordinate, positive upward,
K(TH) = hydraulic conductivity,
TH = moisture content, and
h = hydraulic potential.

The hydraulic potential is the sum of the elevation (gravity) and pressure heads,

$$h = z + PSI \tag{16}$$

where PSI = soil water tension (negative pressure head) in the unsaturated zone.

Equating vertical velocity to percolation, and differentiating the hydraulic potential, h, yields

$$Percolation = -K(TH)*(1+ dPSI/dz) \tag{17}$$

A choice is customarily made between using the tension, PSI, or the moisture content, TH, as parameters in equations for unsaturated zone water flow. Since the quantity of water in the unsaturated zone is identified by TH in

previous equations, it is the choice here. PSI can be related to TH if the characteristics of the unsaturated soil are known. Thus, for use in equation 17, the derivative is

$$dPSI/dz = dPSI/dTH * dTH/dz \tag{18}$$

The slope of the PSI versus TH curve should be obtained from data for the particular soil under consideration. Relationships for a sand, sandy loam and silty loam are shown in Figures 2, 3 and 4 (Laliberte et al., 1966). The data are based on laboratory tests of disturbed soil samples and illustrate only the desaturation (draining) characteristics of the soil. The relationship during the saturation (wetting) phase will ordinarily be different; when both the wetting and draining relationships are shown the curves usually illustrate a hysteresis effect. The figures also show the relationship between the hydraulic conductivity of the unsaturated soils and the moisture content. In some cases (e.g., sand), K(TH) may range through several orders of magnitude. Soils data of this type are becoming more readily available; for example, soil science departments at universities often publish such information (e.g., Carlisle et al., 1981). The data illustrated in Figures 2, 3 and 4 are also useful for extraction of parameters for the Green-Ampt infiltration equations.

Equation 17 may be approximated by finite differences as

$$Percolation = -K(TH)*[1+(\Delta TH/\Delta z)*(\Delta PSI/\Delta TH)] \tag{19}$$

For calculation of percolation, it is assumed that the gradient, $\Delta TH/\Delta z$, is the difference between moisture content TH in the upper zone and field capacity at the boundary with the lower zone, divided by the average depth of the upper zone, DWT1/2. Thus,

$$Percolation = -K(TH)*\{1+[(TH-FD)*2/DWT1]*PCO\} \tag{20}$$

where FD = field capacity, and
PCO = $\Delta PSI/\Delta TH$ in the region between TH and FD.

PCO is obtained from data of the type of Figures 2, 3 and 4.

Finally, the hydraulic conductivity as a function of moisture content is approximated functionally in the moisture zone of interest as

$$K(TH) = HKTH = HKSAT*EXP[(TH-PR)*HCO] \tag{21}$$

where HKTH = hydraulic conductivity as a function of moisture content,
HKSAT = saturated hydraulic conductivity, and
HCO = calibration parameter.

HCO can be estimated by fitting the HKTH versus TH curve to the hydraulic conductivity versus moisture content curve, if such data are available (e.g., Figures 2, 3, 4); three fits are shown in Figure 5. The fits are not optimal over the entire data range because the fit is only performed for the high moisture content region between field capacity and porosity. If soils data

Touchet Silt Loam

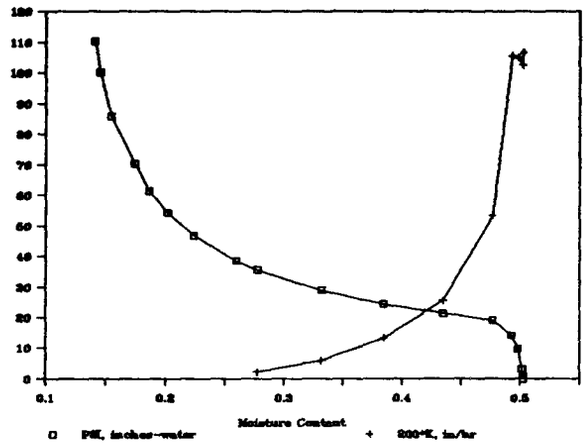


Figure 2. Tension, PSI (squares, in. of water) and hydraulic conductivity, K (crosses, in/hr, K multiplied by 200) versus moisture content. Derived from data of Laliberte et al (1966), Tables B-5 and C-3. Porosity = 0.503, temp. = 26.5 °C, saturated hyd. conductivity = 0.53 in/hr.

Columbia Sandy Loam

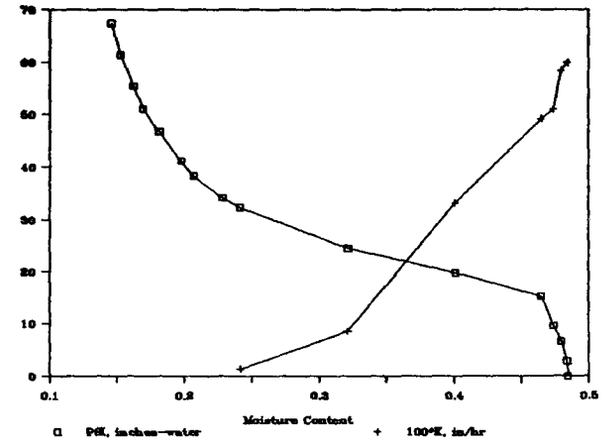


Figure 3. Tension, PSI (squares, in. of water) and hydraulic conductivity, K (crosses, in/hr, K multiplied by 100) versus moisture content. Derived from data of Laliberte et al. (1966), Tables B-8 and C-5. Porosity = 0.485, temp. = 25.1 °C, saturated hyd. conductivity = 0.60 in/hr.

Unconsolidated Sand

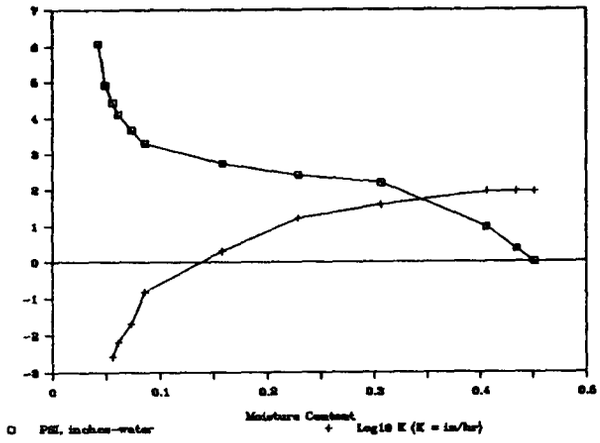


Figure 4. Tension, PSI (squares, in. of water) and log-10 of hydraulic conductivity, K (crosses, K in in/hr) versus moisture content. Derived from data of Laliberte et al. (1966), Tables B-14 and C-11. Porosity = 0.452, temp. = 25.1 °C, saturated hyd. conductivity = 91.5 in/hr.

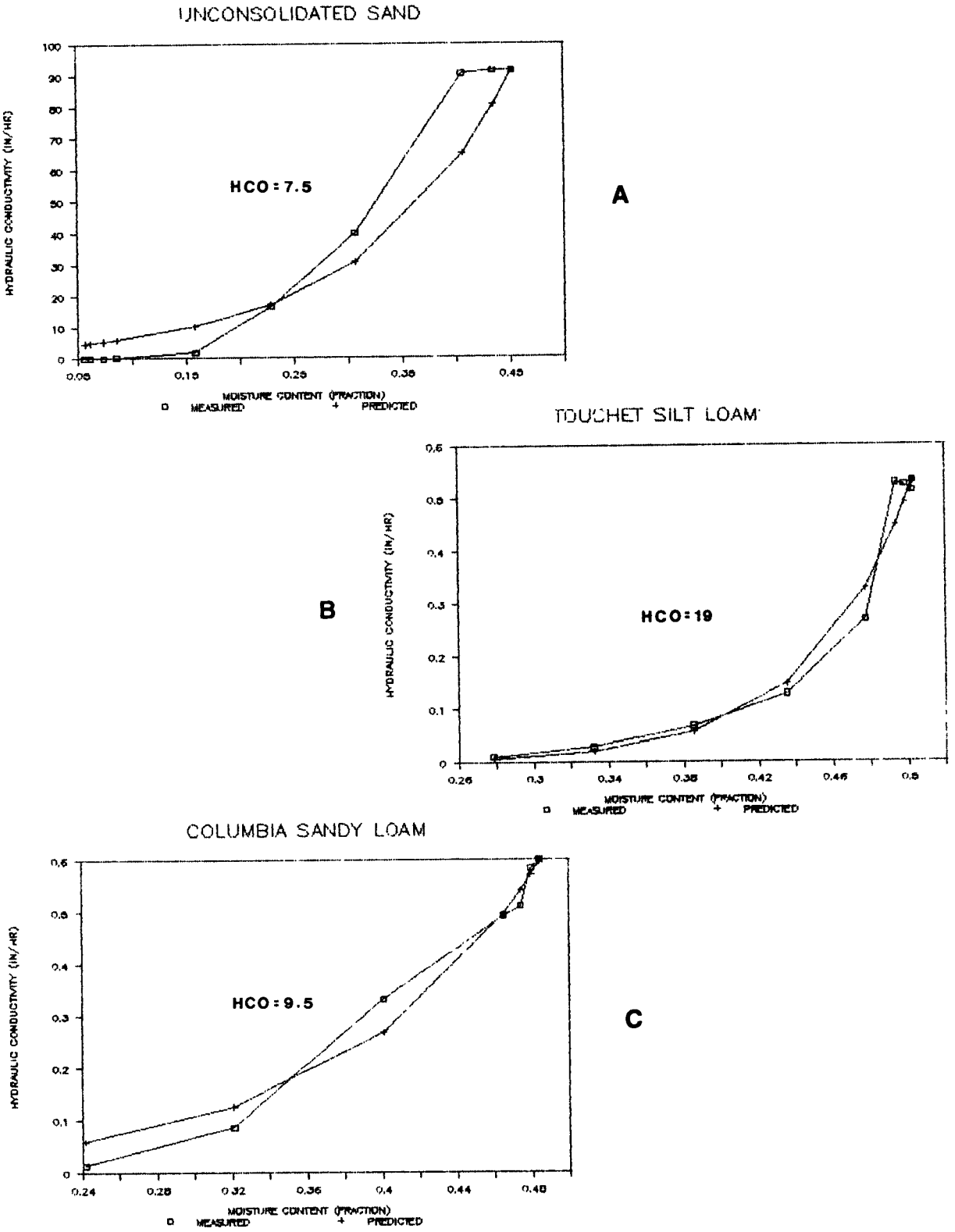


Figure 5. Model representation of and measured hydraulic conductivity curves for three types of soil.

are not available, HCO can be estimated by model calibration.

Combining equations 20 and 21 gives the resulting percolation equation for the model,

$$PERC = HKTH*[1+PCO*(TH-FD)/(DWT1/2)] \quad (22)$$

where PERC = percolation rate (positive downward) and is only nonzero when TH is greater than FD.

If data sources for parameters PCO and HCO are lacking, they may be estimated through the calibration process. On the basis of preliminary runs, the groundwater subroutine is relatively insensitive to changes in PCO and HCO, so a lack of extensive soils data should not discourage one from using the model.

If moisture content is less than or equal to field capacity, percolation becomes zero. This limit is in accordance with the concept of field capacity as the drainable soil water that cannot be removed by gravity, alone (Hillel, 1982, p. 243). Once TH drops below field capacity, it can only be further reduced by upper zone evapotranspiration.

The percolation rate calculated by equation 22 will be reduced by the program if it is high enough to drain the upper zone below field capacity or make the iterations for D2 converge to an unallowable value. Also, since checks must be made on PERC, it is assumed to be constant over the time step and therefore not determined through an iterative process.

DEEP PERCOLATION

Deep percolation represents a lumped sink term for unquantified losses from the saturated zone. The two primary losses are assumed to be percolation through the confining layer and lateral outflow to somewhere other than the receiving water. The arbitrarily chosen equation for deep percolation is

$$DEPPRC = DP*D1/DTOT \quad (23)$$

where DEPPRC = beginning-of-time-step deep percolation rate, and
DP = a recession coefficient derived from interevent water table recession curves.

The ratio of D1 to DTOT allows DEPPRC to be a function of the static pressure head above the confining layer. Although DEPPRC will be very small in most cases, it is included in the iterative process so that an average over the time step can be used. By doing this, large continuity errors will be avoided should DEPPRC be set at a larger value.

GROUNDWATER DISCHARGE

Functional Form

Groundwater discharge represents lateral flow from the saturated zone to the receiving water. The flow equation takes on the following general form:

$$GWFLW = A1*(D1-BO)^{B1} - TWFLW + A3*D1*TA \quad (24)$$

and

$$TWFLW = A2*(TA-BO)^{B2} \quad (25)$$

where GWFLW = beginning-of-time-step groundwater flow rate (per subcatchment area,
TWFLW = channel water influence flow rate (per subcatchment area),
A1,A2 = groundwater and channel water influence flow coefficients,
A3 = coefficient for cross-product,
B1,B2 = groundwater and tailwater influence flow exponents,
BO = elevation of bottom of channel, and
TA = elevation of water in channel.

If D1 is less than BO or TA, GWFLW is set equal to zero. In addition, if TA = BO and B2 = 0, then the indeterminant form of zero raised to the zero power in equation 25 is set equal to 1.0 by the program. The functional form of equations 24 and 25 was selected in order to be able to approximate various horizontal flow conditions, as will be illustrated below.

Since groundwater flow can be a significant volume, an average flow each time step is found by iteration using equation 2 or 3. Groundwater flows can be routed to any previously defined inlet, trapezoidal channel, or pipe, allowing the user to isolate the various components of the total hydrograph, as shown in Figure 6. That is, the groundwater flow does not have to be routed to the same destination as the overland flow from the subcatchment.

The effects of channel water on groundwater flow can be dealt with in two different manners. The first option entails setting TA (elevation of water surface in the channel) to a constant value greater than or equal to BO (bottom-of-channel elevation) and A2, B2 and/or A3 to values greater than zero. If this method is chosen, then the user is specifying an average tailwater influence over the entire run to be used at each time step.

The second option makes the channel water elevation, TA, equal to the elevation of water in an actual channel (trapezoidal channel or circular pipe). For this option, the groundwater must be routed to a trapezoidal channel or pipe -- not an inlet. The depth of water in the channel (TA - BO) at each time step is then determined as the depth in the channel or pipe from the previous time step. (It is assumed that the bottom of the channel is at the elevation BO.) The beginning-of-time-step depth must be used to avoid complex and time-consuming iterations with the coupled channel discharge equations in subroutine GUTTER. Unfortunately, because of this compromise the groundwater flow may pulsate as D1 oscillates between just above and just below elevation TA. This pulsing may introduce errors in continuity and is, of course, unrep-

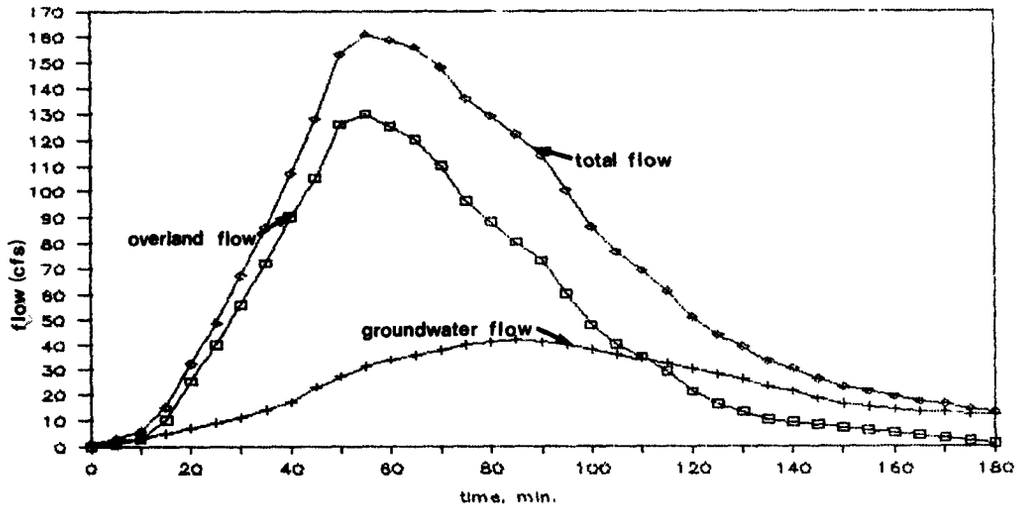


Figure 6. Hydrograph of total flow and its two major components.

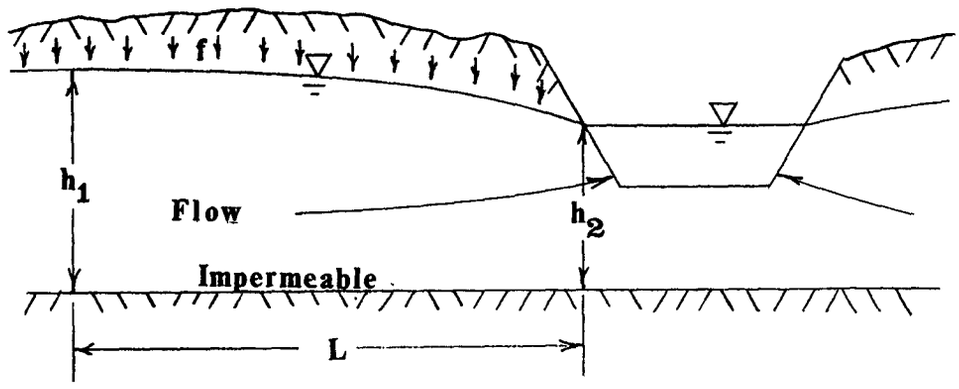


Figure 7. Definition sketch for Dupuit-Forcheimer approximation for drainage to adjacent channel.

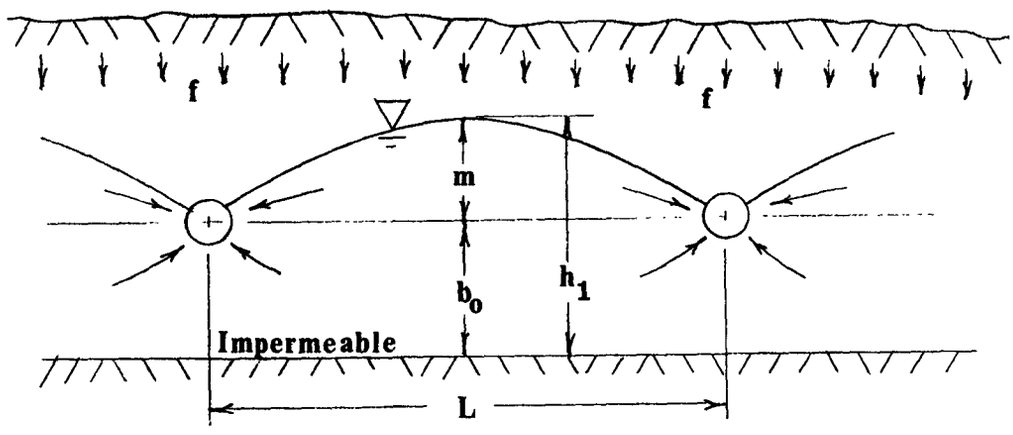


Figure 8. Definition sketch for Hooghoudt's method for flow to circular drains.

resentative of the actual system. Shorter time steps and larger or less steep channels (reducing the response of the channel) can be used to reduce the pulses. Also, caution must be taken when selecting A1, B1, A2, B2 and A3 so that GWFLW cannot be negative. Although this may occur in the actual system and represent recharge from the channel, there is currently no means of representing this reverse flow and subtracting it from the channel. One way of assuring that this cannot happen is to make A1 greater than or equal to A2 and B1 greater than or equal to B2, and A3 equal to zero.

Because of the general nature of the equation, it can take on a variety of functional forms. For example, a linear reservoir can be selected by setting B1 equal to one and A2 and A3 equal to zero. Two drainage examples are illustrated below.

Example: Infiltration and Drainage to Adjacent Channel

Under the assumption of uniform infiltration and horizontal flow by the Dupuit-Forcheimer approximation, the relationship between water table elevation and infiltration for the configuration shown in Figure 7 is (Bouwer, 1978, p. 51)

$$K(h_1^2 - h_2^2) = L^2 f \tag{26}$$

where f = infiltration rate,
K = hydraulic conductivity, and other parameters are as shown on Figure 7.

Before matching coefficients of equations 24 and 25 to equation 26, it should be recognized that the water table elevation in SWMM, D1, represents an average over the catchment, not the maximum at the "upstream" end that is needed for h_1 in equation 26. Let D1 be the average head,

$$D1 = (h_1 + h_2)/2 \tag{27}$$

Substituting $h_1 = 2\ D1 - h_2$ into equation 26 gives, after algebra

$$(D1^2 - D1\ h_2)\ 4K/L^2 = f \tag{28}$$

from which a comparison with equations 24 and 25 yields $A1 = A3 = 4K/L^2$, A2 = 0, and B1 = 2. Note that GWFLW has units of flow per unit area, or length per time, which are the units of infiltration, f, in equation 28.

Example: Hooghoudt's Equation for Tile Drainage

The geometry of a tile drainage installation is illustrated in Figure 8. Hooghoudt's relationship (Bouwer, 1978, p. 295) among the indicated parameters is

$$f = (2D_e + m)\ 4Km/L^2 \tag{29}$$

where D_e = effective depth of impermeable layer below drain center, and other

parameters are defined in Figure 8. D_e is less than or equal to b_o in Figure 8 and is a function of b_o, drain diameter, and drain spacing, L; the complicated relationship is given by Bear (1972, p. 412) and graphed by Bouwer (1978, p. 296). The maximum rise of the water table, $m = h_1 - b_o$. Once again approximating the average water table depth above the impermeable layer by $D1 = 2h_1 - b_o$, equation 29 can be manipulated to

$$f = [(h_1 - b_o)^2 + 2D_e(h_1-b_o)]\ 4K/L^2 =$$

$$= [(D1-b_o)^2 + D_eD1 - D_eb_o]\ 16K/L^2 \qquad (30)$$

Comparing equation 30 with equations 24 and 25 yields

$A1 = 16K/L^2$,

$B1 = 2$

$A2 = 16KD_eb_o/L^2$

$B2 = 0$

$A3 = 16KD_e\ /\ TA\ L^2$

and $TA = BO = b_o$ = constant during the simulation. The equivalent depth, D_e, must be obtained from the sources indicated above. The mathematics of drainage to ditches or circular drains is complex; several alternative formulations are described by van Schilfgaarde (1974).

LIMITATIONS

Since the moisture content of the unsaturated zone is taken as an average over the entire zone, the shape of the moisture profile is totally obscured. Therefore, infiltrated water cannot be modeled as a diffusing slug moving down the unsaturated zone, as is the case in the real system. Furthermore, water from the capillary fringe of the saturated zone cannot move upward by diffusion or "suction" into the unsaturated zone.

The simplistic representation of subsurface storage by one unsaturated "tank" and one saturated "tank" limits the ability of the user to match nonuniform soil columns. Another limitation is the assumption that the infiltrated water is spread uniformly over the entire catchment area, not just over the pervious area. In addition, just as for surface flow, groundwater may not be routed from one subcatchment to another. The tendency of the tailwater influence to cause pulses if TA-BO is equated to the dynamic water depth in the adjacent channel is a limitation that will remain until the channel flow and subsurface flow are solved simultaneously using a set of coupled equations. Such a solution would also permit reverse flow or recharge from the channel to be simulated.

Finally, water quality is not simulated in any of the subsurface routines. If water quality is simulated in RUNOFF and the subsurface flow rou-

tines activated, any loads entering the soil will "disappear," as if the soil provides 100 percent treatment.

SUBROUTINE CONFIGURATION

A flowchart of the subroutine configuration is presented in Figure 9. Initial values and constants used in subroutine GROUND come mostly from subroutine GRIN, designed specifically to read in these values. Subroutine GRIN is called by RHYDRO. Other necessary values are transferred during the CALL statement and from previously calculated values stored in COMMON.

Subroutine GROUND first initializes pertinent parameters, then calculates fluxes that are constant over the time step. Beginning-of-time-step fluxes are calculated next, and the value of percolation is checked to ensure that it will not raise the water table above the ground surface.

After other constants are calculated and TH2 is determined from equation 4, the program branches to one of four areas, as indicated in Figure 9. The first and second areas are for rising and falling water tables, equations 2 and 3, respectively. In both cases, Newton-Raphson iteration is used to solve simultaneously for the final groundwater flow, depth of lower zone, and deep percolation. Each iteration checks whether or not groundwater flow is possible (D1 greater than or equal to TA and BO). After the iterations converge, final conditions are set as the next time step's initial conditions.

In the event of saturation (D1 = DTOT), the third area sets D2 equal to DTOT, sets final groundwater flow equal to the maximum possible (D2 = DTOT), and assumes DEPPRC remains constant over the time step. Any excess infiltration is then routed back to the surface for overland flow calculations, and final conditions are set for the next initial conditions. However, if the maximum groundwater flow and DEPPRC rates permit some infiltration into the subsurface zone, the initial and final groundwater flow are averaged to be used as the new initial groundwater flow, and the program branches back to iterate for the solution. This pathway will rarely, if ever, be taken, but must be included to minimize possible continuity errors.

In the event the available storage in the unsaturated zone is less than 0.0001 ft, the fourth area sets TH2 equal to 90% of porosity and D2 close to DTOT, and returns any infiltration to the surface that causes the final unfilled upper zone volume to be greater than 0.0001 ft. This is to avoid oscillations as the water table hovers near the ground surface. Again, final conditions are then set as the next time step's initial conditions.

EXAMPLE RUNS

CYPRESS CREEK CALIBRATION AND VERIFICATION

Two examples will illustrate the use of the new subroutine. The first example is a year-long simulation of a 47 mi^2 portion of the 117 mi^2 Cypress Creek Watershed in Pasco County, Florida, about 30 miles north of Tampa (Figure 10). The region has been studied in relation to the interaction of sur-

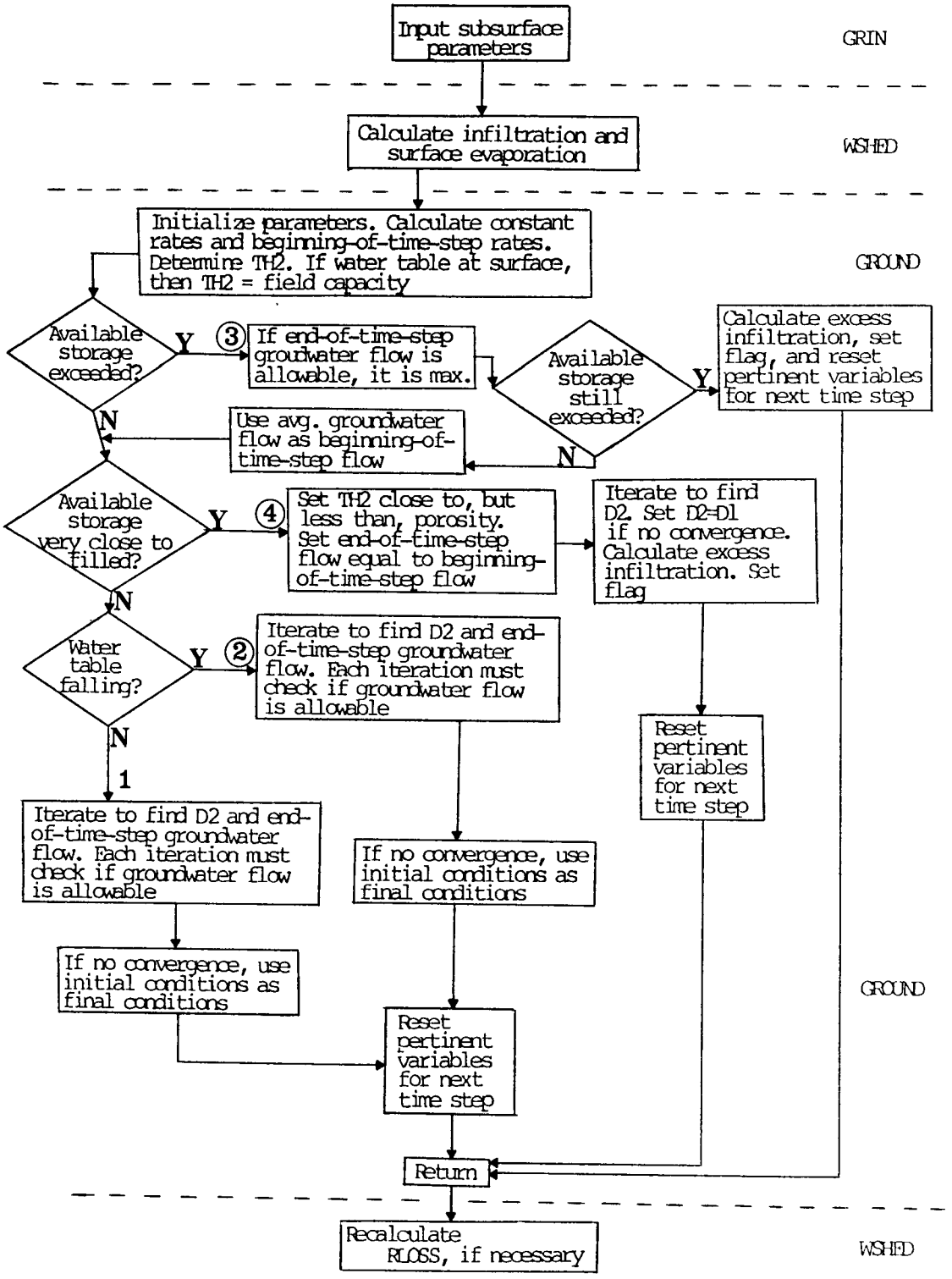


Figure 9. Flowchart of subsurface and directly-connected surface calculations.

face water and ground water under the stress of heavy pumping and drainage activities in the area (Heaney et al., 1986). The watershed is characterized by sandy soils in which most water movement follows subsurface pathways. For this example, only a single 47 mi^2 area above State Road 52 (Figure 10) and tributary to the USGS gage at San Antonio has been simulated.

Twenty-four parameters on three additional H-cards are required for each subsurface subcatchment. (Many of these can be ignored or set to zero during most runs; not all parameters are required for all runs.) Input parameters are echoed on two new pages of output that immediately follow the surface subcatchment information. Figure 11 is an example of these two new pages; the values in Figure 11 are from the calibration run on Cypress Creek. In addition to the new output just mentioned, a subsurface continuity check is provided in addition to the existing surface continuity check. An example of this amended page is shown in Figure 12.

The simulation is divided into two six-month runs: the first six months for calibration, and the second six months for verification. Since Cypress Creek is a very flat, pervious area with well-drained soils and very little surface flow, it was modeled in a manner that would allow groundwater flow to account for most of the flow in the channel. In other words, the groundwater parameters represented by far the most critical part of the calibration. The only complete rainfall data for the calibration period are for the gage at St. Leo, out of the catchment to the east. Although these data are in daily increments, the calibration process was relatively simple because of the existence of both flow and shallow-well stage data. In addition, only one subcatchment (surface and subsurface) was used, since the purpose of this example was only to illustrate the use of subroutine GROUND, not to provide a thorough simulation.

Figure 13 shows the predicted groundwater flow hydrograph and the measured total flow hydrograph for the calibration run, and Figure 14 shows a comparison of the predicted total flow hydrograph to the measured total flow hydrograph for the calibration run. Predicted and measured stages for the calibration and verification can be seen in Figures 15 and 16. The calibration is not especially remarkable in light of the lack of detailed rainfall data for the 47 mi^2 area. The predicted stage hydrograph does not exhibit the short-term variations that are measured, primarily because of the lack of spatial detail in the rain. In addition, the measured stages are at one well near the center of the modeled area and would be expected to show more variation than would the average water table over the 47 mi^2 simulated by SWMM. The existence of more than one gage in the 47 square miles of the catchment and shorter increment rainfall data would have improved the fit seen in Figures 15 and 16. Figures 17 and 18 show flow results for the verification runs. In general, the average recession of the water table is simulated accurately, but not the fluctuations.

HYPOTHETICAL CATCHMENT WITH HIGH WATER TABLE

The second example is a 100 ac hypothetical subcatchment with the same soil properties as Cypress Creek and a water table that is initially one foot

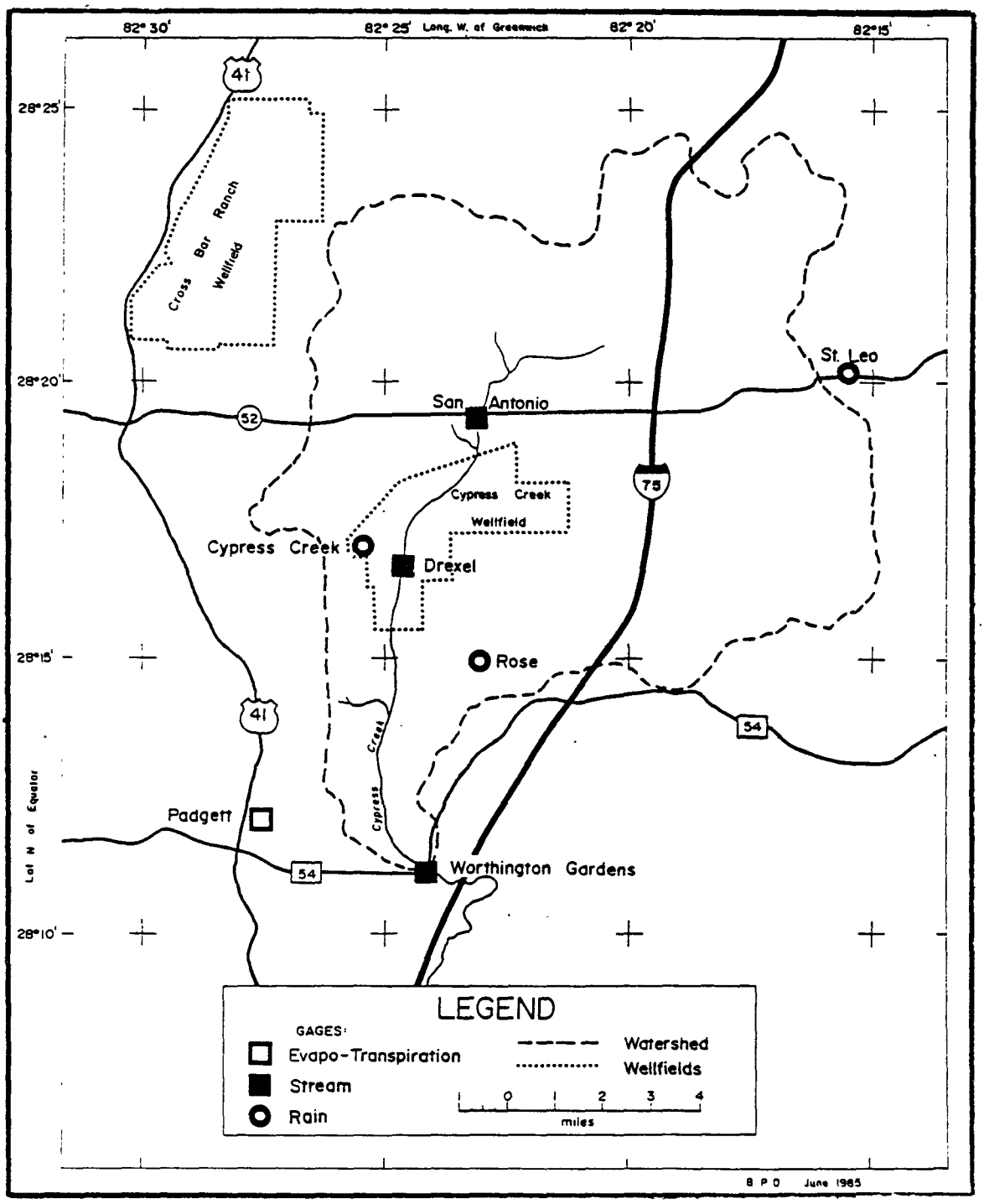


Figure 10. Map of Cypress Creek Watershed in Pasco County, Florida. (Heaney et al., 1986)

```
* * * * * * G R O U N D W A T E R   I N P U T   D A T A * * * * * *
                                  E L E V A T I O N S
 SUBCAT.   GUTTER                    INITIAL                          F L O W   C O N S T A N T S
   NO.    OR INLET   GROUND   BOTTOM  STAGE      BC       TW          A1          B1          A2          B2
                      (FT)     (FT)    (FT)     (FT)     (FT)   (IN/HR-FT**B1)        (IN/HR-FT**B2)
    21       22       20.00    0.00    7.20     8.55     8.55    4.500E-05   2.600    0.000E+00    1.000
```

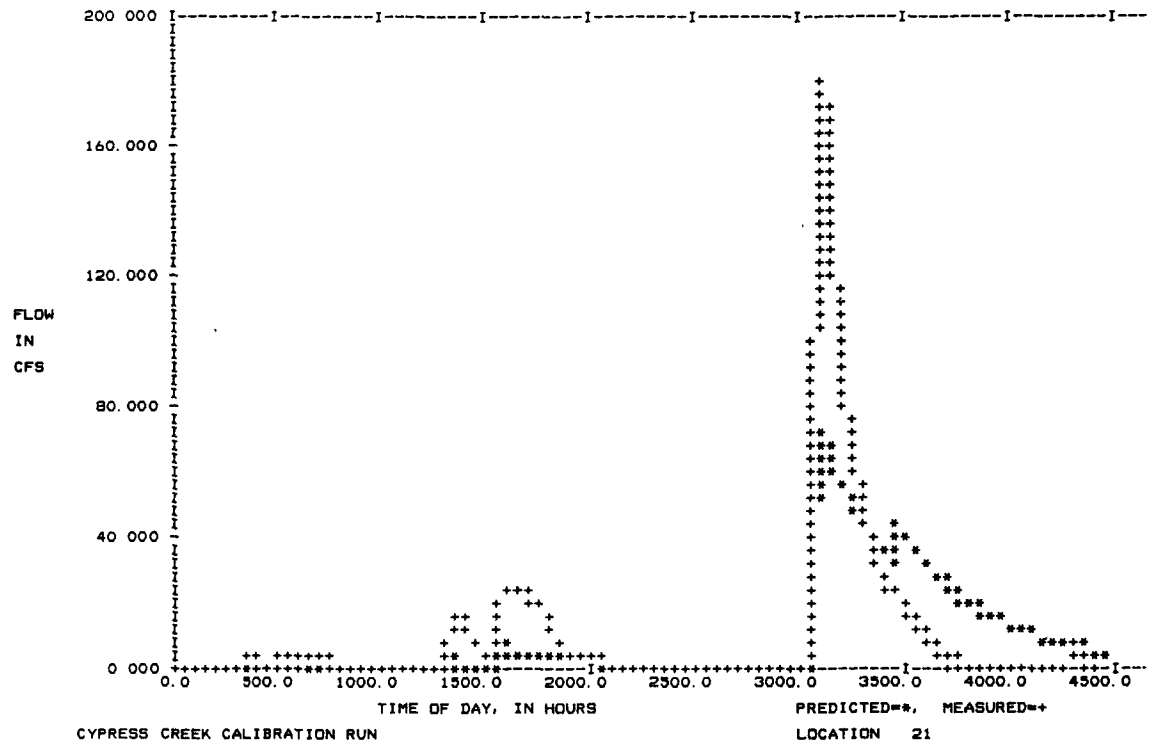
```
* * * * * * G R O U N D W A T E R   I N P U T   D A T A   (CONTINUED) * * * * * *
                S O I L   P R O P E R T I E S                                                  E T   P A R A M E T E R S
                          SATURATED                                                 PERCOLATION
 SUBCAT.                  HYDRAULIC    WILTING   FIELD    INITIAL    MAX. DEEP     PARAMETERS          DEPTH   FRACTION OF ET
   NO.     POROSITY      CONDUCTIVITY   POINT   CAPACITY  MOISTURE   PERCOLATION   HCO *    PCO **      OF ET    TO UPPER ZONE
                           (IN/HR)                                    (IN/HR)                          (FT)
    21      .4600          5.000       .1500    .3000     .3010      2.000E-03    10.00    15.00      14.00        0.350
```

```
* HYD. CONDUCTIVITY = SAT. HYD. COND. * EXP((UPPER Z MOISTURE CONTENT - POROSITY) * HCO)
** PERCOLATION RATE = HYD. COND. * (1 + PCO * (UPPER ZONE MOISTURE CONTENT - FIELD CAPACITY)/(UPPER ZONE DEPTH/2))
```

Figure 11. Subsurface input data for Cypress Creek calibration.

```
$ $ $ --- CONTINUITY CHECK FOR QUANTITY --- $ $ $
                                                          CUBIC FEET     INCHES OVER
                                                                         TOTAL BASIN
TOTAL PRECIPITATION (RAIN PLUS SNOW)                     3.434232E+09      30.518

TOTAL INFILTRATION                                       2.878862E+09      25.583

TOTAL EVAPORATION                                        5.298000E+08       4.708

TOTAL GUTTER/PIPE/SUBCAT FLOW AT INLETS                  2.559983E+07       0.227

TOTAL WATER REMAINING IN GUTTER/PIPES                    0.000000E+00       0.000

TOTAL WATER REMAINING IN SURFACE STORAGE                 0.000000E+00       0.000

INFILTRATION OVER THE PERVIOUS AREA...                   2.878862E+09      25.841
                      --------

INFILTRATION + EVAPORATION +
SNOW REMOVAL + INLET FLOW +
WATER REMAINING IN GUTTER/PIPES +
WATER REMAINING IN SURFACE STORAGE +
WATER REMAINING IN SNOW COVER.........                   3.344122E+09      29.718

*** CONTINUITY CHECK FOR SUBSURFACE WATER ***

                                                        CUBIC FEET     INCHES OVER
                                                                       TOTAL BASIN
TOTAL INFILTRATION                                     2.878862E+09       25.583
TOTAL UPPER ZONE ET                                    1.149578E+09       10.216
TOTAL LOWER ZONE ET                                    6.667578E+08        5.925
TOTAL GROUNDWATER FLOW                                 9.013922E+07        0.801
TOTAL DEEP PERCOLATION                                 4.816257E+08        4.280
INITIAL SUBSURFACE STORAGE                             9.675055E+09       85.978
FINAL SUBSURFACE STORAGE                               1.016489E+10       90.330

UPPER ZONE ET OVER PERVIOUS AREA                       1.149578E+09       10.319
LOWER ZONE ET OVER PERVIOUS AREA                       6.667578E+08        5.985


THE ERROR IN CONTINUITY IS CALCULATED AS

*************************************
* PRECIPITATION + INITIAL SNOW COVER  *
*          - INFILTRATION -           *
*EVAPORATION - SNOW REMOVAL -         *
*INLET FLOW - WATER IN GUTTER/PIPES - *
*WATER IN SURFACE STORAGE -           *
*WATER REMAINING IN SNOW COVER        *
*-----------------------------------*
* PRECIPITATION + INITIAL SNOW COVER  *
*************************************


ERROR.............................                                    2.624 PERCENT

*****************************************
* INFILTRATION + INITIAL STORAGE - FINAL *
* STORAGE - UPPER AND LOWER ZONE ET -    *
* GROUNDWATER FLOW - DEEP PERCOLATION    *
* ------------------------------------- *
* INFILTRATION + INITIAL STORAGE -       *
* FINAL STORAGE                          *
*****************************************

ERROR ............................                                    0.039 PERCENT
```

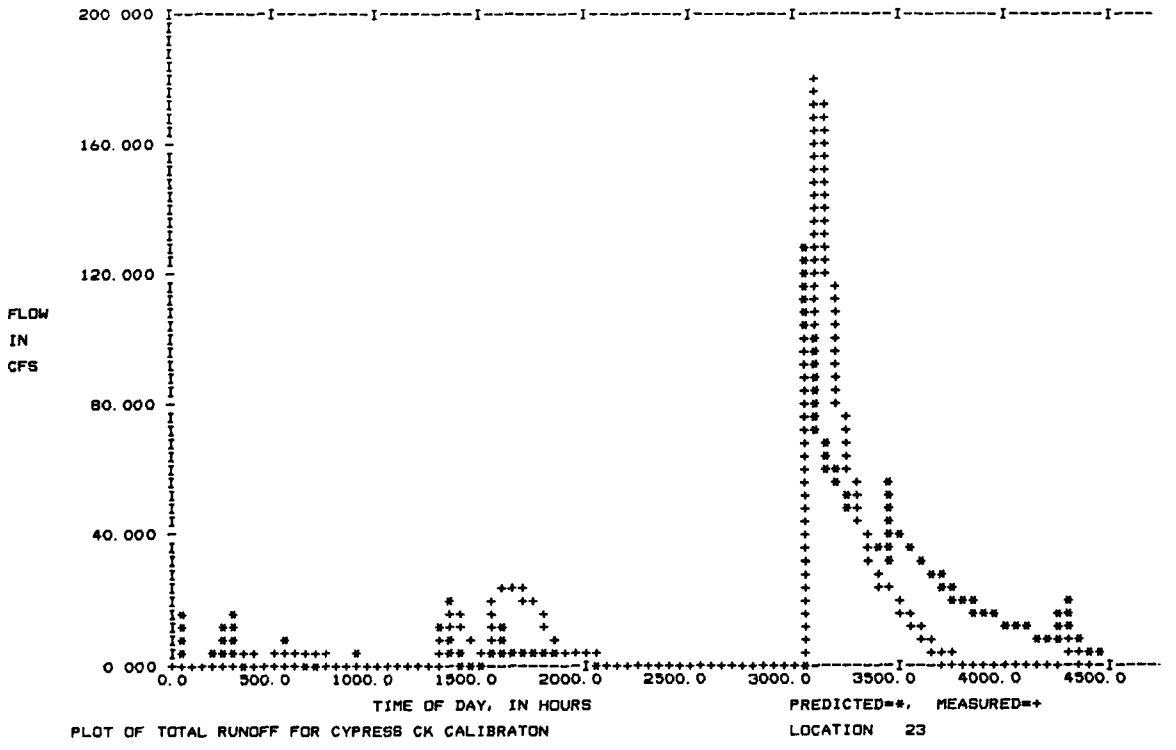
Figure 12. Continuity check for surface and subsurface for Cypress Creek calibration. The relatively large surface continuity error does not actually exist; it comes from a double accounting of the groundwater flow -- a problem that will be fixed.



HYDROGRAPH STATISTICS FOR LOCATION 21

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME, HR	PEAK FLOW FLOW, CFS	DURATION START, HR	DURATION END, HR	DURATION LENGTH, HR	NO. POINTS
PREDICTED, TOTAL TIME	0.14547E+09	1.293	3105.000	73.242	0.000	4430.000	4430.000	194
MEASURED, TOTAL TIME	0.16359E+09	1.454	3120.000	180.000	0.000	4392.000	4392.000	184
PREDICTED, OVERLAPPING TIME	0 14463E+09	1.285	3105.000	73.242	0.000	4393.000	4393.000	192
MEASURED, OVERLAPPING TIME	0.16359E+09	1.454	3120.000	180.000	0.000	4392.000	4392.000	184
DIFFERENCES, ABSOLUTE	0.18964E+08	0.169	15.000	106.758				
% OF MEAS		11.592		59.310				

Figure 13. Predicted groundwater flow hydrograph and total measured flow hydrograph for Cypress Creek calibration.



HYDROGRAPH STATISTICS FOR LOCATION 23

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME, HR	PEAK FLOW FLOW, CFS	DURATION START, HR	DURATION END, HR	DURATION LENGTH, HR	NO. POINTS
PREDICTED, TOTAL TIME	0. 17127E+09	1. 522	3059. 000	128. 228	0. 000	4430. 000	4430. 000	194
MEASURED, TOTAL TIME	0. 16359E+09	1. 454	3120. 000	180. 000	0. 000	4392. 000	4392. 000	184
PREDICTED, OVERLAPPING TIME	0. 17042E+09	1. 514	3059. 000	128. 228	0. 000	4393. 000	4393. 000	192
MEASURED, OVERLAPPING TIME	0 16359E+09	1. 454	3120. 000	180. 000	0. 000	4392 000	4392. 000	184
DIFFERENCES, ABSOLUTE	-0 68276E+07	-0. 061	61. 000	51. 772				
% OF MEAS		-4. 174		28. 762				

Figure 14. Total predicted flow hydrograph and total measured flow hydrograph for Cypress Creek calibration.

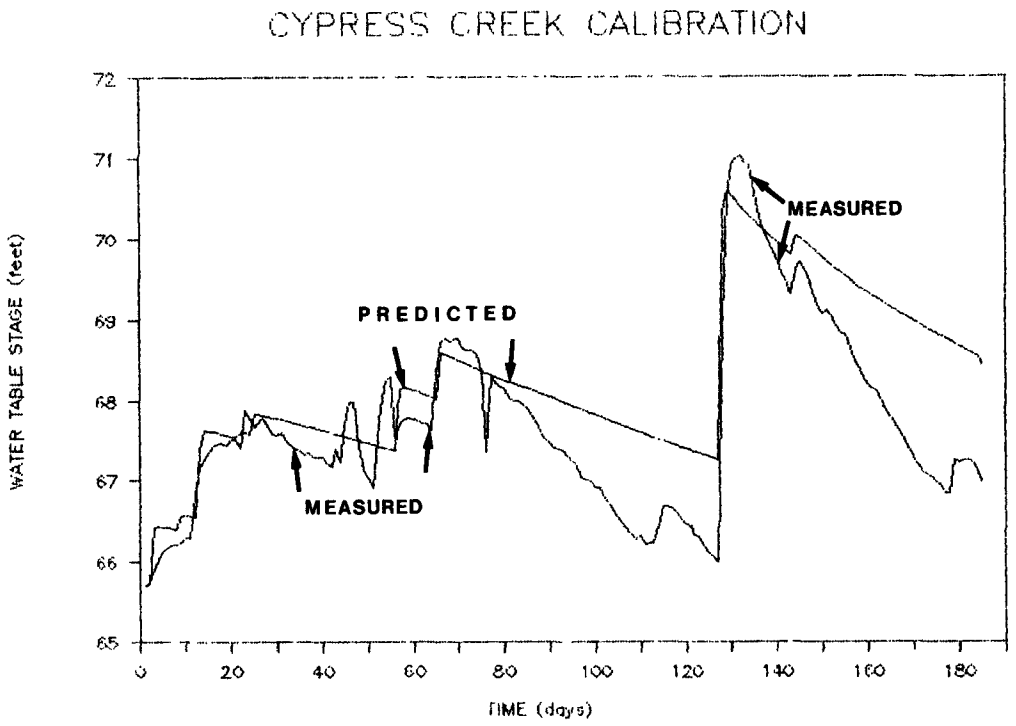


Figure 15. Predicted and measured stages for Cypress Creek calibration.

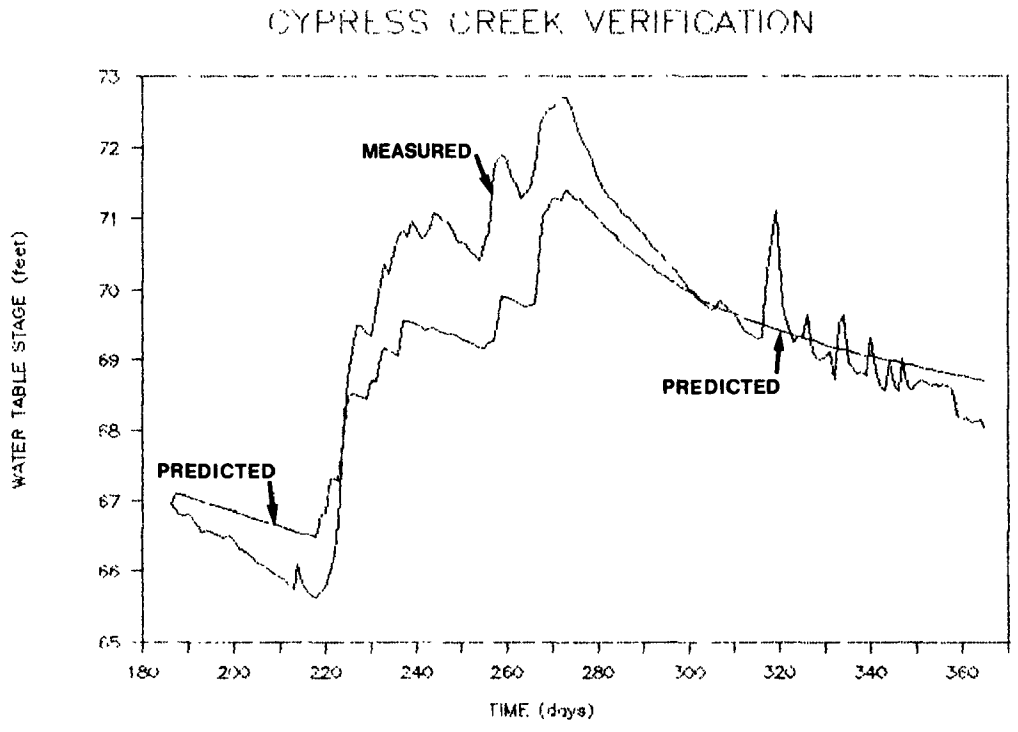
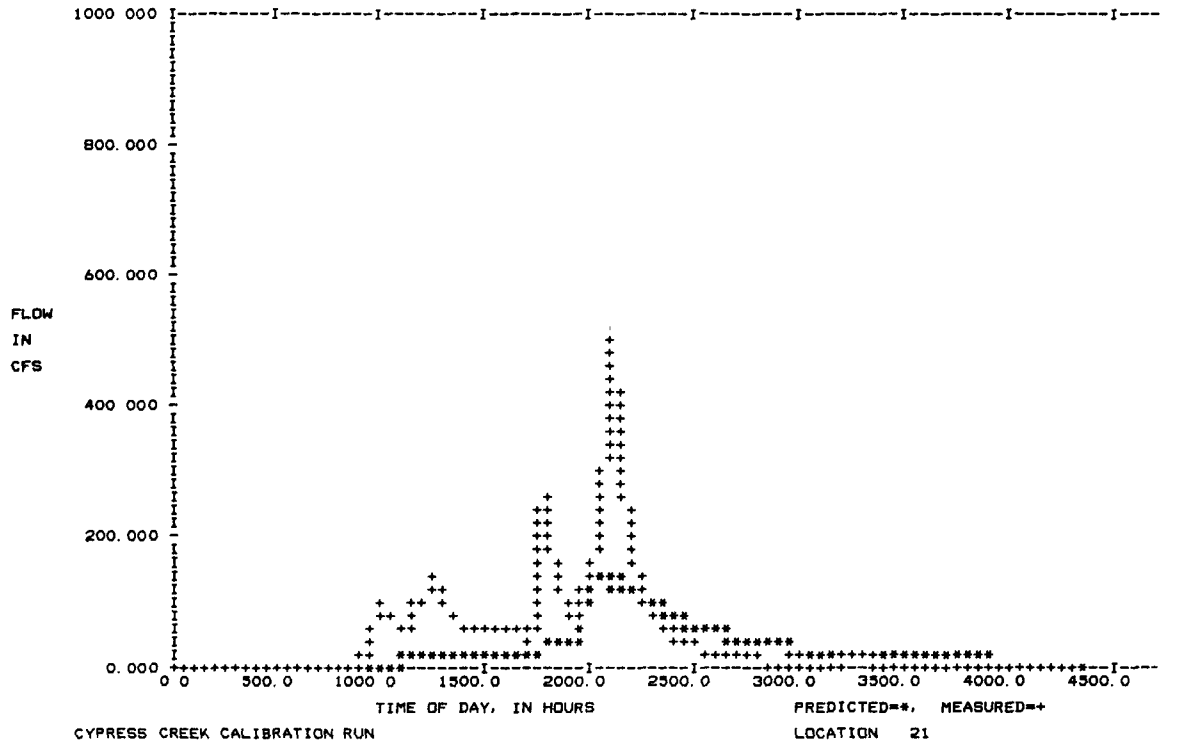


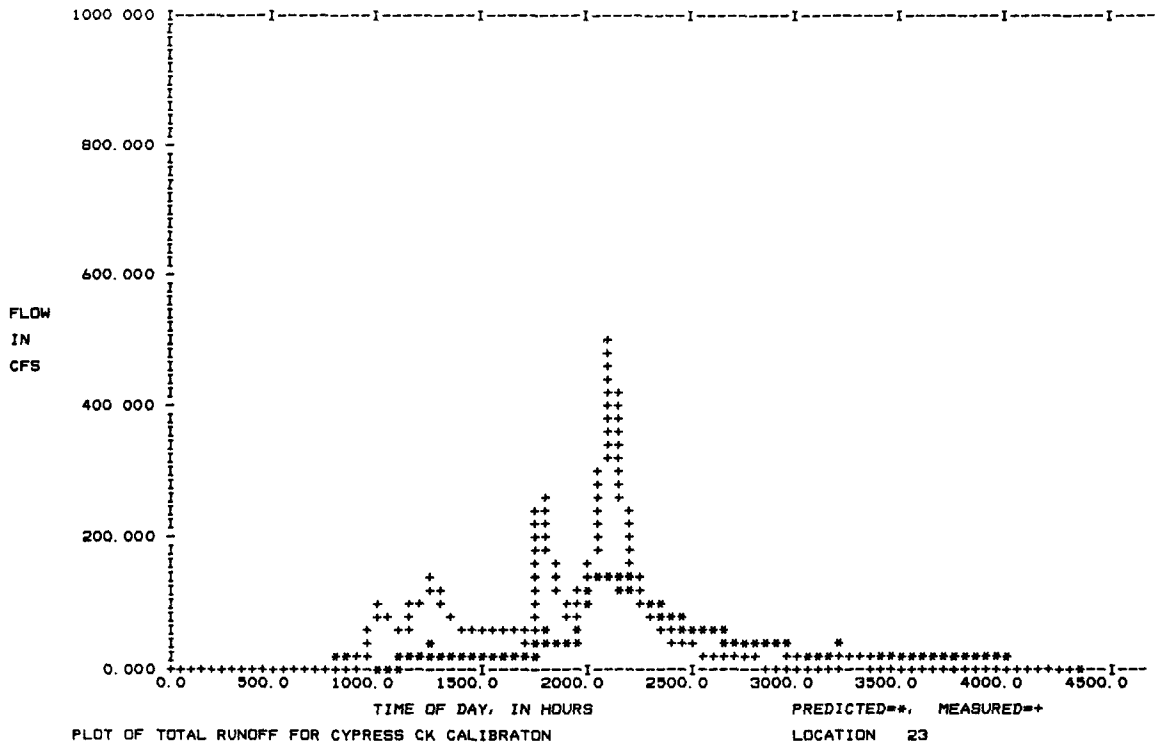
Figure 16. Predicted and measured stages for Cypress Creek verification.



HYDROGRAPH STATISTICS FOR LOCATION 21

	VOLUME CUBIC FEET	VOLUME INCHES	PEAK FLOW TIME, HR	PEAK FLOW FLOW, CFS	DURATION START, HR	DURATION END, HR	DURATION LENGTH, HR	NO. POINTS
PREDICTED, TOTAL TIME	0 42533E+09	3. 780	2112. 000	144. 583	0 000	4350. 000	4350. 000	199
MEASURED, TOTAL TIME	0. 71232E+09	6. 330	2112. 000	500. 000	0. 000	4320. 000	4320. 000	181
PREDICTED, OVERLAPPING TIME	0. 42426E+09	3. 770	2112. 000	144. 583	0. 000	4312. 000	4312. 000	197
MEASURED, OVERLAPPING TIME	0 71232E+09	6. 330	2112. 000	500. 000	0. 000	4320. 000	4320. 000	181
DIFFERENCES, ABSOLUTE	0. 28806E+09	2. 560	0. 000	355. 417				
% OF MEAS		40. 440		71. 083				

Figure 17. Predicted groundwater flow hydrograph and total measured flow hydrograph for Cypress Creek verification.



HYDROGRAPH STATISTICS FOR LOCATION 23

	VOLUME		PEAK FLOW		DURATION			NO.
	CUBIC FEET	INCHES	TIME, HR	FLOW, CFS	START, HR	END, HR	LENGTH, HR	POINTS
PREDICTED, TOTAL TIME	0 44397E+09	3. 945	2112. 000	149. 908	0. 000	4350. 000	4350. 000	199
MEASURED, TOTAL TIME	0. 71232E+09	6. 330	2112. 000	500. 000	0. 000	4320. 000	4320. 000	181
PREDICTED, OVERLAPPING TIME	0 44289E+09	3. 936	2112. 000	149. 908	0. 000	4312. 000	4312. 000	197
MEASURED, OVERLAPPING TIME	0. 71232E+09	6 330	2112. 000	500. 000	0. 000	4320. 000	4320. 000	181
DIFFERENCES, ABSOLUTE	0. 26943E+09	2. 394	0. 000	350. 092				
% OF MEAS		37. 824		70. 018				

Figure 18. Total predicted flow hydrograph and total measured flow hydrograph for Cypress Creek verification.

from the surface. The 10-yr SCS Type II design storm for Tallahassee, Florida is used for the rainfall input (Figure 19). This storm is characterized by very high rainfall between hours 11 and 12.

In order to illustrate the influence of a high water table, runs were made with and without the groundwater subroutine. Table 1 shows the disposition of the rainfall when a high water table is simulated as opposed to when it is ignored. Note that evaporation is about the same, and the difference in the amount of infiltrated water shows up as a direct difference in surface runoff. (The runs were halted before all water had run off.) The two hydrographs and the corresponding water table (for the run in which it is simulated) are shown in Figure 20. A larger difference in peak flows would have resulted if the flows had not been routed to a very large channel. Also, note that the two hydrographs are identical until about hour eleven into the simulation, when the simulated water table rises to the surface.

TABLE 1. FATE OF RUNOFF WITH AND WITHOUT HIGH WATER TABLE SIMULATION

Water Budget Component	Inches Over Total Basin	
	With Water Table Simulation	Without Water Table Simulation
Precipitation	8.399	8.399
Infiltration	6.637	1.731
Evaporation	0.103	0.104
Channel flow at inlet	1.495	2.407
Water remaining in channel	0.015	0.038
Water remaining on surface	0.150	4.124
Continuity error	0.001	0.005

Execution time on the IBM 3033 mainframe increased from 0.32 CPU seconds without the groundwater simulation to 0.42 CPU seconds with the groundwater simulation. Thus, some additional computational expense can be expected.

CONCLUSIONS

Although the subroutine is fairly simple in design and has several limitations, the new groundwater subroutine should increase the applicability of SWMM. Preliminary test runs have determined it to be accurate in the simulation of water table stage and groundwater flow. Further calibration and verification tests need to be done on other areas to confirm these preliminary results. Also, estimation of parameters, although fairly numerous, appears to be relatively uncomplicated. In addition, parameters are physically-based and should be able to be estimated from soils data. The flexible structure of the algorithm should permit a more realistic simulation of catchments in which a major hydrograph component is via subsurface pathways.

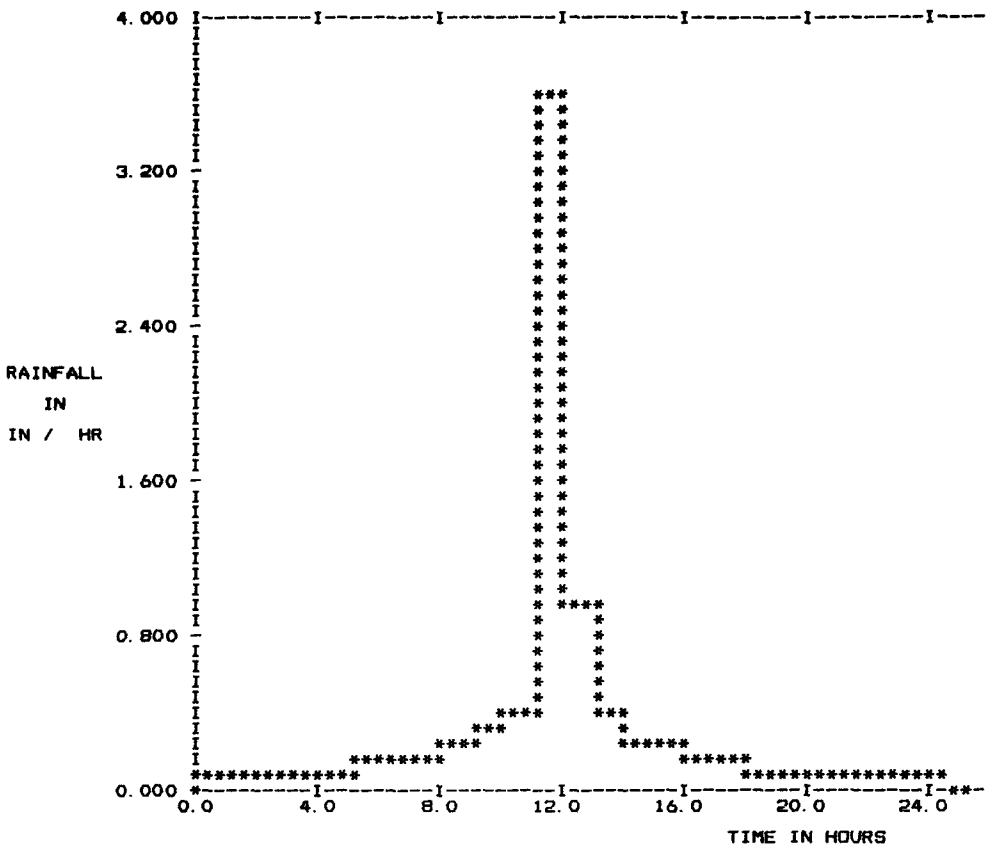


Figure 19. Hyetograph for hypothetical subcatchment (10-yr SCS Type II design storm for Tallahassee, Florida).

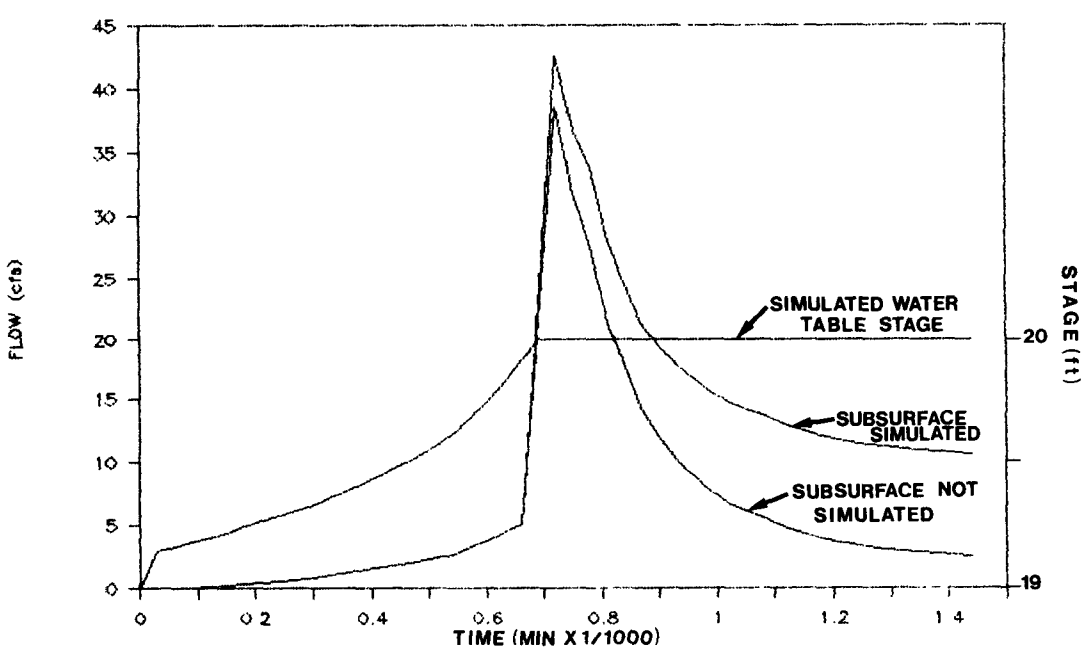


Figure 20. Hydrographs of surface flow and simulated water table stage from hypothetical subcatchment. The hydrographs are identical until the water table reaches the surface (20 ft).

ACKNOWLEDGMENTS

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SWMM APPLICATIONS FOR MUNICIPAL STORMWATER MANAGEMENT: THE EXPERIENCE OF VIRGINIA BEACH

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ABSTRACT

The Stormwater Management Model (SWMM) plays a significant role in the stormwater management program of the City of Virginia Beach, a rapidly growing municipality covering 250 square miles in southeastern Virginia. Numerous subdivision designs are based upon SWMM simulations performed by consultants to land developers. SWMM use has increased due to the need to evaluate complex hydraulic phenomena present in flat coastal areas, and because of easier access to sophisticated computer models and microcomputers. As a rule, RUNOFF is used for hydrologic predictions while EXTRAN is used to size storm sewers, culverts, and detention basins. Guidelines have been issued for SWMM studies of development projects that must be approved by the City, and submittal and review of SWMM input datasets is now required. Problems encountered by municipal public works departments when reviewing land development designs based on computer simulations will be discussed.

SWMM is also the principal planning tool being used to develop a stormwater master plan for the City of Virginia Beach. The master plan will recommend structural and nonstructural control measures for peak flow control and nonpoint pollution management under ultimate development conditions. SWMM is ideal for this study because of the importance of backwater, flow reversals, interconnected canal and lake systems, and tidal boundary conditions. A key use of the EXTRAN Block is the evaluation of the primary drainage system (40 mi), which consists of several large interconnected canals controlled by three major tidal outlets with different boundary conditions. SWMM has been enhanced for this study to accept multiple boundary conditions and simulate channels of irregular cross-section input in a HEC-2 format. Upon completion of the master plan, the enhanced SWMM model and master plan data sets will be used by the City to evaluate changing land use patterns, to establish tailwater design conditions for drainage projects, and to evaluate individual development proposals.

INTRODUCTION

The City of Virginia Beach in the southeastern, or Tidewater, portion of Virginia, is the fastest growing city on the east coast (Figure 1). A wide diversity of land uses is found within the borders of this 250-square mile City, including dense suburban development in the north and northwest, commercial and retail space along the toll road corridor, high rise hotels along the beachfront, several major military bases, a rapidly developing region in the center of the City, and vast farm land and unspoiled wetlands in the southern half of the City. Development is encroaching rapidly on the Back Bay, a large estuary whose water quality has been declining in recent years. Environmental concerns have prompted the establishment of a planned limit to development--the Green Line--which currently precludes most development in the Back Bay Watershed.

Hydraulically, the stormwater conveyance system in Virginia Beach is characterized by an interconnected canal system which provides primary drainage for over half of the City. This system has three major boundaries: Chesapeake Bay; the Elizabeth River, a tributary of the Chesapeake Bay; and Currituck Sound in North Carolina. Many of these streams flow through major freshwater and saltwater wetlands, constraining potential channelization projects. The primary stormwater management controls currently used in Virginia Beach are on-site detention ponds serving large subdivisions.

The purpose of this paper is two-fold. First, the experience of the City in stormwater management is presented. A key highlight is the effort required by the City to review stormwater facility designs based upon SWMM simulations submitted for individual subdivisions. In several cases, the review process has been hindered because models have been misapplied or designs have been based upon erroneous results. The City has issued guidelines on the use of SWMM for subdivision design and is considering the submission and analysis of SWMM run streams as a component of subdivision review.

The need for a stormwater model to aid in subdivision review, coupled with the rapid growth of the City, prompted the development of a stormwater master plan for the City. The second purpose of this paper is to present the key issues involved in the development of the master plan, the techniques required for modeling the stormwater conveyance system, modifications to SWMM required for the model, and future uses of the master plan model by the City.

VIRGINIA BEACH STORMWATER MANAGEMENT PRACTICES

CITY SUBDIVISION REVIEW PROCEDURES

Virginia Beach regulates stormwater management of new development through a three stage subdivision review process. Requests for rezoning are evaluated based on general impact to the City drainage system, impacts to wetlands, and location of the project with respect to known floodplains. As development proceeds, a hydrology/hydraulic study for an entire subdivision site are prepared by consultants for the developer and reviewed by the City. The City's review ensures that overall site drainage is designed according to City standards and criteria. Finally, detailed subdivision site plans are

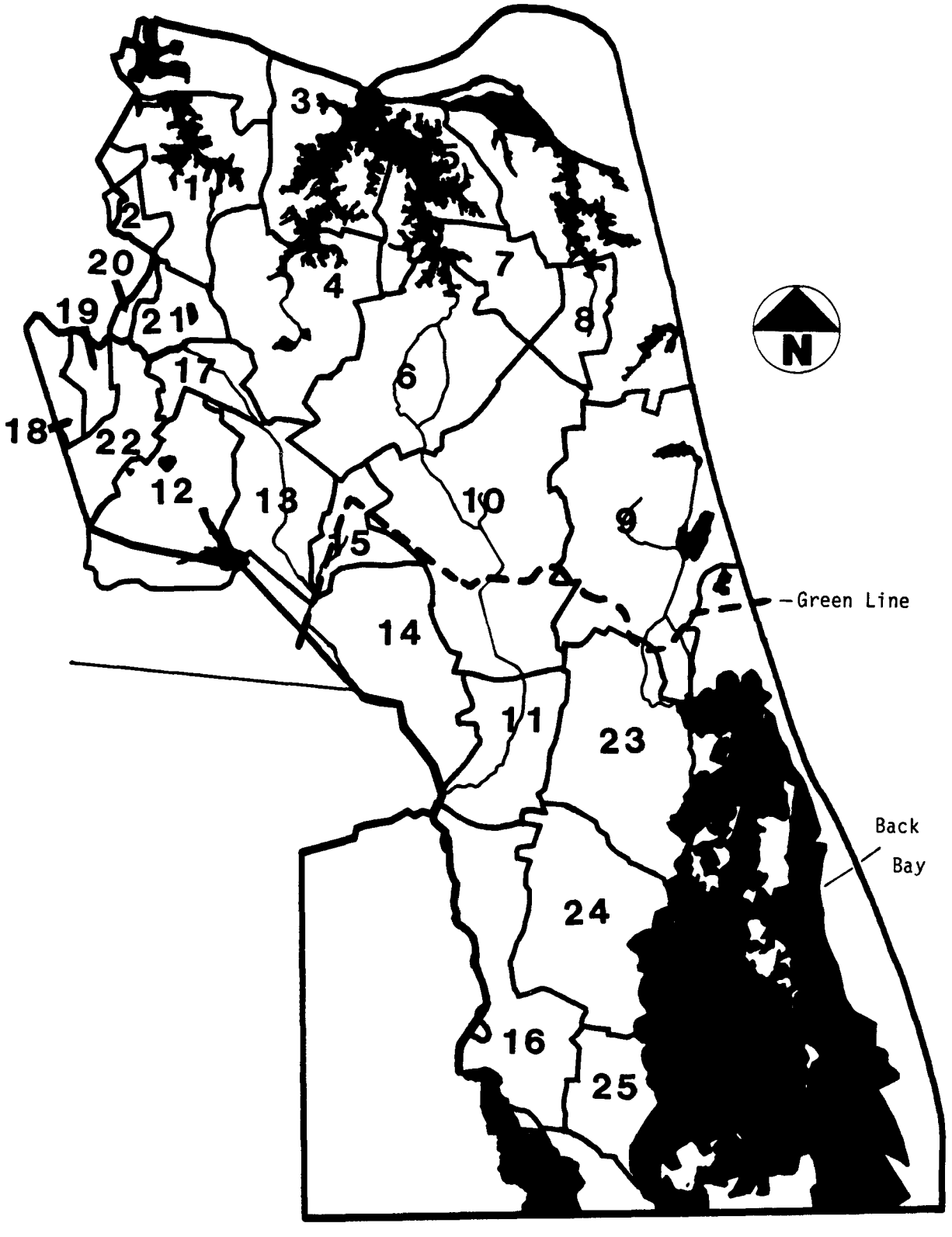

Figure 1. Virginia Beach Watersheds and Primary Channels

reviewed, with attention given to storm sewer design, inlet/outlet sizing and location, and adequate site grading. Improvements to existing stormwater facilities required for existing development are capital improvement projects (CIP) evaluated and designed by the City's Special Projects section and financed through the General Fund.

USE OF SWMM FOR SUBDIVISION DESIGN

Each year the City Plan Review Bureau receives a limited number of subdivision hydrology studies performed for developers by engineering consultants. Approximately six were received in 1986; however, the number is increasing each year.

These subdivision studies typically rely upon SWMM to model systems of interconnecting detention lakes for large (greater than 100 acres) proposed subdivisions. The Virginia Beach Department of Public Works (VAB-DPW) reports that few SWMM studies are approved on the first submittal, largely because a "final" version of the SWMM modeling study, with a voluminous printout and limited documentation, is submitted without prior consultation with the City engineering review staff. To ease this problem, the City issued guidelines for hydrology studies based upon SWMM. These guidelines can be summarized as follows:

- A clear schematic of the drainage system and tables of parameters for each subcatchment, channel, and lake must be provided,

- Either 12-hour design storms with wet antcedent moisture conditions or 24-hour design storms with dry antecedent moisture conditions may be simulated, and

- Equivalent conduit calculations must be documented and EXTRAN simulations must be shown to be numerically stable.

VAB-DPW has found that problems with subdivision hydrology studies using SWMM result from four main factors:

1. Failure to coordinate the SWMM study activities with City engineering review staff prior to submittal; i.e., the guidelines are not followed.

2. Lack of understanding of the theoretical basis of the program, specifically that stability criteria must be satisfied when approximating solutions to the governing St. - Venant equations with the explicit finite difference methods used by EXTRAN.

3. Failure to check the SWMM results for either numerical errors, printed error messages, or accuracy versus simple hand calculations.

4. Use of SWMM for subdivision design in lieu of easier to apply models which are more appropriate for certain drainage analyses simply because SWMM is considered to be a "trendy" and sophisticated tool.

The following example illustrates some of the problems encountered by VAB-DPW with subdivision hydrology studies based on SWMM. The SWMM modeled hydrology study for a proposed subdivision (430 acres, 9 detention lakes outfalling to an existing ditch) was first submitted to the City without prior coordination on input data. The study was disapproved six weeks later (the long review time due to insufficient documentation of the study) for 12 reasons, primarily involving RUNOFF block data and large continuity errors. The study was modified and resubmitted in six weeks. Large continuity errors, for some runs on the order of 35%, still existed. The consultant submitted hand calculations attempting to show the continuity error was due entirely to EXTRAN's failure to compute the initial volume. However, after accounting for this volume of water in the wet detention system continuity errors were still between 13% and 16%. After many discussions and meetings, the study was approved--approximately five months after the first submittal.

VIRGINIA BEACH STORMWATER MASTER PLAN

PRODUCTS OF THE STORMWATER MASTER PLAN

A stormwater master plan analyzes the watershedwide impacts of stormwater runoff and proposes an appropriate mix of controls to alleviate stormwater impacts. A master plan focuses on an overall framework of management alternatives which may include the following:

- Regional detention systems,
- Improvements to the primary stream or sewer within a subbasin (about 200-300 acres),
- Improvements to major stream crossings on this stream, and
- Nonstructural measures within the subbasin, such as flood plain zoning, land acquisition, land use controls, etc.

The objective of master planning is to locate facilities and propose management schemes which provide the greatest benefits, minimize capital and O&M costs, and provide the greatest environmental sensitivity throughout the entire watershed. Master planning usually does not address local subdivision and highway drainage systems since these are typically evaluated for lesser design events and seldom influence watershedwide drainage individually. The primary concerns of the master planning study are the interactions between large stormwater facilities.

SWMM was selected for the master plan study because EXTRAN allows dynamic simulation of interactions between the major facilities proposed for stormwater management. On-site stormwater controls typically limit peak flows to predevelopment levels. Analysis seldom proceeds outside the subdivision, where the timing and duration of peak runoff from on-site controls may interact with runoff flows from other parts of the watershed to increase downstream flows and water surface elevations. In many cases, these dynamic interactions cause flooding of the downstream conveyance system, causing backwaters which may be severe enough to impact the performance of the on-site facility.

For regional facilities, design is typically based on more severe rainfall conditions and larger drainage areas, thus the potential for adverse impacts due to regional interactions is increased. By dynamically predicting the time history of flow and water surface level throughout the entire watershed, EXTRAN is able to identify and indicate solutions to adverse conditions caused by hydrograph interaction, backwater, varying tidal boundary conditions, and flow diversions.

KEY ISSUES OF THE VIRGINIA BEACH STORMWATER MASTER PLAN

Virginia Beach is currently in a period of rapid growth. Figure 2 shows the recent and planned growth within the past five to ten years in a typical watershed in the City. This growth is stressing both the hydraulic conveyance and the environmental quality of drainage systems throughout the City, a phenomenon which is drawing increased public awareness to the costs of rapid growth. Master planning for the entire City is desired to determine the overall capacity of the City drainage system, identify capacity problems under existing and ultimate land use conditions, and develop controls which solve these capacity problems. The City has found that on-site drainage controls and a strict program of subdivision review cannot ensure that the impacts of rapid growth are controlled.

Water quality protection is a serious concern in the City. The major estuaries within the City are severely stressed, with water quality perceived to be worsening under development pressure. Much new development in Virginia Beach includes lakes with permanent pools which provide recreation and aethstetics as well as drainage control. Thus many existing and proposed lakes, ponds and detention facilities may provide pollutant removal if operated and maintained properly. The master plan will propose detention facilities suitable for pollutant removal as well as drainage control.

Virtually all major streams within Virginia Beach flow through freshwater and estuarine wetlands. Many drainage projects and a few subdivision proposals have been denied under Federal wetlands regulations. Federal opposition has arisen from channelization projects through wetlands, drainage diversions out of wetlands, and drainage bypasses of lakes providing wetlands protection to the stressed estuaries. Because of the increased emphasis on wetlands protection, the cost to study, design, and construct major drainage improvements has increased dramatically. Thus alternatives based upon regional detention storage and various non-structural measures (flood plain zoning, land acquisition, land use controls, down zoning) are proving to be attractive. These measures correspond well with the current mood of many residents demanding controls and limits to growth, but may be resisted by major developers and large land owners unless compensation is provided.

Currently, drainage projects for existing development are financed from the City's general fund. These capital improvements are budgeted annually. Developers must design and construct stormwater facilities on new development sites. These stormwater facilities are reviewed by the City and traditionally have been deeded to the City following construction. Recently, the City has advocated continued private ownership of stormwater facilities since insufficient public funding is available for O&M of these facilities. These O&M requirements are increasing as regulations and environmental

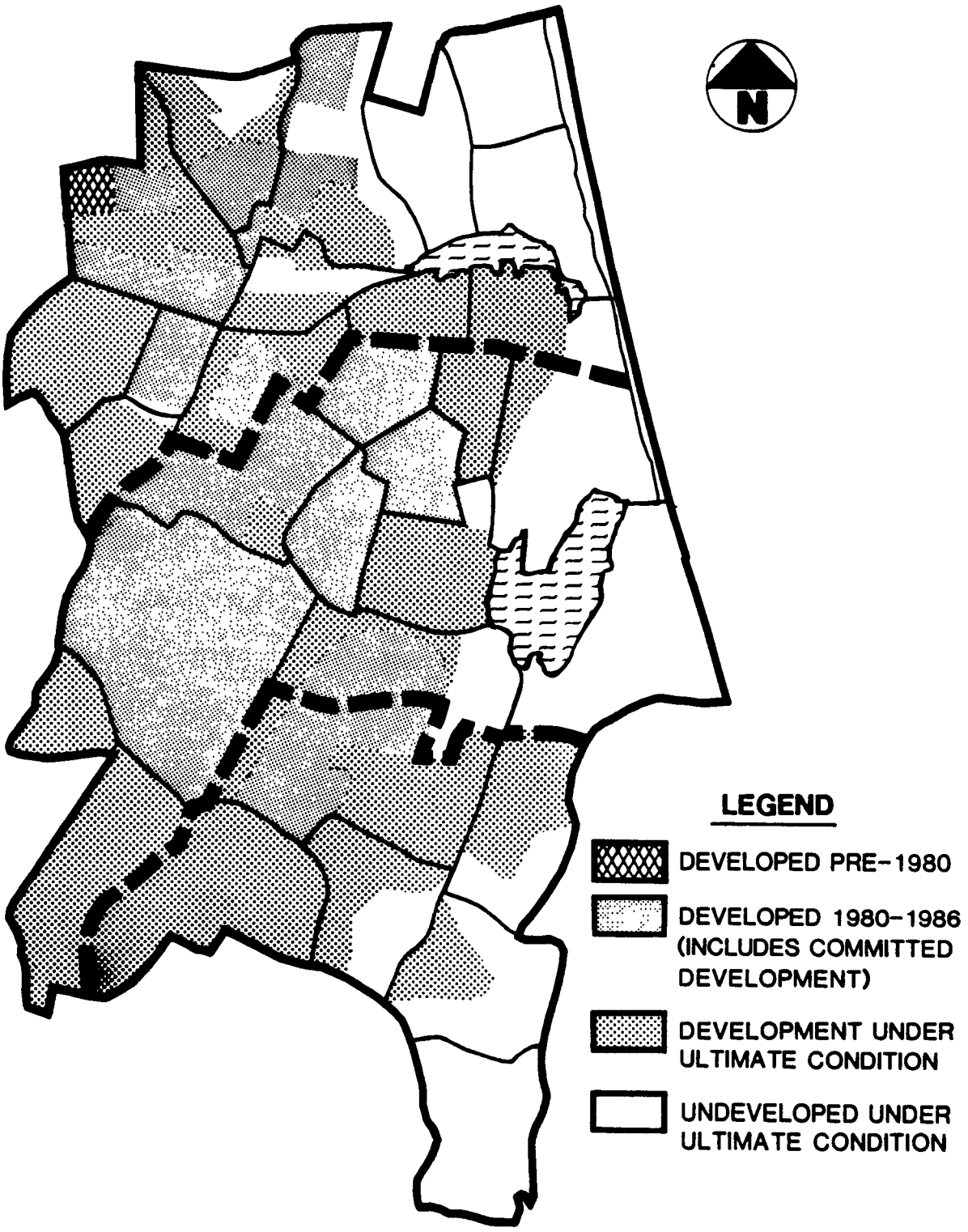

Figure 2. Development Trends - Redwing Lake Study Area

constraints promote detention/retention ponds as the bulk of stormwater facilities within the City. Regular programs for routine maintenance of ditches and channels are performed regularly by the City's Highway Department. Maintenance requirements for detention ponds which provide effective water quality control have not been established by the City, nor is this maintenance typically funded either publiclly or privately. Therefore, feasible institutional and financial methods for funding stormwater management are another focus of the stormwater master plan. Existing and potential City ordinances and regulatory proceedures will be explored, as will innovative financing methods for the construction and maintenance of stormwater facilities. Innovative financing methods include stormwater utilities, which impose a fee on 'users' of stormwater facilities, and pro rata share assessments, where new development pays a portion of the costs for regional stormwater management controls.

SWMM MODELING APPROACH

Watershedwide Models

Twenty-five major watersheds were identified in Virginia Beach. Regions where maximum flood elevations are caused by coastal storm surges were excluded from the master plan since stormwater management controls for rainfall-runoff flooding would be ineffective. Watershed sizes ranged from 20 square miles to less than one square mile, with average subbasin size ranging between 200 and 300 acres. Only those channels and storm sewers which provide the primary drainage for one or more subbasins are modeled for the master plan. A separate SWMM model of each watershed was established to study the performance of the existing regional stormwater drainage system, to identify flooding locations under existing and ultimate land use conditions, and to study alternative plans for relief, control, or management of this flooding. Runoff from Design storms with return periods of 10, 25, and 50 years was predicted to evaluated drainage system performance according to Virginia Beach design standards. All stormwater management alternatives were screened to insure that their interactions did not worsen flooding elsewhere in the City. Water surface profiles under the master plan recommendations for the 10-year storm will serve as tailwater elevations for subdivision drainage design. Minimum floor elevations will be set at one foot above the 100-year storm water surface elevation under the master plan recommendations.

City-wide Model

The interconnected canals which comprise the bulk of the primary stormwater drainage system for the City are a unique concern in coastal areas such as Virginia Beach. The major north-south drainage divide moves depending on interactions between tidal conditions, rainfall intensity, drainage improvements and growth patterns throughout the City. Therefore a City-wide SWMM model of the entire interconnected canal system was created to help understand these interactions and permit delineation of detailed master planning models of smaller areas. The 200-300 acre subbasins from the watershedwide models were also used in the City-wide model. EXTRAN channels in the City-wide model were generally greater than 3000 feet long, permiting a three minute computational time step and simulations of 30-40 hours. For initial screening of overall hydraulic performance, only culverts shown to be significant hydraulic constraints were included in the City-wide model. This decreased computation time and permitted estimation of maximum conveyance of

the canals. Spreadsheets were used to compute culvert capacity under various headwater and tailwater conditions. Comparisons between culvert capacity and peak channel flows allowed identification of additional stream crossings as hydraulic constraints.

Tidal Boundary Conditions

Tidal boundaries are an important consideration for stormwater management in Virginia Beach. Even though the stormwater master plan does not study or recommend controls for tidal flooding, the interaction between fluvial and tidal flooding must ensure that flooding in tidal zones is not increased by stormwater controls for fluvial flooding. For this study, stormwater management controls were designed based on average annual astronomical or wind-driven high tides (i.e., the 50-year rainfall event coinciding with a one-year tide is a 50 year event). The performance of stormwater controls during tidal flooding events was checked by predicting flood elevations caused by a design storm surge coinciding with the average rainfall observed during historical surge events (typically rainfall on the order of a two-year storm). From this analysis, three zones were identified within each watershed. The zone of fluvial flooding is bounded downstream at the point where rainfall-runoff does not raise the water surface above high tide elevation. The zone of tidal flooding is bounded upstream by the limit of tidal backwater during a design storm surge event. The third zone lies between the first two and defines where tidal flooding sets the design flood elevation but where stormwater controls may be effective during more frequent events. The stormwater master plan focuses on control in the zone of fluvial flooding, but investigates benefits of controls in the intermediate zone where tidal and fluvial flooding is significant.

SWMM ENHANCEMENTS FOR MASTER PLANNING

Several modifications of SWMM were required for watershedwide master planning:

- Irregular channel cross-sections can now be entered in the same format used for HEC-2, permitting data transport between hydraulic models,

- Multiple boundary conditions allow specification of different constant or time-varying tidal stages at different locations within the model, and

- Variable stage/storage/discharge relationships in RUNOFF and variable stage/surface area relationships in EXTRAN permit simulation of lakes, ponds, and detention/retention facilities.

FUTURE USES OF THE MODEL BY THE CITY

A key goal of the stormwater master plan for Virginia Beach was to establish appropriate algorithms and modeled representations of City drainage for future planning, evaluation, and design. Extensive documentation of sources for model parameters is being compiled to aid understanding of the model and modeling concepts. Spreadsheets and CAD drawing files compiled during the study will be used by the City to help maintain the model and perform modifications as growth occurs. The City is considering the

submission of SWMM data sets by land developers to assist with review of subdivision drainage.

DISCLAIMER

The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

THE EFFECT OF SUBWATERSHED BASIN CHARACTERISTICS ON DOWNSTREAM DIFFERENCES IN STORM-RUNOFF QUALITY AND QUANTITY

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ABSTRACT

Runoff quantity and quality were calculated in four subwatersheds of Lamberts Creek, located near St. Paul, Minnesota, during 12 storms in 1985. Downstream differences in storm-runoff quantity and quality in Lamberts Creek are affected by four basin characteristics of the subwatersheds; urban land use (impervious areas), presence of wetlands (surface-water storage), basin slope, and channel slope. Storm-runoff quantity is smallest in subwatersheds that have (1) small amounts of urban land use (impervious area), minimizing surface runoff, (2) gentle basin slopes, impeding subsurface flow, and (3) large amounts of surface-water storage (wetlands), temporarily retaining storm runoff. Storm-runoff loading of total suspended solids, total phosphorus, and total nitrogen is smallest in subwatersheds that have (1) gentle channel slopes, minimizing channel erosion and (2) large wetland areas, allowing for retention of loads through sedimentation. Channelized wetlands are not as effective as unchannelized wetlands in storing storm runoff or in retaining loads.

INTRODUCTION

BACKGROUND

Lamberts Creek, located near St. Paul, Minnesota, USA, flows into a lake from which St. Paul obtains its municipal-water supply. During the summer, the water supply commonly has an undesirable taste and odor that has been linked to algal species associated with eutrophication of the lake (1). The eutrophication is the result of sediment and nutrient input from several point and nonpoint sources, including nonpoint sources in storm runoff from Lamberts Creek. Therefore the quantity and quality of storm runoff from Lamberts Creek required assessment. Lamberts Creek channel is a drainage ditch that initially was constructed to drain wetlands for vegetable farming and subsequently was used to drain urban areas. Additional urban development is proposed in the watershed which may cause additional sediment and nutrient input to the lake from Lamberts Creek.

PURPOSE AND SCOPE

Storm-runoff quantity and quality in Lamberts Creek is largely affected by basin characteristics of the four subwatersheds. This paper presents the results of a study evaluating the effects of basin characteristics on storm-runoff quantity and quality during 12 storms in 1985 within each subwatershed. Storm runoff is defined as the runoff that occurs during the hydrograph resulting from a storm. The paper addresses the downstream differences in storm runoff by evaluating the differences in basin characteristics between subwatersheds. The differences in selected basin characteristics are analyzed to determine what effect a particular basin characteristic has on storm-runoff quantity and quality in Lamberts Creek. The selected basin characteristics include land use, basin slope, channel slope, soil type, and surface-water storage. The downstream differences in storm runoff will be determined by comparing the storm-runoff quantity and quality exported from each subwatershed. The interpretation and discussion is limited to storm-related differences in runoff as annual-related differences (which include storm and nonstorm runoff) are likely different.

METHODS

STUDY AREA

Figure 1 shows the subwatershed basins, data-collection sites, major storm-sewer outlets, and land use. The data-collection sites were placed at the downstream and upstream ends of each subwatershed basin: (a) subwatershed 1 is the basin upstream from data-collection site 1, (b) subwatershed 2 is the basin between data-collection sites 1 and 2, (c) subwatershed 3 is the basin between data-collection sites 2 and 3, and (d) subwatershed 4 is the basin between data-collection sites 3 and 4.

Basin characteristics of each subwatershed are given in table 1. Urban land use areas are those used for residential and commercial purposes. Nonurban land use areas are those presently undeveloped, excluding wetlands or lakes. Vegetated wetlands are the dominant type of waterbody in the study area except for Goose Lake (located southeast of data-collection site 1). Lamberts Creek originates as a large storm-sewer system in subwatershed 1 that discharges into an open ditch at data-collection site 1. Lamberts Creek is channelized through all wetlands except through the wetland immediately downstream from data-collection site 1. The storm-sewered urban area southeast of Goose Lake drains directly into the lake, which has an outlet into the wetland immediately downstream from data-collection site 1. Soils in each subwatershed are classified as either sand, loam, or clay. Surface-water storage is the maximum surface-water storage capacity within each subwatershed expressed in centimeters (cm) derived from cubic meters (m^3) of water storage per square kilometer (km^2) of drainage area.

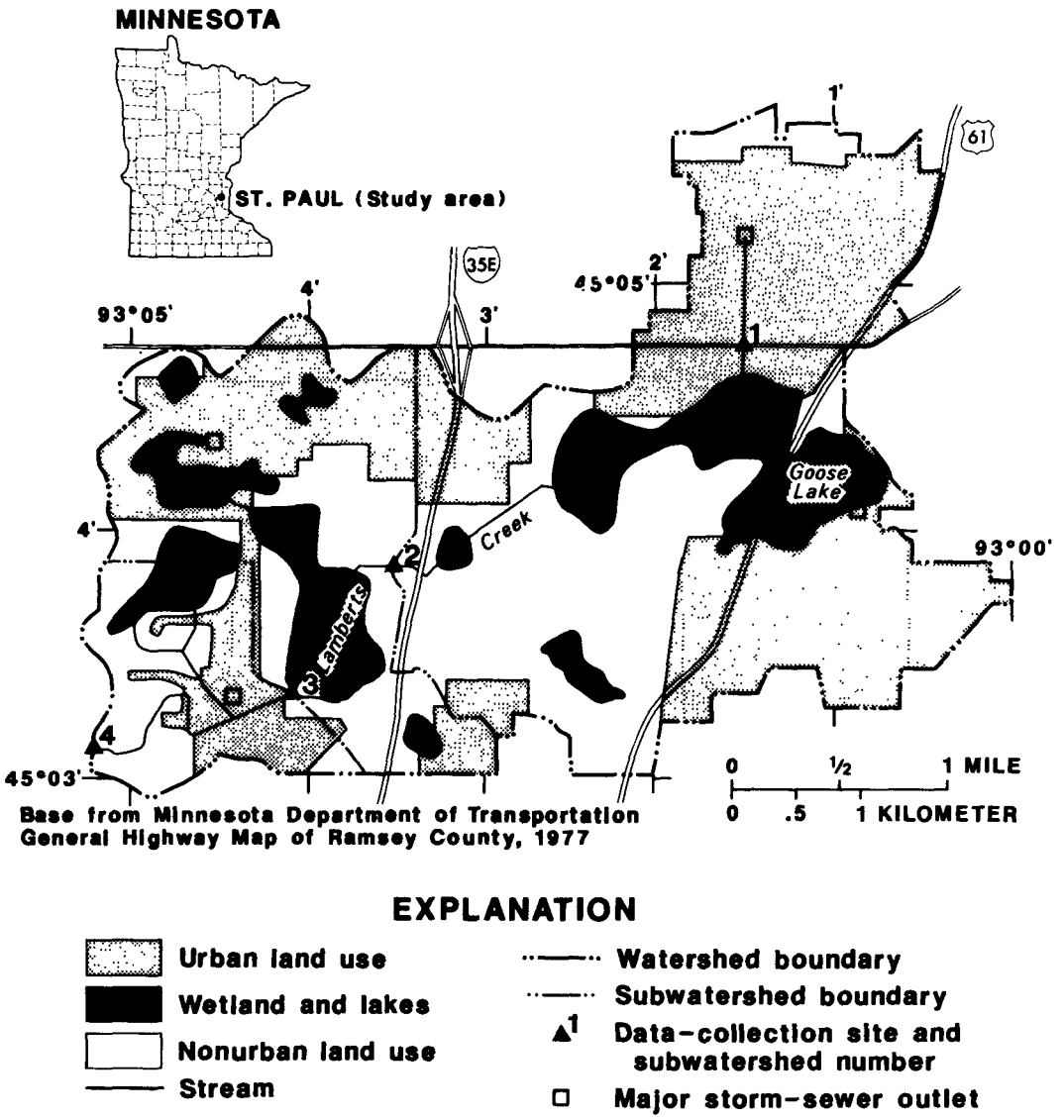

Figure 1. Location of Lambert Creek watershed and subwatersheds showing land use and data-collection sites.

TABLE 1. BASIN CHARACTERISTICS OF THE LAMBERTS CREEK SUBWATERSHEDS*

Basin Characteristics	1	2	3	4
Drainage area (km^2)	2.77	9.04	4.80	2.92
Urban land use (km^2)	2.27(82)	4.10(45)	1.78(37)	1.32(45)
Wetlands and lakes (km^2)	0.03 (1)	2.01(22)	1.69(35)	0.51(17)
Nonurban land use (km^2)	0.47(17)	2.93(33)	1.33(28)	1.09(38)
Basin slope (meters per kilometer)	1	3	10	4
Channel slope (meters per kilometer)	0.19	0.56	0.64	1.2
Channelized wetlands (km^2)	0	0.77	1.51	0.46
Unchannelized wetlands (km^2)	0.03	0.63	0.18	0.05
Soil type (loam, sand, or clay)	loam	loam	sand	sand
Surface-water storage (cm)	2	10	6	5

*number in parentheses is the percent of basin

DATA COLLECTION

Storm-runoff quantity and quality data at each of the four data-collection sites were obtained during 12 storms in 1985: one in March, three in April, one in May, one in July, three in August, two in September, and one in October. Storm-runoff quantity was derived from a continuous record of streamflow during each storm hydrograph, and storm-runoff quality was determined by chemical analysis of water samples. Streamflow was calculated from stream-stage data collected every 15 minutes by use of a relation between stage and measured flow (2).

Discrete samples for water-quality analysis were collected throughout the storm hydrograph at the data-collection sites by automatic samplers. Runoff quality at each site was measured from either the discrete samples collected during each storm or a flow-weighted composite sample of the discrete samples. Flow-weighted composite samples represent the flow-weighted-mean concentration of the discrete samples collected during the event and were calculated by using an equation based on the theory of "mid-interval determination of suspended-sediment discharge" (3).

Discrete and composite samples were analyzed for concentrations of total suspended solids, total phosphorus, and total nitrogen (ammonia, organic, nitrate, and nitrite nitrogen) according to methods described by Fishman and Friedman (4). Storm-runoff loads of each constituent, in kilograms (kg), were calculated by multiplying the volume of streamflow associated with the sample by the constituent concentration (5).

Storm runoff (in cm derived from m^3 per km^2) and storm-load yields (in kg per km^2) from each subwatershed were calculated by subtracting the streamflow (in m^3) and load (in kg) determined at the site upstream from the subwatershed from those determined at the site downstream from the subwatershed and dividing the difference by the subwatershed drainage areas (in km^2). Storm runoff (or runoff quantity) and storm-load yields (or runoff quality) of each subwatershed are the data used for evaluating the downstream differences in storm runoff as affected by subwatershed basin characteristics.

RESULTS AND DISCUSSION

DIFFERENCES IN STREAM DISCHARGE BETWEEN SUBWATERSHEDS

The hyetograph and hydrograph during the first 24 hours of storm 4 are shown in figure 2. The figure illustrates the downstream differences in the shape of the Lamberts Creek hydrograph between data-collection sites. The shape of the hydrograph at data-collection site 1 is affected by storm runoff from the impervious area within the highly-urbanized subwatershed 1. The shape of the hydrograph includes a large sharp peak in discharge occurring over a short time period which is typical of urban runoff hydrographs (6).

The long and flat shape of the hydrograph at data-collection site 2 (compared to data-collection site 1) is a result of the surface-water storage capacity in both the unchannelized wetland and Goose Lake. The storm-sewer discharge from subwatershed 1 enters the wetland, disperses throughout it, and is temporarily stored. Storm runoff from the urban area in the southeast section of subwatershed 2 enters Goose Lake, disperses throughout, and also is temporarily stored.

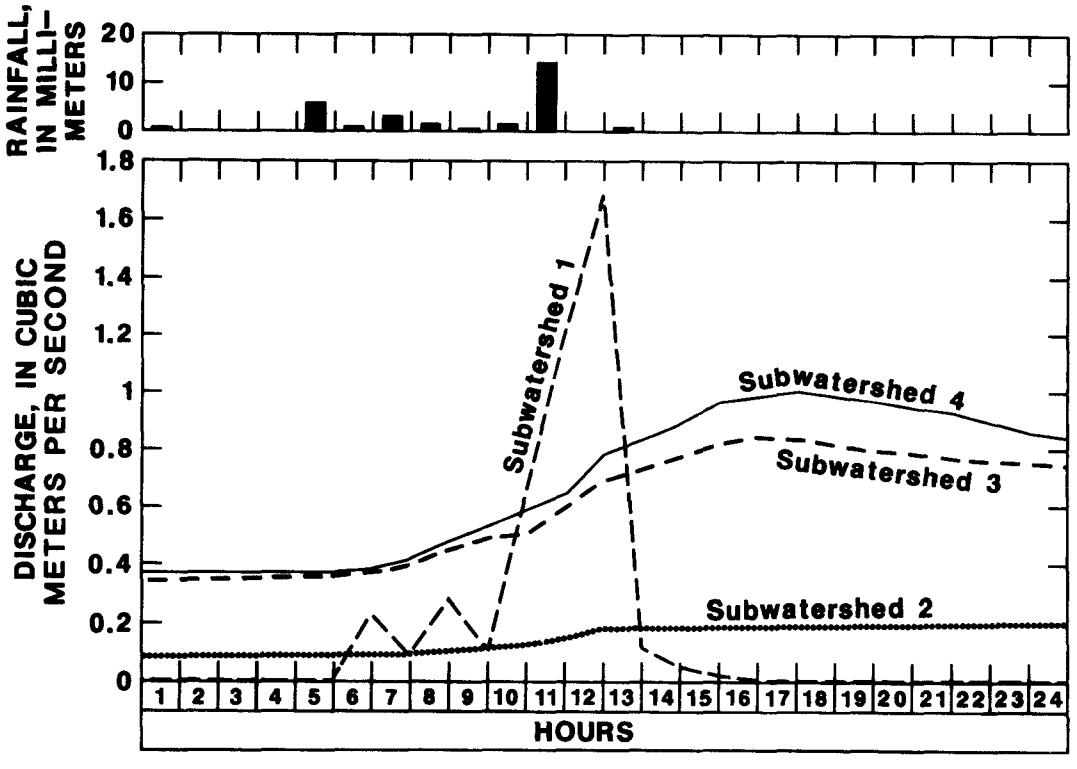

Figure 2. Total-hourly rainfall and average-hourly discharge for data-collection sites during the first 24 hours of storm 4.

The shape of the hydrographs at data-collection sites 3 and 4 have greater ascending slopes as compared to data-collection site 2 because of storm runoff that enters the channel, primarily from subwatershed 3. The increase in stream discharge between data-collection sites 2 and 3 differs substantially from that between data-collection sites 3 and 4. This large increase in stream discharge is primarily attributable to storm runoff from the steeply-sloped, sandy-soil basin in subwatershed 3.

DIFFERENCES IN STORM RUNOFF BETWEEN SUBWATERSHEDS

Storm runoff and storm-load yields from subwatersheds during each of the 12 storms are shown in figure 3. The storm runoff from subwatershed 3 is generally greater than that from the other subwatersheds during all 12 storms. The greatest difference in basin characteristics between subwatershed 3 and the other subwatersheds is basin slope (table 1). Basin slope within subwatershed 3 is more than twice that of any of the other subwatersheds. The rate at which subsurface runoff moves within the basin is directly proportional to basin slope, all else being equal (7). Soils generally are more permeable on steep slopes than on gentle slopes because fine particles have been removed from steep slopes allowing subsurface-flow velocity that is proportional to basin slope. Soils on gentle slopes contain fine particles which impede lateral subsurface flow because the soil is less permeable (8). The steeply-sloped basin and sandy soils in subwatershed 3 allow for substantially greater storm runoff during the hydrograph than that observed in the other subwatersheds.

The initial hypothesis was that storm runoff would be greatest from the predominantly urbanized subwatershed 1 because of the amount of impervious area associated with urbanized basins. However, the relatively flat basin slope in subwatershed 1 allows the few pervious areas to become temporary surface-water storage areas of runoff during storm periods. The retention of runoff in the pervious areas allows for substantial inflitration of runoff into the soil and, therefore, reducing the amount of runoff leaving the basin.

During the hydrograph period following the rainfall event (36 millimeters) of storm 6, the amount of storm runoff leaving subwatershed 2 (at data-collection site 2) was less than the amount of storm runoff entering the subwatershed (at data-collection site 1). This can be attributed to two factors related to surface-water storage capacity for the storm: (1) storm runoff from within the subwatershed was small, as a result of low antecedent-soil moisture conditions, and a portion of the storm runoff was stored within the wetland and Goose Lake and (2) a portion of the storm runoff entering the subwatershed from subwatershed 1 also was stored in the wetland. Surface-water storage capacity in the subwatershed was substantially greater for storm 6 compared to other storms because of a 24-day dry period prior to storm 6.

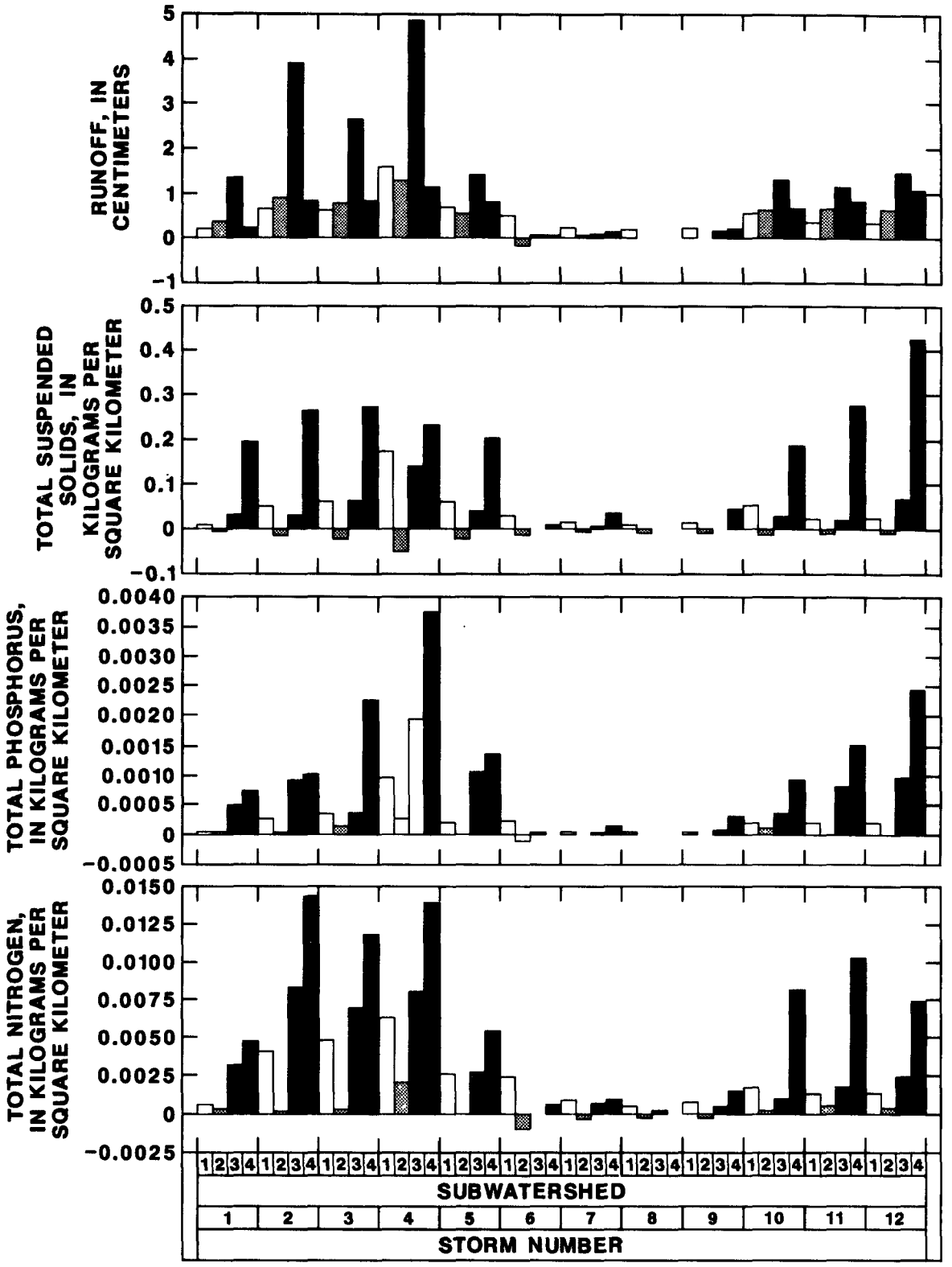

Figure 3. Storm runoff and storm-runoff yields for each subwatershed during the 12 storms of 1985.

DIFFERENCES IN STORM-LOAD YIELDS BETWEEN SUBWATERSHEDS

Storm-load yields of all three constituents were generally highest from subwatershed 4 during the 12 storms. The high yields are probably the result of channel erosion within the subwatershed because (a) the channel is steeply sloped, nearly twice that of the other subwatersheds, (b) high flow is common in the channel as a result of the large amount of runoff entering the channel from subwatershed 3, and (c) the organic soils in the channel are highly erosive. Erosion of organic soils is typically associated with high concentrations of suspended solids, phosphorus, and nitrogen (8). Although organic soils are present in the channel reaches of subwatersheds 2, 3, and 4 the steeply-sloped channel and high flow in the channel reach of subwatershed 4 results in substantial greater channel erosion compared to the other channels reaches.

Storm-load yields of all three constituents leaving subwatershed 2 (at data-collection site 2) during the 12 storms were generally less than the storm-load yields entering the subwatershed (at data-collection site 1). The storm-load yields leaving the subwatershed are lower because (a) loads from subwatershed 1 are retained in the wetland and (b) loads from the urbanized area within the subwatershed are retained in Goose Lake. Retention of suspended material in the wetland (and lake) is directly related to a decrease in water velocity as the water enters the wetland (and lake). As flow velocity decreases, sedimentation increases (9). Vegetation in a wetland tends to decrease water velocity beyond that of pooling alone (such as in the lake) and promotes fallout of suspended material (10). Nitrogen and phosphorus from the suspended material is deposited within the wetland (and lake) and removed from the water column (11). The wetlands in subwatersheds 3 and 4 are not as effective in retaining suspended material because of channelization.

CONCLUSIONS

Downstream differences in storm-runoff quantity and quality in Lamberts Creek are affected by four basin characteristics of the subwatersheds; urban land use (impervious areas), presence of wetlands (surface-water storage), basin slope, and channel slope. Storm-runoff quantity is smallest in subwatersheds that have (1) small amounts of urban land use (impervious area), minimizing surface runoff, (2) gentle basin slopes, impeding subsurface flow, and (3) large amounts of surface-water storage (wetlands), temporarily retaining storm runoff. Storm-runoff loading of total suspended solids, total phosphorus, and total nitrogen are smallest in subwatersheds that have (1) gentle channel slopes, minimizing channel erosion and (2) large wetland areas, allowing for retention of loads through sedimentation. Channelized wetlands are not as effective as unchannelized wetlands in storing storm runoff or in retaining loads.

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SOME THOUGHTS ON THE SELECTION OF DESIGN RAINFALL INPUTS FOR URBAN DRAINAGE SYSTEMS

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ABSTRACT

The art of runoff simulation has made significant advances during the past decade, owing to improvements in computer hardware and software. Further development would be however impaired if the information content of rainfall input, used in runoff simulation, is not improved. Dynamics of rainfall fields is far too complex to be adequately represented by a design storm. The design storm concept needs to be replaced by continuous simulation using distributed rainfalls in time and space for input.

INTRODUCTION

One of the key objectives of urban hydrology is the capability of predicting hydrographs or peak discharges of storm runoff corresponding to a certain frequency of occurrence. Statistical analysis of long series of observed runoff events could provide an approach to deal with the problem. However, measured data are seldom available. It would seem reasonable to assume that a plausible alternative approach would be to derive the runoff series from observed rainfall. All that is necessary to accomplish this task is to simulate the rainfall-runoff process reasonably well. This approach, formalized perhaps for the first time by Mulvaney(1) in 1847 when he introduced the rational formula, has become today the main tool of modern hydrology. Throughout the years it has also become accepted that the rainfall input, which drives the equations of our mathematical models, is known, that it can be sufficiently well described. Consequently, the major effort has been expanded on building and refinement of runoff models while the study of rainfall characteristics has been relatively neglected, or perhaps put more fairly, whatever new information about rainfall became available has been mostly ignored by runoff modellers.

One can reason that urban runoff, in contrast with rural runoff, will be relatively sensitive to the variability of rainfall input due to the fact

that urban catchments consist of many hydraulic components with fast response times, such as impervious surfaces and storm sewers. However, it was not until recently that researchers(2,3,4,5,6) began to study the effect of storm dynamics on urban runoff. The present paper attempts to discuss the problem of rainfall input selection for the use in runoff simulation in view of the presently available information on rainfall properties. A short overview of the historical development of rainfall and runoff concepts is given, followed by a discussion of problems related to the determination of rainfall over a catchment in terms of its temporal and spatial distribution, and of its frequency. The non-correspondence of rainfall and runoff frequencies is also addressed.

RAINFALL AND RUNOFF CONCEPTS

It may be useful to start the discussion of rainfall-runoff modelling by recounting first a few facts from the history of hydrology. The reader, equipped with a historical perspective, may judge for himself the state of our present day knowledge and practice.

HISTORICAL BACKGROUND

According to Biswas(7) the earliest reference to a rain gage was made by Kautilya in his book about the science of politics and administration (in India) written towards the end of the fourth century B.C. However, the causality of rain and runoff was not recognized for many centuries to come. A typical view upon the origin of water in streams during the Middle Ages is offered, for example, by Leonardo da Vinci(7): "...if the body of the earth were not like that of man, it would be impossible that the waters of the sea, being so much lower than the mountains, could by their nature rise up to the summits of these mountains. Hence it is to be believed that the same cause which keeps the water at the top of the head in man keeps the water at the summits of the mountains."

It was not until the seventeenth century that Perrault and Mariotte(7) established a quantitative link between rainfall and runoff. By measuring both rainfall and river discharge they showed that rainfall was adequate to supply the water flowing in streams and rivers. A first proposal to predict discharge on the basis of rainfall appeared in 1850 when Mulvaney published a paper(7) describing the use of a well known rational formula. Then, it was not again until 1932 that a new concept of the rainfall-runoff relationship was introduced, the unit hydrography concept. Since the nineteen sixties the development of rainfall-runoff modelling took on a new face. This was made possible chiefly by advances made in computer technology and the proliferation of numerical techniques for solution of unsteady flow equations. Distributed models became a possibility.

PRESENT-DAY PRACTICE

Today a catchment is viewed upon as a complex system with multiple inputs and outputs. Runoff simulation models are in essence logically arranged mathematical relations between major variables controlling runoff. These relations can be express in form of state and output equations. The

state equations are usually continuity equations and the output equations are often given in form of a momentum equation or a rating curve. The input which drives these equations is a hyetograph.

The hyetograph used in most simulation models usually belongs to one of the following groups: (1) a block of rainfall of constant intensity, based on IDF curves and the time of concentration concepts, (2) a synthetic design storm hyetograph, such as the Chicago design storm, and (3) observed point-rainfall hyetograph. In all three cases the rainfall is usually assumed to be uniformly distributed over the catchment area, and movement of the storm is not considered. The frequency of the generated runoff is considered to be the same as that of the used rainfall input.

It would appear that a gap has developed between our best distributed models, capable of simulating unsteady flow throughout a catchment, and the quality of input information we are able to supply to these models. This view follows from the results of recent studies (3,5) showing that the dynamic properties of rainfall are significant for the runoff generating processes in urban setting.

STRUCTURE OF RAINFALL FIELDS

Far from being stationary and isotropic, as depicted by idealized design storms, the actual rainfall fields are complex dynamic systems composed of rainfall cells undergoing periods of grows and decay. Three aspects of rainfall dynamics will be discussed here. The movement of the storm over the catchment, the areal distribution of rainfall, and the temporal distribution of rainfall at single points on the ground.

STORM MOVEMENT

The velocity of raincells producing heavy rainfall over urban catchments was observed to vary between 7 m/s and 25 m/s (5,8). The size of the raincells was calculated by Niemczynowicz (5) to vary between 2.0 km^2 and 7.6 km^2. The storm movement effects the runoff in such a way that the maximum discharge is produced when the direction of storm movement coincides with the main direction of sewers in the catchment, and the velocity of storm movement is about the same as the sewer flow velocity. According to Niemczynowicz (5) the peak discharge from a storm moving downstream may be about 80% higher than discharge which derives from a storm moving upstream, or about 30% higher than the peak discharge caused by a stationary storm.

Assessment of the probability of occurrence of storm movement in a certain direction with a certain speed is difficult at the moment. It is obvious, however, that this probability influences the probability of occurrence of peak discharge.

AREAL DISTRIBUTION

It is generally recognized that areally averaged intensities are lower than point intensities. It is not an uncommon practice to base design storms on areal IDF curves or area reduction factors applied to point

intensities. This approach, however, still cannot account for the actual spatial distribution of rainfall intensity during a storm event. This spatial variability is often significant in terms of runoff production (5).

TEMPORAL DISTRIBUTION

As a result of internal dynamics and the storm's movement both areal and temporal distributions of rainfall intensity are very seldom close to uniform. Smaller catchments are believed to be generally more sensitive to temporal (and spatial) variability of rainfall than very large basins. Yen and Chow (9) defined a "small" basin as such that "...its sensitivities to high intensity rainfalls of short durations and to land use are not suppressed by the channel-storage characteristics". Most urban catchments will display a high degree of sensitivity to temporal variation of rainfall. Only a few researchers have attempted to model the temporal distribution of rainfall (10,11). Such models can be subdivided into: (a) models assuming that the rainfall series consists of internally independent random values, (b) the Markov chain type models, which allow for sequential dependence of data and (c) the time series models, which try to preserve also trends, periodicities and persistence observed in rainfall series. Niemczynowicz (5) gives a review of existing models.

THE DESIGN STORM CONCEPT

Experience shows that for a given rainfall total depth over a particular duration, the hyetograph may differ considerably for different storms. Yen and Chow (9) stated that "one has to yet find two natural rainstorms that are identical. Therefore, the design hyetograph for drainage facilities, which are expected to serve future needs, can only be estimated through statistical analysis of past records".

The design hyetograph, or the socalled design storm, is a synthesized rainfall sequence of a duration varying from several minutes to several hours, supposedly preserving some statistical characteristics of observed (usually point data) rainfalls. The design storm is assigned a return interval, usually on the basis of elementary statistical analysis of a simple parameter, such as the rainfall total or average intensity over a given duration (IDF curves). The return interval of the computed design discharge is deemed to be the same.

This procedure, while expedient, has a number of serious flaws. First, the design storm concept does not account for storm movement or spatial variability. Second, temporal variation of rainfall intensity is neglected or oversimplified. Third, the return interval of the design storm is a fictitious value which is not based on any probability considerations of occurrence of real sequences of rainfall, storm movement (direction and speed) and spatial variability. Lastly, the assumed equality of return intervals of a design storm and the generated runoff is only valid for blocks of rainfall of constant intensity uniformly distributed in space, and for hydrographs having exactly the same initial antecedent moisture con-

ditions. Since such conditions are unlikely to occur naturally, the design storm concept does neither allow computation of the correct return interval of the "design" discharge, nor does it provide for the estimation of its tolerance limits.

CONTINUOUS SIMULATION

The foregoing discussion shows that conventional frequency analysis, such as applied to maximum discharge series, cannot be used to analyze statistical character of rainfall fields. This is because unlike the peak discharge, which can be considered as a point variable, rainfall is multi-dimensional, having three space and one time coordinates. Because of this complexity there is no technique or sufficient data available as yet to assign a probability value to an observed rainfall event. A number of researchers (3,5,12) suggested, as a possible way of overcomming the shortcomings of the design storm procedure, to use long rainfall series in continuous simulation of runoff. This approach allows making statistical analysis of the output - the discharge series, without actually assigning probability values to the input rainfall series. With the advent of micro-computer hardware and software the continuous simulation is likely to replace the conventional design storm, a one event type of analysis.

CONCLUSIONS

Rain and runoff have been part of man's everyday experience since the origin of man. Yet, it took over 2200 years, according to preserved written information, for man to progress from making first measurements of rainfall to the stage when he realised the connection between the two phenomena, and was able to make an estimate of runoff from rainfall by means of rational formula. The rational formula requires only a block of uniform rainfall as an input. Today, the rational formula is still extensively used along a number of relatively sophisticated computerized models. While the degree of sophistication of these models has been steadily increasing over the past decade, the information content of rainfall input, which drives these models, has stagnated at the design storm level. The design storm concept cannot adequately represent the complexity of rainfall fields dynamics. The return interval of design discharge cannot be inferred correctly from the design storm. The return interval assigned to a design storm is a rather fictitious value itself. Future research should concentrate on continuous simulation, and the use of upgraded information on rainfall input in terms of its spatial and temporal variations. Continuous simulation, which uses long series of realistic rainfall events, does not require assignment of a probability of occurrence to individual events. Furthermore, the probability of a given design discharge is estimated by statistical analysis of the simulated discharge series, rather than being considered identical with the rainfall event.

The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

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FIELD MEASUREMENT AND MATHEMATICAL MODELING OF COMBINED SEWER OVERFLOWS TO FLUSHING BAY

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ABSTRACT

Virtually all of the direct loading of conventional pollutants to Flushing Bay, a tidal embayment connected to the East River (Figure 1), comes from combined sewer overflows (CSOs). As part of the New York City Department of Environmental Protection's effort to improve the water quality of Flushing Bay and Creek, a comprehensive study that included modeling of CSO discharges and water quality was performed. This paper discusses three aspects of the overall study: (1) measurement and evaluation of CSO discharge and pollutant loadings in tidally affected outfalls, (2) application of the Stormwater Management Model (SWMM) to simulate the major CSO, and (3) development and application of the Storage Pumping Model (SPM) to evaluate CSO retention for the major CSO discharge.

The method for measuring the net (nontidal) CSO discharge and pollutant loading consisted of profiling velocity over the depth of flow in the outfall and compositing a sample based on the flow in the depth intervals. The largest outfall, CS4, which has three conduits or barrels with dimensions of 18.5 ft (width) by 10 ft (height), accounted for more than half the total CSO discharge and load to Flushing Bay and Creek.

The CS4 system, which has a drainage area of 7409 acres, was modeled in two stages using SWMM: (1) the upper half of the system, which is not tidally affected, was calibrated to field survey data collected at sampling locations approximately 12,000 ft upstream of the outfall bulkhead; (2) both the upper and lower parts were then verified to data collected at the outfall during three surveys. SWMM was applied to evaluate alternative locations for CSO control; Extran was used to hydraulically analyze in-line storage within the tidally affected outfall referred to as the Kissena Corridor storm sewer.

SPM was developed to determine the appropriate capacity for a storage facility that would pump CSO retained during storms to one of New York

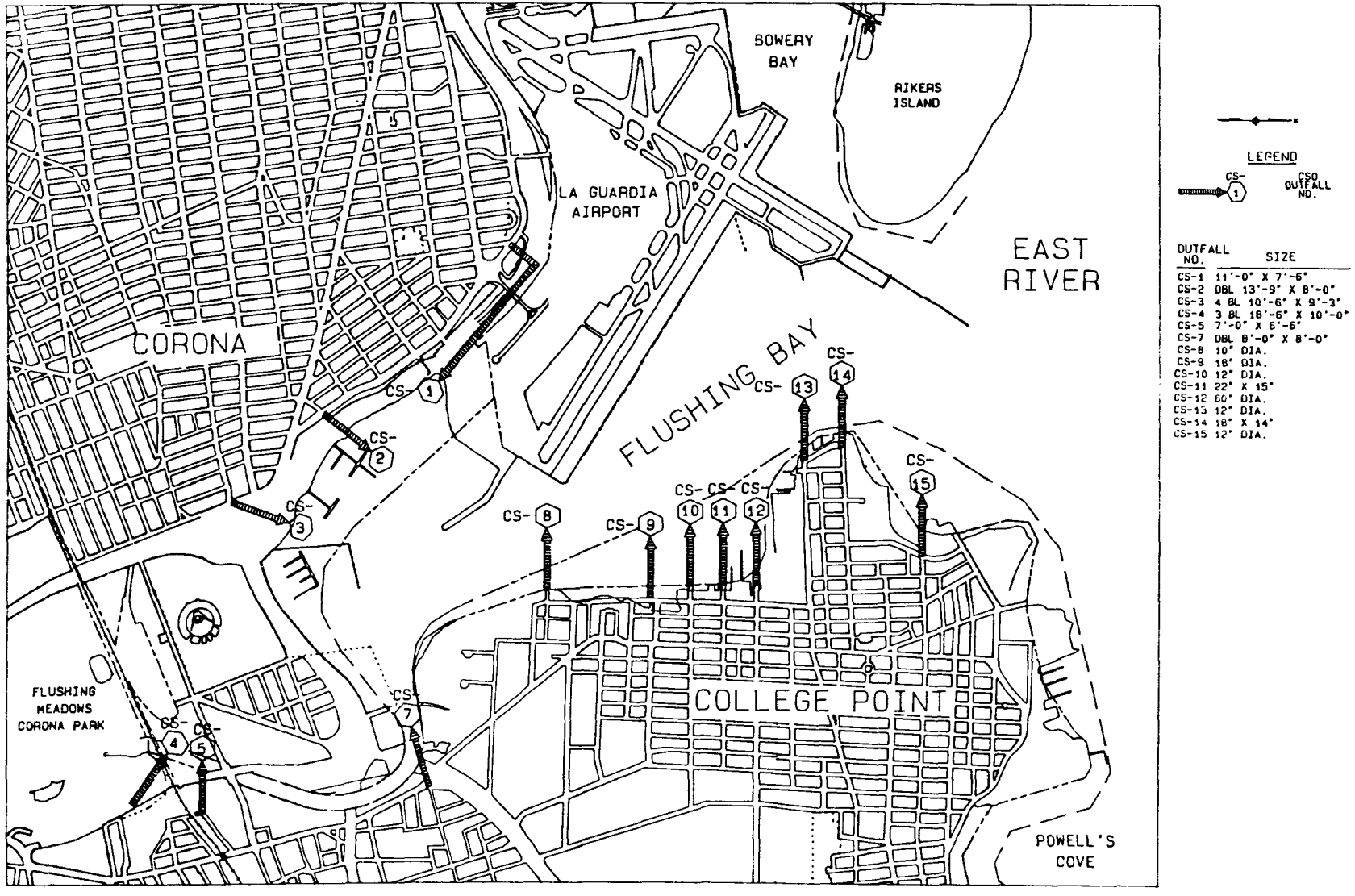


Figure 1. Location of combined sewer overflow (CSO) discharges to Flushing Bay and Creek.

City's water pollution control plants. The SPM results projected that a 40 million gallon (MG) offline storage facility would yield a 58% reduction in CSO discharge and 73% and 76% reductions in BOD and total suspended solids loading, respectively. Water quality models of Flushing Bay and Creek projected a substantial improvement in dissolved oxygen and coliform bacteria concentrations from such a facility, including disinfection of overflows at CS4.

INTRODUCTION

The work described herein was performed by Lawler, Matusky & Skelly Engineers (LMS) under subcontract to URS Company, Inc. The relationship of the work to the Flushing Bay Water Quality Facility Plan is shown schematically in Figure 2. Since the CSO discharges to Flushing Bay are affected by the tide, sampling and flow measurement techniques were devel-

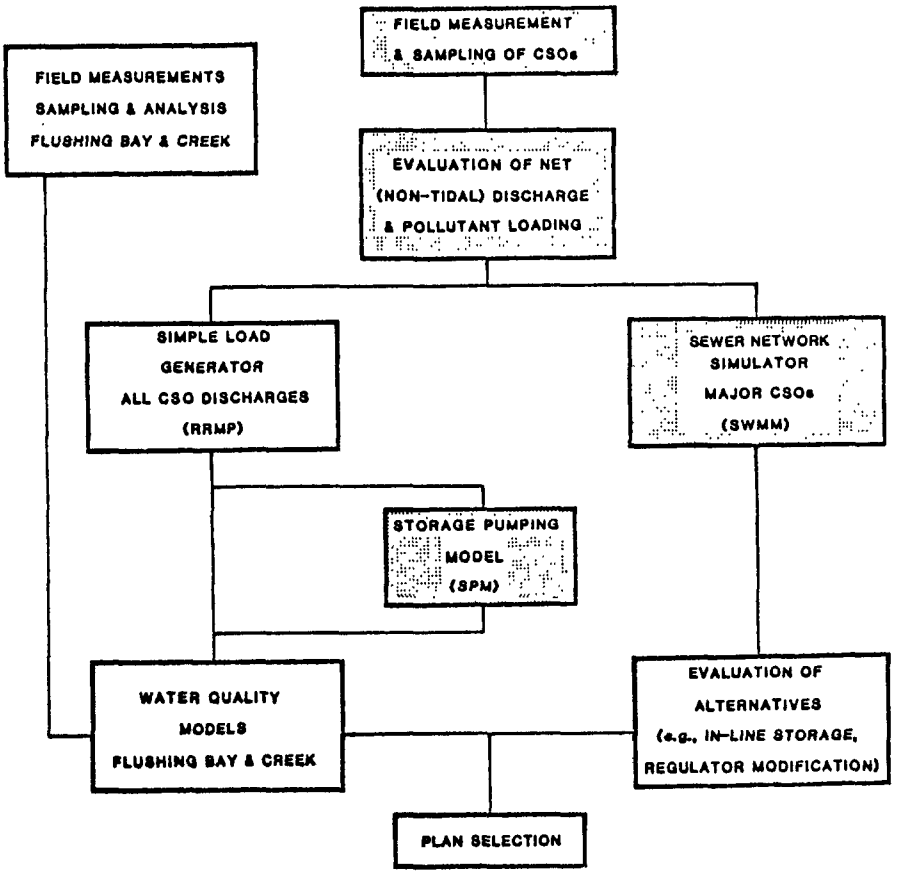


Figure 2. Linkage of this paper to Flushing Bay Water Quality Facility Plan.

oped to evaluate the net (nontidal) discharge and pollutant loadings. The sewer networks of the largest CSO, which accounted for over half the total CSO discharge, was simulated by using the Stormwater Management Model (SWMM). The purpose of SWMM was to assist in evaluating CSO abatement alternatives (e.g., in-line storage) as well as to provide a check on the simpler Rainfall Runoff Modeling Program (RRMP) used to generate loads for all CSO discharges as input to the water quality model (1). The Storage Pumping Model (SPM) was developed to evaluate the relationship between CSO storage capacity and the reduction in loadings of biological oxygen demand (BOD) and total suspended solids (TSS). Calibrated mathematical models of water quality were applied to project the water quality improvement from an array of alternatives, and a facility plan for CSO abatement was selected.

CSO MEASUREMENTS

For water quality modeling purposes and the planning and design of CSO abatement facilities, the discharge and pollutant loading from CSOs must be measured accurately. The typical Flushing Bay CSO outfall consists of one or more rectangular sewers from 10 to 18 ft wide and approximately 8 to 10 ft high with the crown above high water elevation and the invert below low water. Salinity, tidal flow, and tidal stage (which has a mean range of 6.5 ft) extend into the outfall up to the regulators. Because of the many complex interconnections of sewers and regulators, the combined sewage flow was reregulated and the outfall was the only location where the CSO discharge could be sampled and measured.

The method for CSO sampling performed at the shoreline bulkheads was devised to measure the net discharge and loading into the receiving water body. Preparation of the sites was required. Platforms were constructed outboard of the bulkhead for access by the sampling crews (Figure 3). Probes for velocity and conductivity meters and sampling hoses were attached to the bottom of stadia rods that were raised and lowered through braces mounted to the platform.

The measurement and sampling procedures were as follows:

1. Velocity, conductivity, and temperature were measured at depth intervals generally 1 to 2 ft (or less at the surface).

2. A water sample was pumped from each depth position where the velocity was positive (into the receiving water).

3. Flow-weighting factors equal to the ratio of the positive flow in a depth interval to the total positive flow were calculated.

4. Aliquot volumes of sample were composited according to the flow-weighting factors.

The frequency of sampling during a storm event was geared to the CSO discharge rate, which depended on the rainfall pattern, ambient tide, and other conditions. High velocity, turbidity, and negligible salinity that

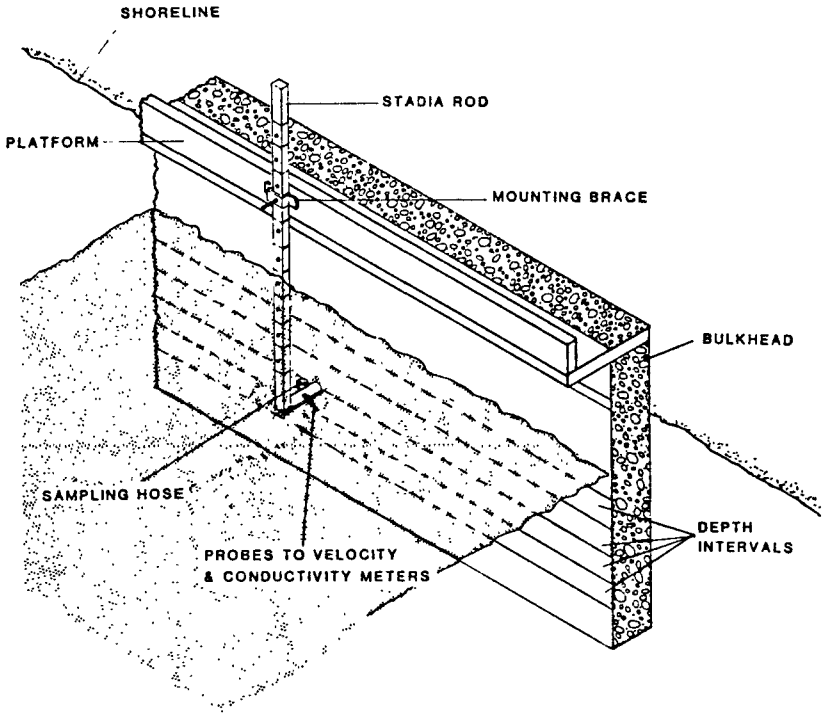


Figure 3. Sketch of typical CSO sampling station.

occurred over the entire depth just after intense rainfall marked the onset of a "first flush," when measurements and sampling were performed as rapidly as possible. Sampling frequency was approximately every 15 to 30 min during a first flush and 30 to 60 min at other times.

An electromagnetic meter was used to measure velocity profiles manually at all tidally affected CSO locations. At the largest outfall (CS4), acoustic flow-measuring equipment and manual measurements were used for the second and third surveys; manual measurements alone were used for the first survey and acoustic equipment alone was used for the fourth through seventh surveys. The equipment was installed in two sewers with ultrasonic signal transmission paths at four depth positions in each (Figure 4). Depth and velocity data were output to a console printer and a data logger.

Comparison of the velocity measurements between the electromagnetic meter and the acoustic equipment generally showed differences of less than 20% over the range of velocity up to approximately 2 fps. The electromagnetic meter produced lower readings than the acoustic equipment when velocities were greater than 2 to 3 fps. The manually deployed electromagnetic meter was probably in error at these high velocities because of bending of the stadia rod, which yielded a component of the normal velocity, and/or debris getting caught on the probe, which interfered with the meter functioning. Measures were taken to minimize these interferences; however, extremely high outfall velocities necessitated the removal of the stadia rods to prevent damage.

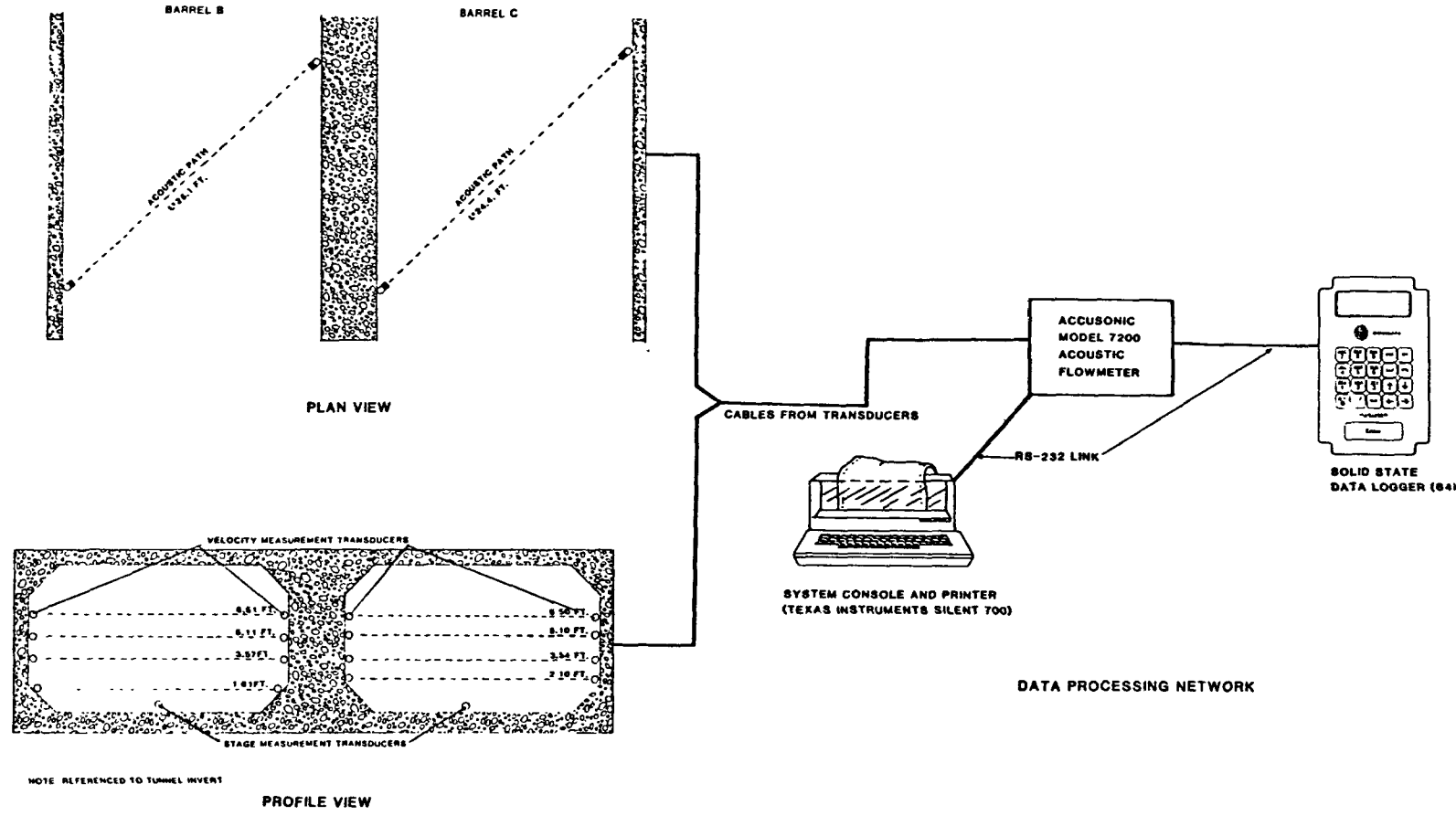


Figure 4. Acoustic flowmeter installation at CSO outfall CS4.

Acoustic flow-measuring equipment, although expensive to use, has the following advantages: it measures laterally averaged velocity; it is accurate at velocities (up to 10 fps) that would bend manually deployed rods; and measurements are automatically recorded at short intervals (e.g., 5 min).

Combined sewers were sampled during one survey at locations upstream of the tidal effects. The dual purpose of this survey, which was conducted in the Kissena Corridor system upstream of CS4 (sampled routinely during this CSO survey), was (1) to compare upstream data with the data collected at the CSO outfall, and (2) to obtain data from locations that would facilitate the calibration of SWMM.

Six CSO outfalls were surveyed during three to seven storm events. Flow-weighted composite CSO samples were analyzed in the laboratory for BOD, suspended solids, coliforms, and nutrients.

The tidal phase affected the timing of peak flow because flood tide generally held back the CSO discharge and slack or ebb tide allowed it to pass. Interaction of the tide and rainfall can cause various sequences of flushing. For example, an initial flush can be cut short by an incoming tide and then a flush will occur later during the outgoing tide. Although the surging turbid discharge did not always occur at the beginning of a storm, sampling crews usually noticed peak discharge and solids loading that generally fits the term "first flush."

EVALUATION OF NET DISCHARGE AND POLLUTANT LOADING

The results of the CSO surveys are summarized in this paper for the largest outfall, CS4, which discharges at the upstream end of Flushing Creek. Data for other outfalls are presented in LMS 1986 (2). The CS4 outfall has a total drainage area of 7409 acres, or 44% of the entire drainage area to the bay and creek. Sewers convey stormwater runoff from approximately 20% of this primarily residential area; combined sewers are used for the remainder.

Two of the three CS4 outfall barrels (width 18.5 ft, height 10.0 ft) were sampled; the remaining barrel, identified as CS4A, was assumed to be identical to CS4B. Water quality data for these two sampling stations are summarized in Table 1. The mean concentrations reflect tidal dilution since the outfall - in essence the Kissena Corridor storm sewer - is tidal for approximately 12,000 ft from the bulkhead, and the total mean tidal volume is approximately 22 million gallons (MG). The fourth and fifth surveys conducted during light rainfall have BOD_5 concentrations approaching ambient Flushing Creek levels. The BOD concentrations for periods of greater rainfall, which are relatively low compared to the normal range for CSO (3), are attributable to the predominance of storm water as opposed to sanitary wastewater. TSS concentrations show a relationship with total rainfall, suggesting that stormwater runoff, which flushes solids from the streets as well as scours deposited solids from the combined sewers, appears to control the solids loading.

TABLE 1. SUMMARY OF WATER QUALITY DATA FOR SAMPLING OF LARGEST CSO OUTFALL

		Mean concentration*								Mean ratios		
Station	Survey	Total rain (in.)	Total BOD_5 (mg/l)	TSS (mg/l)	Coliforms (10^6 counts/ 100 ml)	Ammonia (mg/l)	TKN (mg/l)	NO_2+NO_3 (mg/l)	TP (mg/l)	Filtered: total BOD_5	TVSS: TSS	Fecal: total coliforms
CS4B	1	1.02	9.5	67.2	0.85	1.34	8.66	0.65			0.25	0.10
	2	0.80	23.8	69.0	0.84		7.39	0.90		0.58	0.54	0.43
	3	1.24	19.5	52.1	1.48	2.64	6.81	0.28	0.86	0.40	0.44	0.39
	4	0.09	8.3	12.5	2.35	3.66				0.56	0.76	0.37
	5	0.31	10.0	17.3	2.02	3.96	6.76	0.53	0.75	0.82	0.59	0.19
	6	1.35	13.8	75.5	1.49	1.49	4.00	0.21	1.11	0.65	0.49	0.46
	7	2.66	14.5	55.8	1.52	2.75	3.97	0.38	0.52	0.55	0.40	0.19
CS4C	1	1.02	8.8	47.2	0.54		8.79	1.04			0.22	0.12
	2	0.80	15.2	38.5	0.74		4.63	0.45		0.45	0.45	0.28
	3	1.24	16.8	70.0	2.18	2.70		0.30	0.60	0.61	0.40	0.10
	4	0.09	7.0	22.5	2.55	3.73				0.68	0.67	0.20
	5	0.31	9.4	15.8	1.56	2.96	6.29	0.50	0.67	0.71	0.57	0.30
	6	1.35	13.0	47.6	2.07	1.78	4.80	0.20	1.02	0.56	0.48	0.18
	7	2.66	16.4	61.4	0.98	3.07	4.43	0.37	0.65	0.62	0.43	0.22

*Arithmetic average of all samples except for coliforms, which are geometric averages.

The net discharge of CSO to the receiving water body, evaluated from the velocity measurements at each depth interval, is computed as the summation of flow in each depth interval:

$$Q_{net} = \sum_{i=1}^{n} v_i \, d_i \, w_i \qquad (1)$$

where

Q_{net} = net discharge (cfs)
v_i = velocity of depth interval (fps)
d_i = depth of interval (ft)
w_i = width of interval (ft)
i = interval number from bottom to water surface
n = total number of depth intervals

For the rectangular outfall sewers, the width is constant:

$$Q_{net} = w \sum_{i=1}^{n} v_i \, d_i \qquad (2)$$

The depth of an interval was normally constant except for the top interval, which may be less.

A schematic of typical vertical profiles of velocity in CSO outfalls illustrates two flow regimes (Figure 5). Regime 1 is defined as having unidirectional velocity at all depth intervals. Regime 2 is characterized as having an upper layer of seaward flow and a lower layer of landward flow. The timing of the tide and rainfall dictated the resulting flow regime. In general, unidirectional (Regime 1) flow was more common, particularly during flood and ebb and during high rainfall runoff that produced high positive velocity. Regime 2 was sometimes encountered when rainfall accumulated just before low water slack, causing CSO bypasses of low density water that flowed seaward as the higher density ambient (bay or creek) water began to flood. Heavy rainfall during a flood phase triggered high CSO outflow that essentially forced the tidal volume out of CS4 with Regime 1 flow just as the tide approached high water.

The total CSO discharge for a storm survey is computed as the integration of the net discharge over the time of measurements. To account for the total CSO discharge, velocity measurements should extend to the low water following the end of rainfall. The latter surveys covered full tidal periods that encompass all rainfall during the surveys and provide reliable data for evaluation of total CSO discharge.

The net pollutant loading is analyzed separately for Regime 1 and Regime 2 flow. The equations for net pollutant loading are presented below with reference to Figure 5, the schematic illustrating the two regimes.

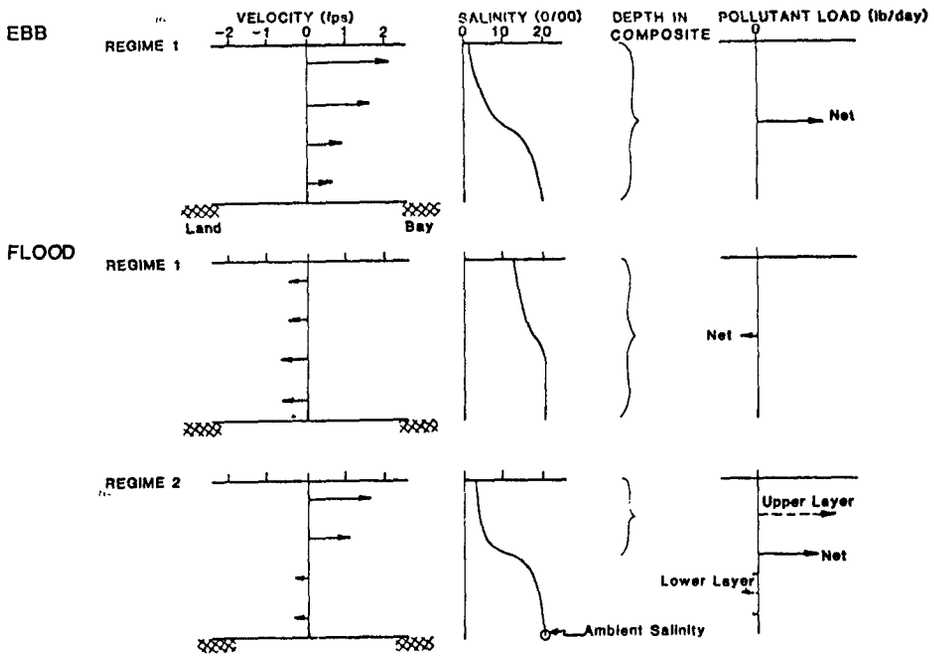


Figure 5. Typical vertical profiles of flow in CSO outfalls.

For Regime 1 flow, the calculation of pollutant mass loading is the product of the net discharge and the concentration of the flow-weighted composite sample.

$$M = Q_{net}\ C_{comp} \tag{3}$$

where

M = mass loading of pollutant (lb/day)

C_{comp} = pollutant concentration of flow-weighted composite (mg/l)

(The units are different for coliforms and conversion factors are necessary.)

Regime 2 is analogous to the two-layered transport in partially stratified estuaries. For Regime 2 flow, the pollutant mass loading is computed separately for the upper and lower layers of stratification.

$$M_{UL} = Q_{UL}\ C_{comp} \tag{4}$$

$$M_{LL} = Q_{LL} C_{LL} \quad (5)$$

where the subscripts UL and LL refer to upper layer and lower layer, respectively.

The upper layer loading is similar to Regime 1 in that the concentration is a flow-weighted composite of the depth intervals that have positive flow. The lower layer concentration is not measured by the CSO surveys but is computed based on the degree of mixing between the upper layer and the ambient water. The equation for the concentration of the lower layer (C_{LL}) is:

$$C_{LL} = f\ C_a + (1-f)\ C_{UL} \quad (6)$$

where

f = mixing factor, decimal fraction of ambient water in the lower layer

C_a = concentration of the ambient water outside the outfall

Conductivity and temperature measurements yield salinity that is used as a tracer of ambient Flushing Bay or Creek water. The salinities of the upper and lower layers are known from the measurements at each depth interval. Water quality data showed that the ambient salinity was generally found at the deepest sampling point (depth interval 1) within the outfall. Solving Equation 6 for salinity yields a mixing factor, f, that is then used to compute the lower layer concentration for pollutants (BOD, TSS, and coliforms). The total mass loading for Regime 2 is the sum of the upper and lower layers.

The variability of ambient pollutant concentrations was evaluated by sensitivity analysis: concentrations were reduced by 50% and increased by 100% and the net pollutant loading for several outfalls was computed. The resulting variation in total pollutant loading for the first two surveys was less than 5%, primarily because lower layer flow into the outfall was low in comparison to the total positive flow.

The total discharge, BOD_5, TSS, and total coliform loadings for seven surveys performed at the two CS4 sampling stations are summarized in Table 2.

The results for net discharge, BOD, and TSS loading for sampling station CS4B are illustrated for one survey in Figure 6; the rainfall and tidal stage are plotted vs time in Figure 7. The accumulation of rainfall through approximately 0200 hr on 5 November caused a substantial CSO outflow just before the end of flood tide, with a peak discharge of 370 cfs in CS4B occurring at high water slack. Although the BOD_5 concentration varied minimally over time, the maximum TSS concentration increased to over four times its average and resulted in a peak solids loading of almost 300 tons/day.

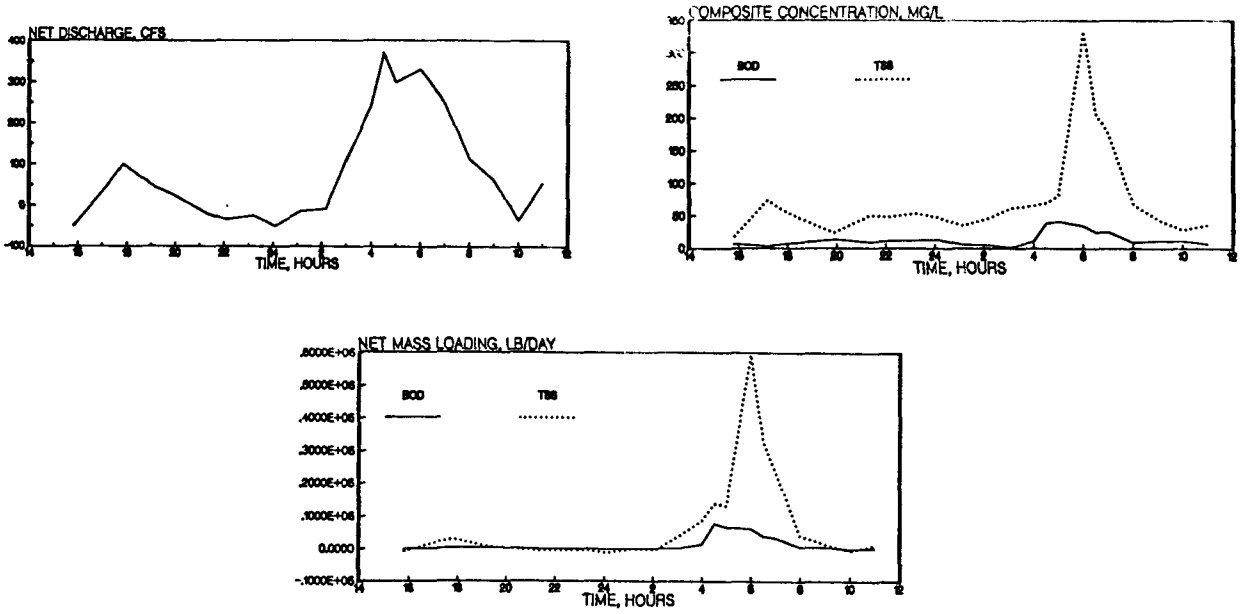


Figure 6. Net discharge, composite concentration, and net mass loading at Station CS4B for CSO Survey No. 6, 4-5 November 1985.

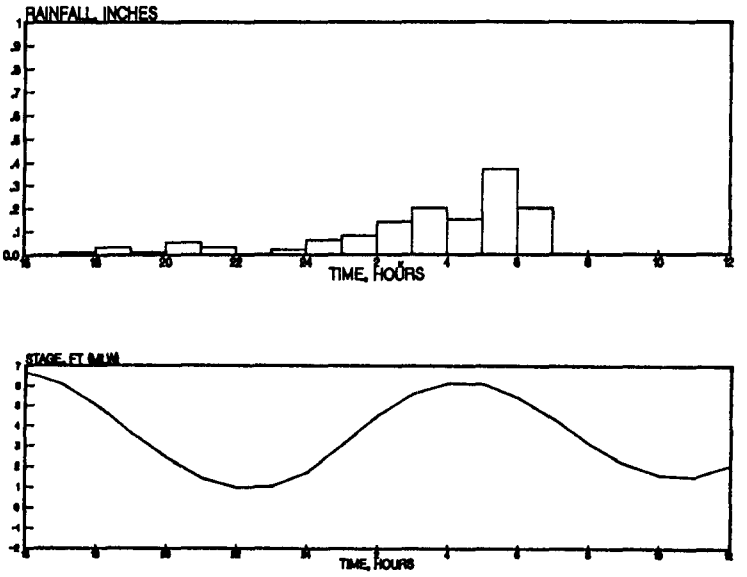


Figure 7. Rain and tidal stage vs time for CSO Survey No. 6, 4-5 November 1985.

CSO discharge from the upper half of the CS4 drainage area was measured for the same survey in the outfall (labeled Kissena Corridor storm sewer) and at the Regulator 40 bypass. The upstream field measurement locations are approximately 2.5 miles upstream of the outfall. The travel time to the outfall was measured as approximately 2.5 hr. Beginning at about 0100 hr on 5 November the upstream flow shows a response to rainfall attributable to the approximately 1200 acres with separate storm sewers (Figure 8). The bypass from R40 occurs after the Kissena Corridor flow has peaked. The flow at the CS4 outfall shows an attenuated peak flow with some fluctuation during the ebb tide.

The total flow and loadings from the upstream drainage area (referred to as the upper Kissena Corridor) are compared with the total CS4 outfall results. Flow from the upstream area is approximately 34% of the flow at the CS4 outfall; the BOD, suspended solids, and coliform loadings are about 15 to 20% of the pollutant loadings at the outfall. The reasons for these differences, which are shown in the schematics of the SWMM model network, are:

- Approximately 40% of the sanitary flow from the upstream area is conveyed by separate sewers to combined sewers in the lower Kissena Corridor that are then regulated.

- The portion of the combined sewer flow that is not bypassed at Regulator 40 and passes along the interceptor sewer is reregulated and can be discharged to the CS4 outfall.

- The lower Kissena Corridor has a greater percentage of imperviousness that yields higher runoff and CSO discharge.

STORMWATER MANAGEMENT MODEL (SWMM)

The version of the U.S. Environmental Protection Agency's Stormwater Management Model (known as PCSWMM3.2) adapted for the personal computer and distributed by Computational Hydraulics, Inc. (4) was applied to the CS4 system in two parts. First, the upper half of the CS4 or Kissena Corridor system, which is unaffected by tide, was modeled and calibrated to field survey data; second, SWMM output from the upper part was used as input to the model of the lower Kissena Corridor, which was verified to data collected at the CS4 outfall during three surveys.

The order in which the SWMM modules were applied to the CS4 system is shown in Figure 9; pertinent characteristics are listed in Table 3. Schematics of the SWMM networks for the upper and lower Kissena Corridor CSO systems are shown in Figure 10. Our approach was to use the Transport module for simulations of the field survey periods; however, a hydraulic analysis of regulators and interceptors was performed using Extran. As most of the regulators in the CS4 system are diversion chambers having transverse or side-flow weirs, the hydraulic capacity of key regulators was analyzed to evaluate any variations in flow to the interceptor as a function of stage

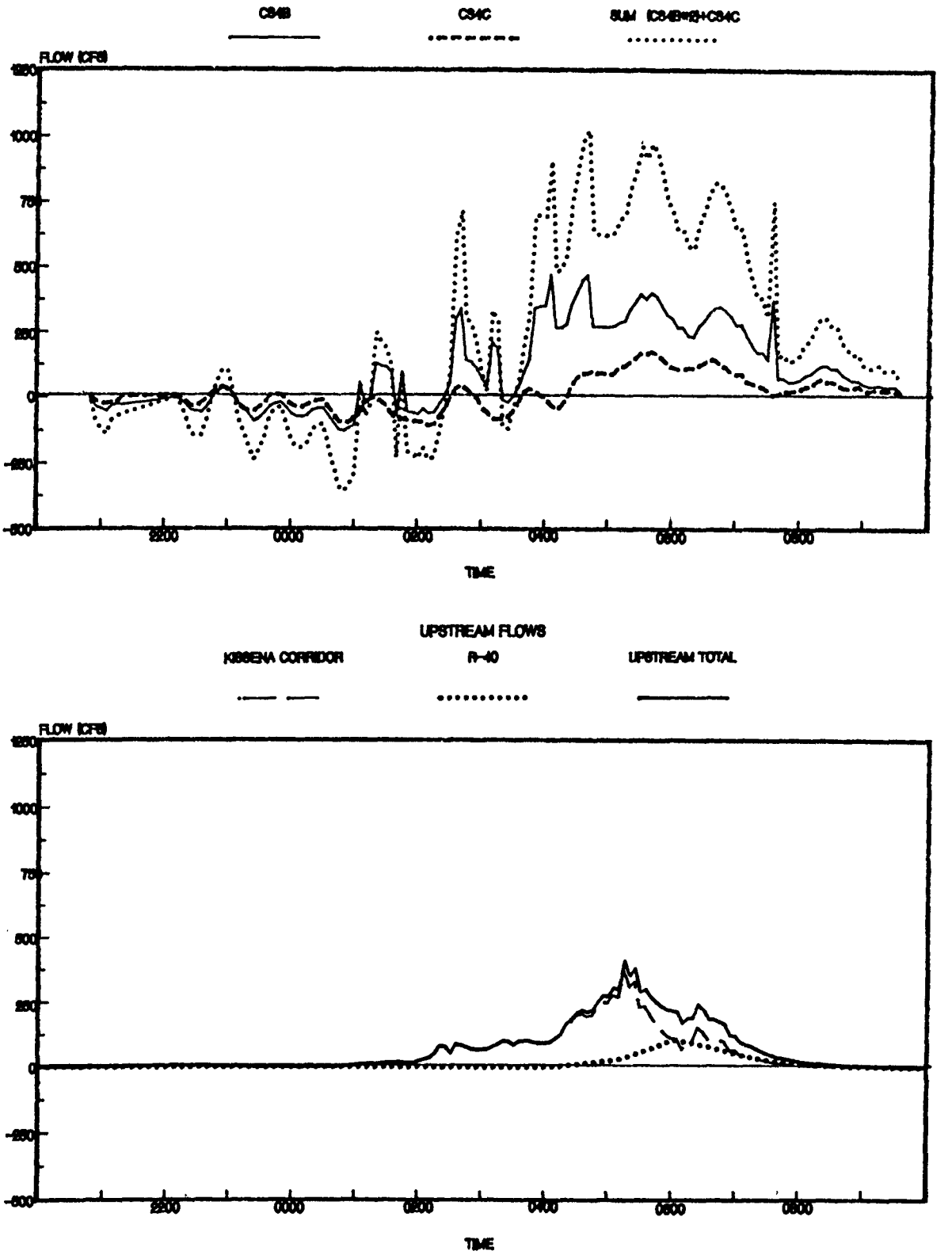


Figure 8. CSO Survey No.6, 4-5 November 1985, comparison of flows at CS4 and upstream locations.

TABLE 2. SUMMARY OF WATER QUALITY DATA FOR SAMPLING OF LARGEST CSO OUTFALL

		Discharge		BOD_5		TSS		Total Coliform	
Station	Survey	Flow (cfs)	Volume (MG)	Loading lb/day)	Load (lb)	Loading (lb/day)	Load (lb)	Loading (counts/day)	Load (counts)
CS4B	1	40.8	9.7	1544.0	570.6	14634.9	5408.8	8.5E+14	3.1E+14
	2	91.2	21.5	14755.0	5379.4	67159.0	24485.1	1.4E+15	5.1E+14
	3	137.5	25.9	12617.8	3680.2	56511.9	16482.6	6.8E+15	2.0E+15
	4	0.6	0.2	171.2	109.5	-72.3	-46.2	-8.9E+14	-5.7E+14
	5	11.2	3.4	434.8	202.9	993.1	463.4	6.7E+14	3.1E+14
	6	101.8	35.1	14966.5	7982.1	85051.8	45361.0	9.0E+15	4.8E+15
	7	202.5	82.7	15176.6	9592.9	87472.4	55289.8	7.8E+15	4.9E+15
CS4C	1	69.7	16.0	4880.7	1738.7	32769.7	11674.2	2.3E+15	8.2E+14
	2	43.5	9.0	3077.9	988.8	8324.1	2674.1	1.9E+15	6.1E+14
	3	48.5	6.9	6588.8	1460.5	27779.7	6157.8	4.1E+15	9.1E+14
	4	-9.8	-4.0	-226.4	-144.6	-3235.5	-2066.7	-6.8E+14	-4.3E+14
	5	-1.3	-0.4	-0.6	-0.3	-40.7	-20.1	-5.0E+13	-2.5E+13
	6	12.3	4.2	3137.6	1673.4	3664.5	1954.4	2.3E+15	1.2E+15
	7	44.0	18.1	4478.6	2845.8	25314.9	16085.5	2.3E+15	1.5E+15

Note: Negative discharge indicates landward flow due to tidal effect during period of measurement.

over the weir. Diversion chambers within the study area can be classified according to three categories, as illustrated in Figure 11. The dependency of interceptor flow on stage above the weir is summarized for each category of diversion chamber: Category I, low; Category II, high; Category III, inversely proportional to weir length.

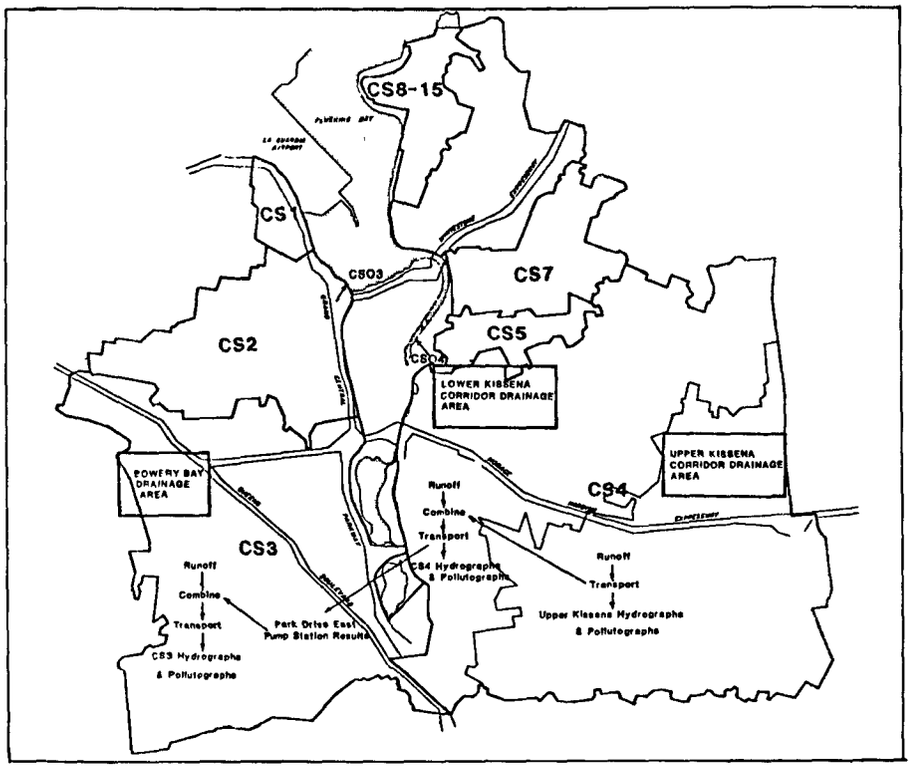


Figure 9. CSO drainage areas and SWMM order of module application: I upper Kissena Corridor, II lower Kissena Corridor, III Bowery Bay. (See Reference 5 for III Bowery Bay.)

Table 3. PERTINENT CHARACTERISTICS OF UPPER/LOWER KISSENA CORRIDOR

SWMM model characteristics	Upper Kissena Corridor	Lower Kissena Corridor
Drainage area (acres)	3700	3709
Area with separate sewers (acres)	1200	366
Percent impervious	43.2	50.3
Number of regulators	8	12
Number of catchment areas	13	15

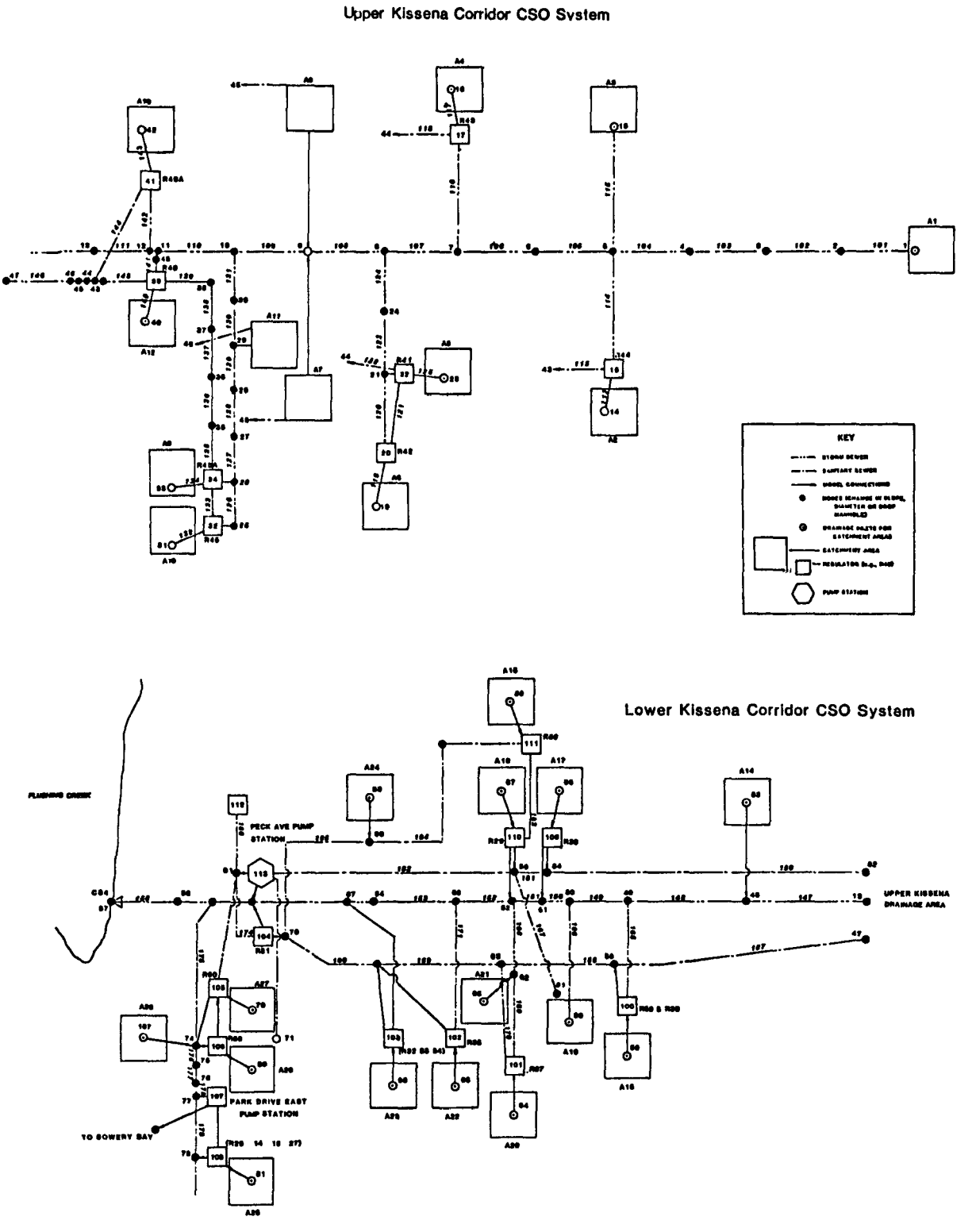


Figure 10. Schematics of SWMM model network, upper and lower Kissena Corridor CSO system.

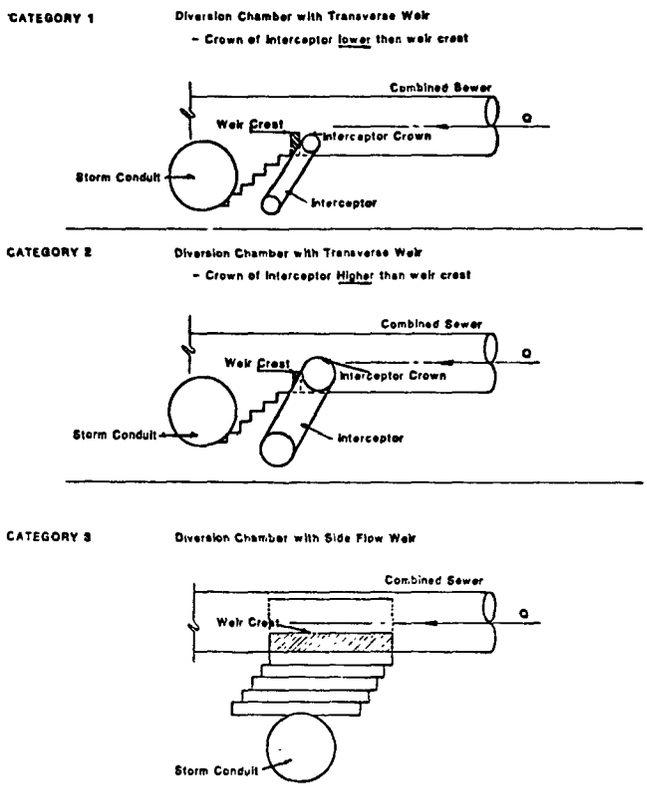


Figure 11. Simplified schematic of diversion chambers.

Regulators with a significant flow-stage dependency were defined as stage-dependent flow dividers (referred to as Type 20) in the Transport module, which was used to simulate discharge and pollutant concentration. Water quality constituents (BOD_5 and total coliform) were modeled by setting concentrations for sanitary wastewater and storm water based on previous New York City data (Table 4):

TABLE 4. POLLUTANT CONCENTRATIONS USED IN SWMM

	Sanitary wastewater	Storm water
BOD_5 (mg/l)	130	15
Coliform (counts/100 ml)	1.1×10^7	1.5×10^6

The model calibration of flow in the upper Kissena Corridor is illustrated in Figure 12. The total CSO volume computed by SWMM is approximately

10% greater than the field measurements. The observed peak flow precedes the model peak by nearly 1 hr. Hourly rainfall data from National Weather Service gages indicate that the storm tracked over the model study area prior to reaching the La Guardia rain gage, which was the basis for model input. The observed BOD_5 concentration of the CSO is compared to SWMM results with stormwater concentrations of 10 and 15 mg/l (Figure 13). Initially, the SWMM simulation with 15 mg/l fits the observed data; for the latter part of the survey the 10 mg/l simulation is closer. This suggests a diminishing pollutant concentration in storm water for high rainfall accumulations due to a finite pollutant source in the drainage area.

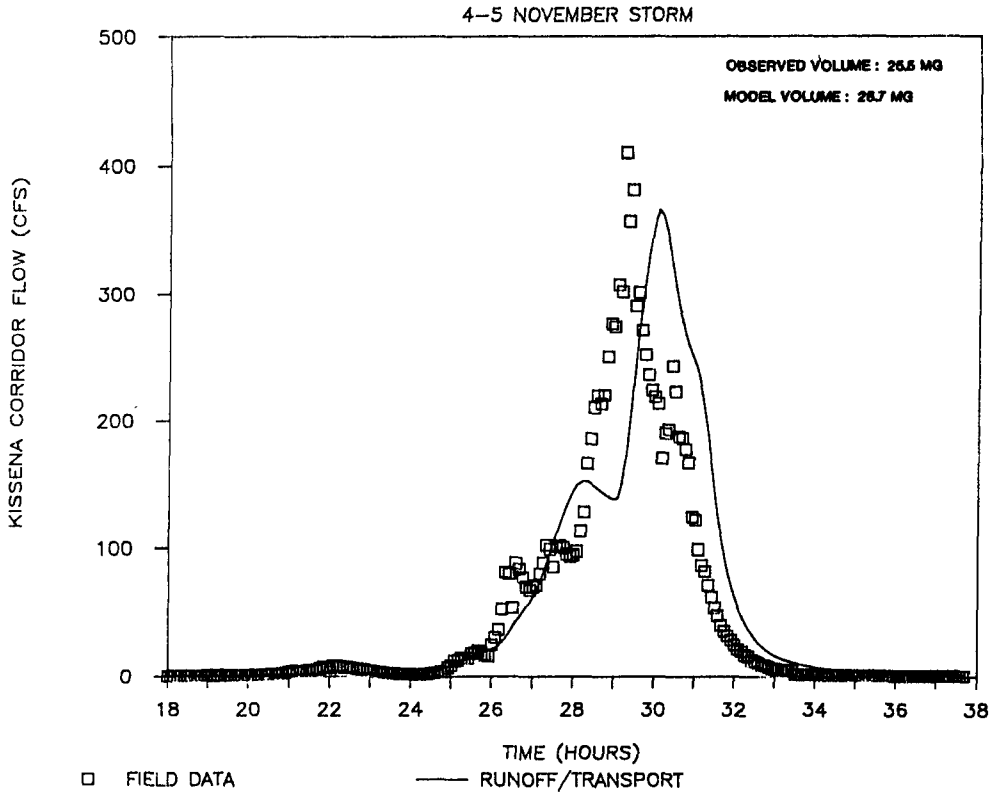


Figure 12. Upper Kissena Corridor flow calibration.

The SWMM model of the entire CS4 system was verified by simulating the CSO discharge and pollutant loads during three surveys with cumulative rainfall of 0.31, 1.35, and 2.67 in. The SWMM results are compared graphically with the observed net CSO discharge volume, BOD, and coliform loadings in Figure 14. It should be noted that a BOD_5 concentration of 10 mg/l in storm water was used in SWMM for the highest rainfall survey, based on the hypothesis of diminishing concentration with high rainfall accumulations. The agreement between the computed and observed results demonstrates the accuracy of SWMM in modeling the CS4 system over a wide range of rainfall conditions.

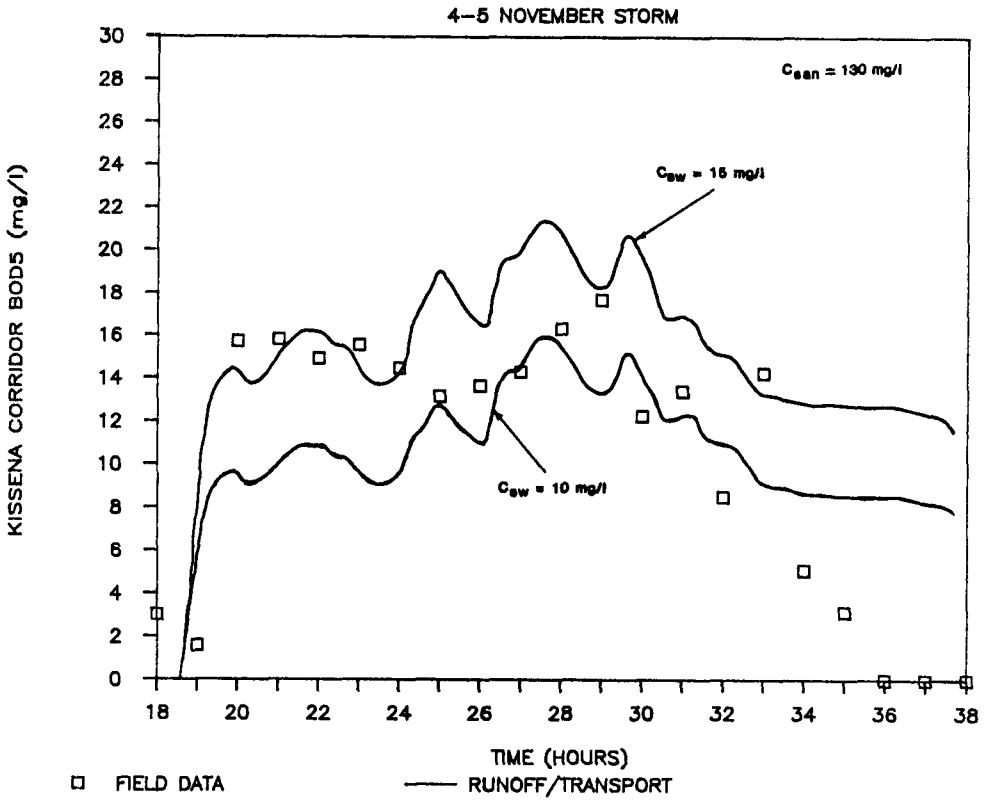


Figure 13. Upper Kissena Corridor BOD calibration.

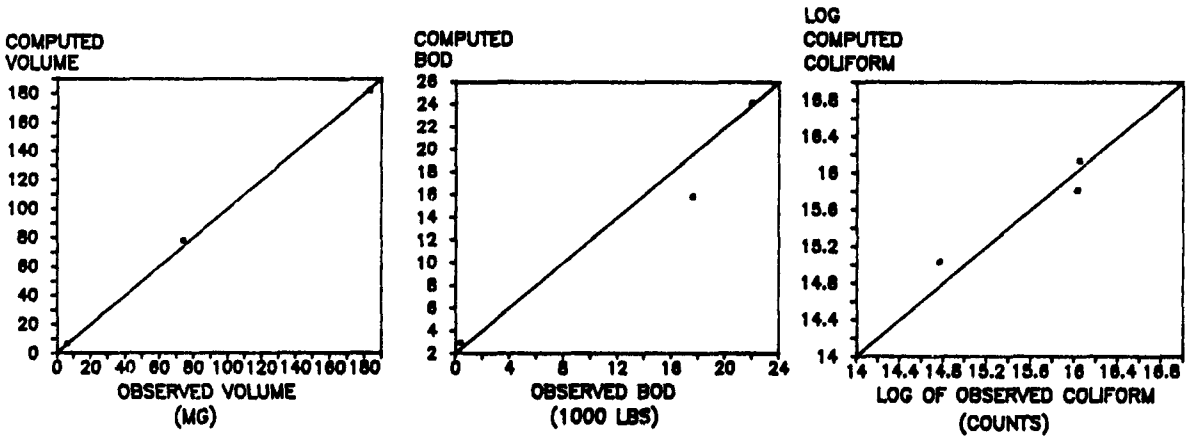


Figure 14. SWMM vs observed results for CSO surveys 5, 6, and 7.

SWMM was applied to evaluate two specific considerations for CSO abatement from CS4. First, the model showed a fairly uniform contribution of CSO discharge and pollutant loads along the entire length of the Kissena

Corridor system so that control of one or two specific regulators would not be sufficient. Thus, the most effective abatement strategy would be to locate a CSO storage facility near the outfall so that it would encompass bypasses from all regulators. Second, a hydraulic analysis of the Kissena Corridor storm sewer as an in-line storage facility was performed with Extran, by generating backwater profiles for two scenarios involving storage dams (simulated as transverse weirs). When the in-line storage capacity of approximately 11 to 17 MG is exceeded, overflows through the systems would cause CSO to back up over the weir at Regulator 30 (Figure 15). Potential backups in a sewer system already prone to flooding made this option unattractive. Furthermore, velocities of 1.0 fps would occur during typical overflows so that in-line storage would not be effective in settling solids.

STORAGE-PUMPING MODEL

SPM was developed to evaluate the effectiveness of the CSO retention facility in reducing the CSO discharge, BOD, and TSS loadings. The theoretical development of the SPM computer program entails flow and mass balance equations for the operational sequence of storage and pumping, as shown schematically in Figure 16. Hourly CSO inflow is routed to a storage volume and the settling of suspended solids and associated BOD is computed. After each rainfall ends, CSO is pumped from the storage facility to a treatment plant according to the available capacity based on the plant's diurnal inflow.

A relationship between percent removal of TSS by settling and overflow rate was developed using settling column results for CSO samples taken at the upper Kissena Corridor. The following equation relates the fraction of TSS settled to the overflow rate:

$$F_{sed} = 0.80e^{-0.00042(OR)} \qquad (7)$$

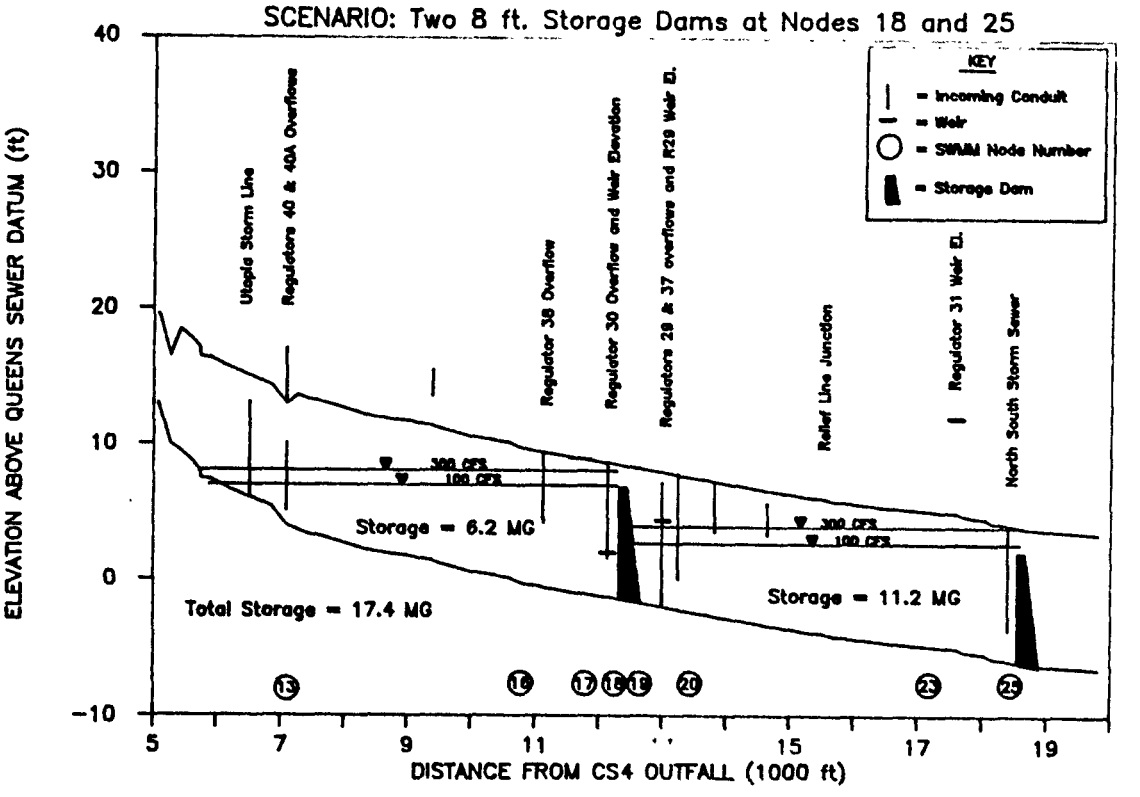
where

F_{sed} = fraction settled

OR = overflow rate (gpd/ft^2)

Incoming BOD is assumed to be 50% particulate and 50% dissolved based on the observed ratio of filtered to total BOD. A lower concentration limit of 15 mg/l is set for TSS. In the SPM, overflow rate (OR) is calculated as the flow passing through the storage facility divided by the surface area of the tank.

The SPM was run for a four-month (summer seasonal) period at storage capacities for CS4 ranging from 10 to 80 MG. The resulting percent reductions in BOD and TSS loads are presented for off-line storage capacities that include the effects of settling and also without settling, which is indicative of in-line storage (Figure 17). An off-line storage facility with a 40 MG capacity would reduce the volume, BOD, and TSS load of CSO discharged from CS4 by 58, 73, and 76% respectively. Hence, the 40 MG



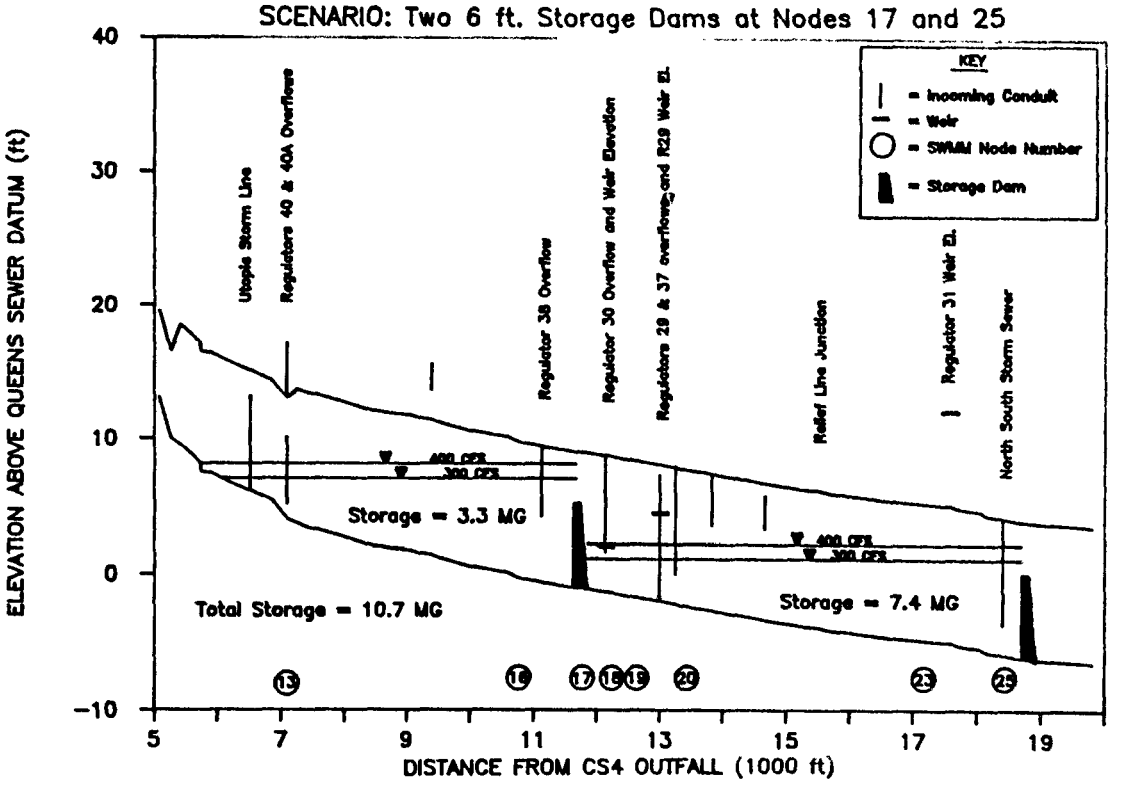


Figure 15. In-line storage backwater profiles for Kissena Corridor storm sewer.

capacity was found to maximize load reduction effectiveness as additional load reductions diminish for further increase in storage volume. Water quality models were applied to evaluate the response in dissolved oxygen and coliforms to various alternatives including an array of CSO storage capacities for the two largest discharges. Based on those results a 40 MG storage facility is recommended for CS4 to improve water quality particularly in Flushing Creek.

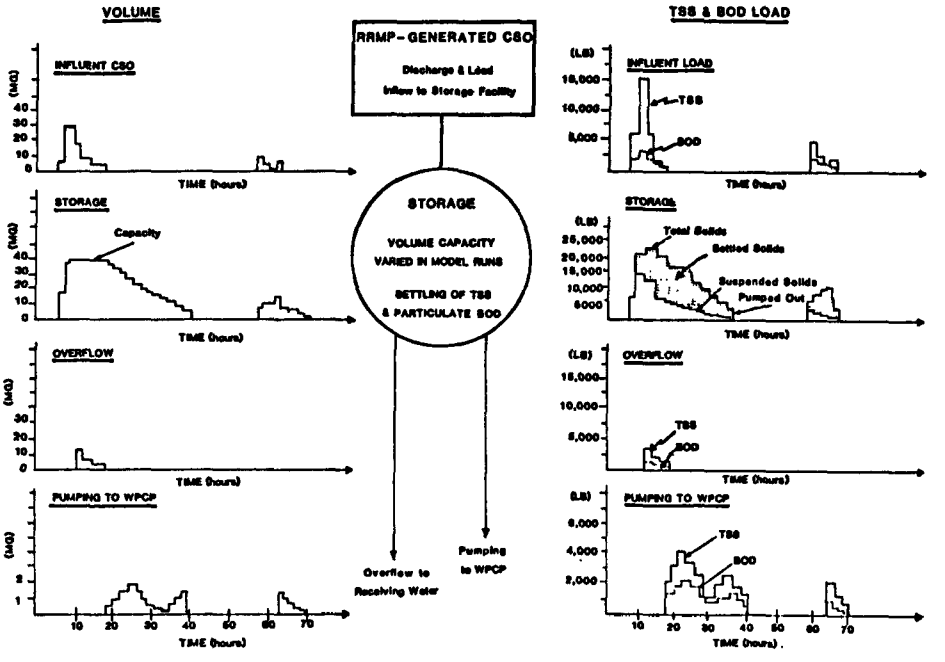


Figure 16. Schematic of CSO storage and pumping to WPCP for SPM.

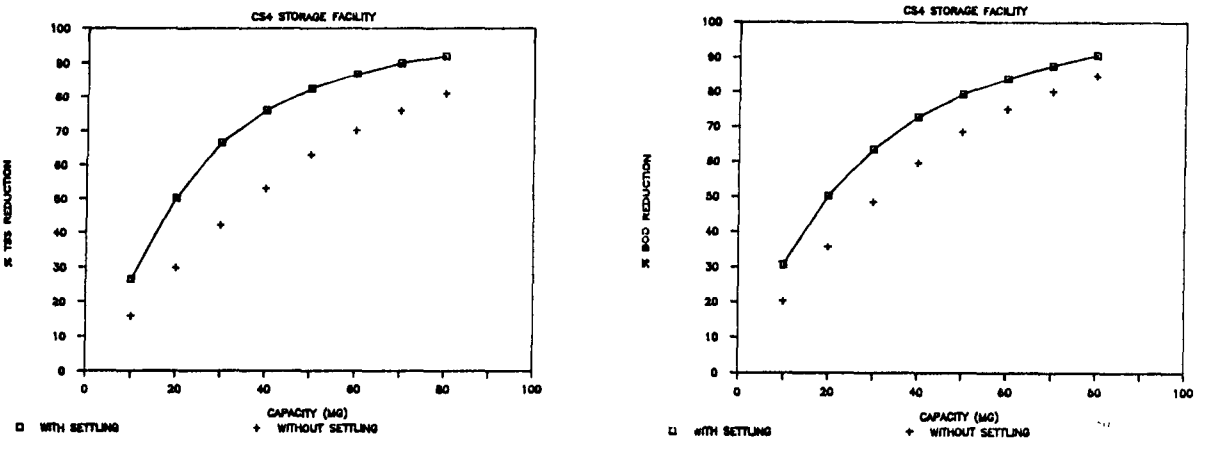


Figure 17. Load reduction vs storage capacity for CS4 storage facility.

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ACCOUNTING FOR TIDAL FLOODING IN DEVELOPING URBAN STORMWATER MANAGEMENT MASTER PLANS

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ABSTRACT

Typical receiving waters in urban storm drainage systems are rivers (conveyance systems) and lakes (storage). Depending on their size and hydrologic behavior, they can be treated as true boundary conditions, i.e., as conditions not affected by the storm-drainage system. Coastal receiving waters present a different challenge. First, there is the diurnal tidal fluctuation, which poses the problem of phasing with the runoff hydrograph. Second, there is the question of surge-induced coastal flooding and its coincidence with riverine flooding.

Flood insurance studies, which are concerned with the delineation of flood zones under existing conditions, use the joint probability method. Stormwater management studies, with the goal to improve inland flooding conditions, require a different approach. Such an approach was developed for the stormwater master plan for Virginia Beach, Virginia. It encompasses the following steps: delineation of tidal zones affected mostly by the astronomic tide; delineation of fluvial zones affected exclusively by inland runoff; and delineation of transition zones usually affected by runoff and base flow but occasionally affected by inland propagating surges. Stormwater management alternatives are thus designed to improve conditions primarily in the fluvial zone. They are also designed to improve conditions in the transition zone under runoff conditions, with the constraint that they must perform adequately during extreme surge events.

Procedures are developed for the delineation of the above three zones, including addressing the question of astronomic tide phase lag, and joint probability of occurrence of surge and runoff. A separate procedure is also developed for non-tidal, wind set-up prone embayments. Application of the methodology to the Virginia Beach stormwater master plan is presented as a case study.

INTRODUCTION

The effect of urbanization on the rainfall-runoff portion of the hydrologic cycle is manifested by an increase in peak runoff-rate and total runoff volume. This phenomenon is primarily due to the increase in connected impervious areas. Flooding may result from these changed conditions when the "receiving waters" (i.e., the area where the urbanized watershed ties into the larger scale natural drainage basin) do not have the conveyance to drain away the increased peak runoff rate nor the volume to store or temporarily accommodate the excess runoff volume. Stormwater management plans are developed to alleviate such conditions.

Coastal receiving waters present a different challenge: they are large masses of water that are at once more accommodating of large runoff volumes but which also exhibit their own flooding conditions that require interaction and coordination with the inland flooding conditions.

Receiving waters are often considered as boundary conditions in stormwater models such as SWMM. However, accounting for coastal receiving waters in such models clearly requires more than a simple incorporation of an "elevation-versus-time" relationship for these boundary conditions. The purpose of this paper is to present a general methodology to account for tidal flooding in stormwater studies and its implementation in the context of the SWMM model. This methodology was developed as part of the Virginia Beach, Virginia stormwater master plan but is generally applicable to most coastal communities.

PROBLEM STATEMENT

Coastal communities in the mid-Atlantic states, from New Jersey to Florida, and along the shores of the Gulf of Mexico, are built on the coastal plain which is characterized by a flat, almost relief-less topography. These communities are notorious for their flooding problems. For example, several communities in the Florida panhandle were flooded three consecutive times in the same season, the hurricane season of 1985. Yet, all flooding is not always caused by tidal waves (surges). Inland runoff in the absence of coastal surges also causes flooding, and the poor natural drainage of the flat lands ususally exacerbates the problem.

One way to deal with the coastal flooding problem is to avoid development in these areas. The government provides incentives to this end by offering affordable insurance to communities participating in the National Flood Insurance Program. To participate in this program, the communities have to adopt floodplain management measures to reduce future losses. Flood-risk zones are delineated to establish the insurance rates. The critical level of risk is the 1% risk of flooding per year. Equivalently, protection is sought against the 100-year event. Where many causes of flooding are present, a composite risk is estimated by means of the joint probability of occurrence. For example, if a certain flood level is exceeded on the average once every 100 years from riverine flooding (0.01 probability of exceedance) and once every 100 years from coastal flooding (0.01 probability of exceedance), and if these are independent, then that level will be exceeded on the average

twice in 100 years. That is, the average return period is 50 years. For events that are not independent, the procedure is amended by use of conditional probabilities.

The joint probability method (Myers, 1970) is adequate for describing existing conditions. Stormwater management plans, however, deal with measures to improve drainage conditions, not just to describe them as they exist. In stormwater management plans, therefore, it is necessary to discriminate between the various modes of flooding, to address each one separately, to encourage synergisms and to coordinate conflicts.

The questions that need to be addressed are:

(1) What are the areas that are predominantly affected by coastal conditions?

(2) What are the areas that are predominantly affected by inland flooding?

(3) What is the extent of the areas that are affected by both coastal and riverine flooding?

(4) Are there any measures that can improve <u>either</u> coastal or riverine flooding conditions?

(5) Are there any measures that can improve <u>both</u> coastal and riverine flooding conditions?

(6) Are there any measures that could exacerbate the situation under adverse conditions? E.g., a weir or flood wall to protect against coastal surges that would also impede inland runoff.

METHODOLOGY - APPROACH

The first step in addressing the above questions is the delineation of the three zones, respectively, of exclusive astronomic tidal influence, inland runoff influence, and transition zone of occasional coastal surge flooding. This is accomplished by performing the following tasks:

(1) Determine astronomic tide range, amplitude, and their variation through the lunar cycle at the closest tidal gage.

(2) Simulate a 25-year storm of duration comparable to the watershed time of concentration for various phases and amplitudes of the astronomic tide. From these runs, establish the uppermost extent of propagation of the astronomic tide. Note whether this section is sensitive to tidal phase and/or amplitude.

(3) Repeat the same above simulations with a fixed tidal boundary condition of high tide. Establish whether these simulations produce similar or identical envelopes of high water marks along the tidal reaches of the watershed. If they do, then the analysis can be simplified by neglecting the tidal phasing parameter.

(4) Analyze all hurricanes and winter storm surges of record at the nearest tidal gage. In particular, establish correlation between surges and associated rainfall.

(5) Simulate several hurricanes and/or winter storms with their attendant rainfall. Observe maximum extent of propagation of surge. Establish sensitivity of that location to surge amplitude.

(6) Repeat above simulations with constant downstream boundary condition at peak surge level. Establish whether these simulations produce similar or identical envelopes of high water marks along the tidal reaches of the watershed. If they do, then the analysis can be simplified by not having to account for the time variation of the surge.

The above determined river cross-sections delineate three zones as illustrated in Figure 1; namely, the zone of exclusive coastal flooding, the zone of riverine flooding, and the buffer zone which is occasionally affected by surges. Design storms or other conventional methods can be used for stormwater management in the riverine zone, with the additional constraint that these practices should perform adequately during surge events as well. This would actually be monitored in the transition zone.

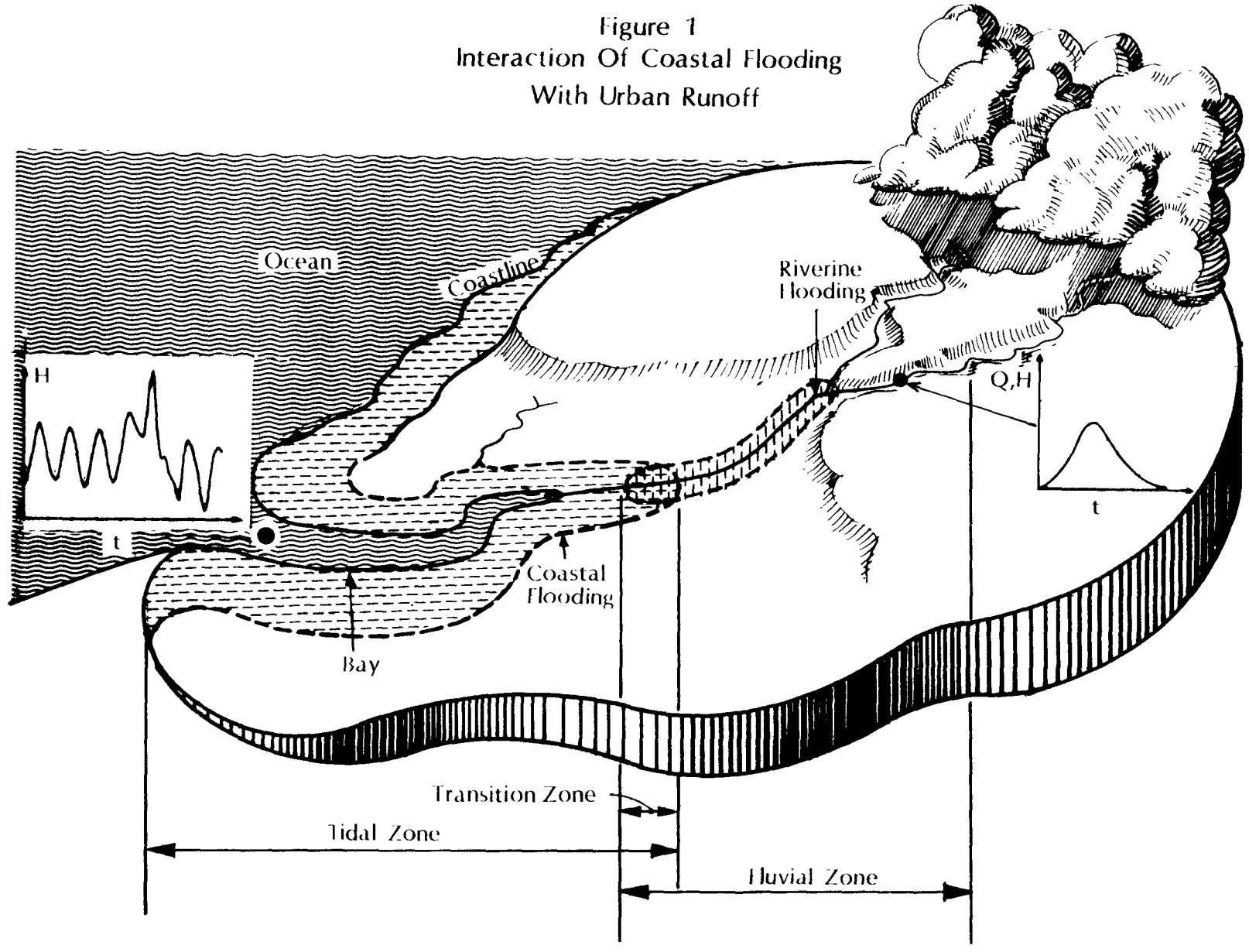
Where representative tidal gages are not available, the above procedure should be augmented to include simulation of representative surges. A special case is that of landlocked embayments subject primarily to local wind set-up. Furthermore, when multiple tidal boundaries exist, the above procedure should be expanded to incorporate analysis of the interaction of the various embayments. The above procedure was implemented using the model SWMM.

APPLICATION - IMPLEMENTATION

Implementation of the above methodology is shown for the Virginia Beach, Virginia, stormwater master plan. The City of Virginia Beach is located along the Atlantic coast at the mouth of the Chesapeake Bay, in the south-eastern corner of Virginia (see Figure 2). This 250-square mile city is one of the most rapidly growing areas of the country. It is drained primarily by a complex system of interconnected channels, canals, and lakes, and has few large storm sewer systems.

The City was divided into 25 major watersheds for master planning analysis. The majority of watersheds in Virginia Beach drain into coastal receiving waters; these receiving waters vary significantly in tidal influence. For example, as shown in Figure 3, the watersheds in the northern section of the City are bounded by a large bay system which drains into the Chesapeake Bay; thus, these watersheds are directly influenced by the tides and the storm surges of the Atlantic Ocean. Several watersheds in the eastern section of the City are bounded by the Elizabeth River estuary, which is also influenced by the astronomic tide fluctuations of the Chesapeake Bay. In the southern section of the City, several of the eastern watersheds are influenced by wind-driven tides in the landlocked Back Bay. The watersheds in the southwestern section of the City drain to the North Landing River, which ties into the Atlantic Ocean.

Figure 1
Interaction Of Coastal Flooding
With Urban Runoff



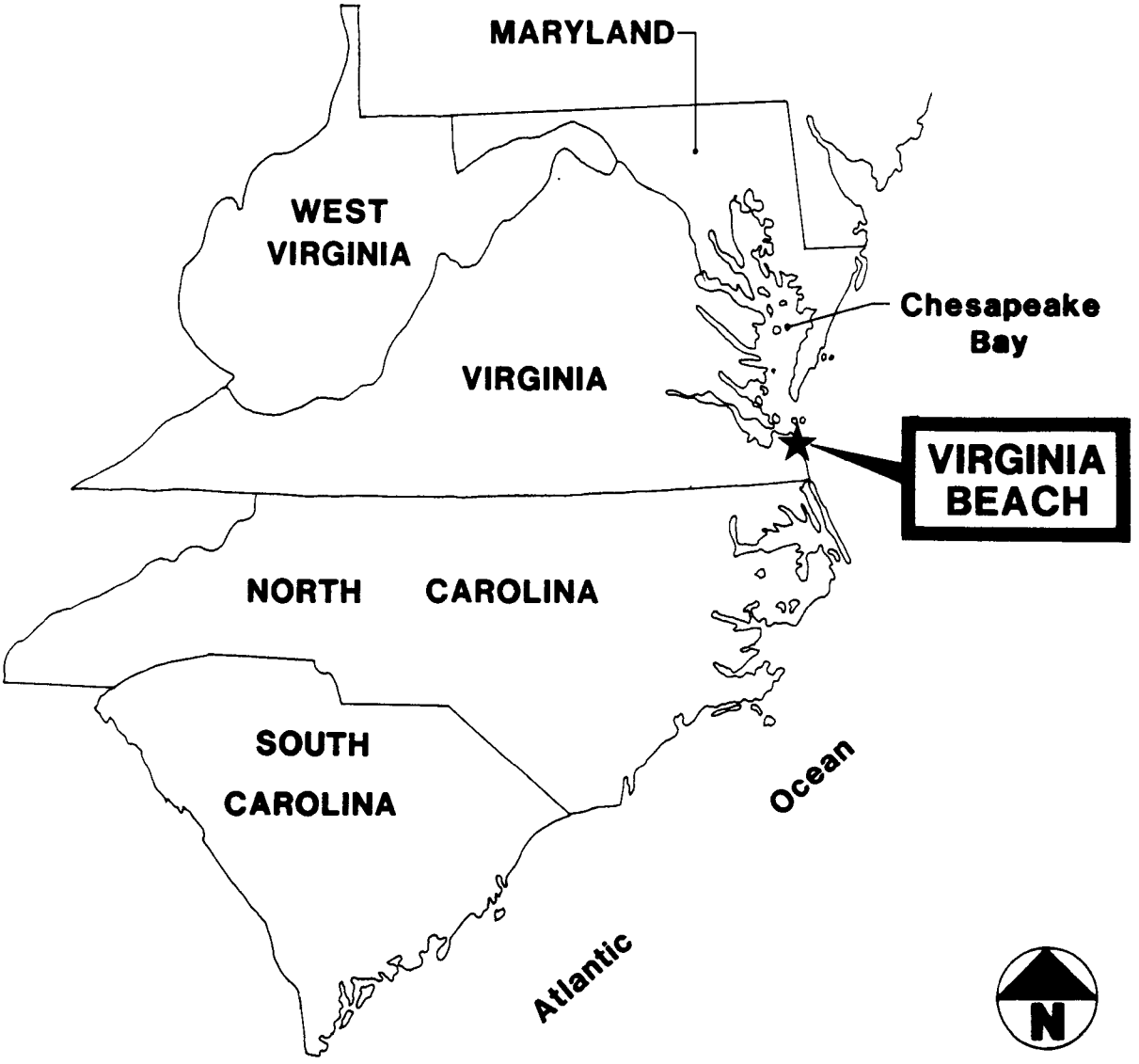


Figure 2. Virginia Beach Location Map.

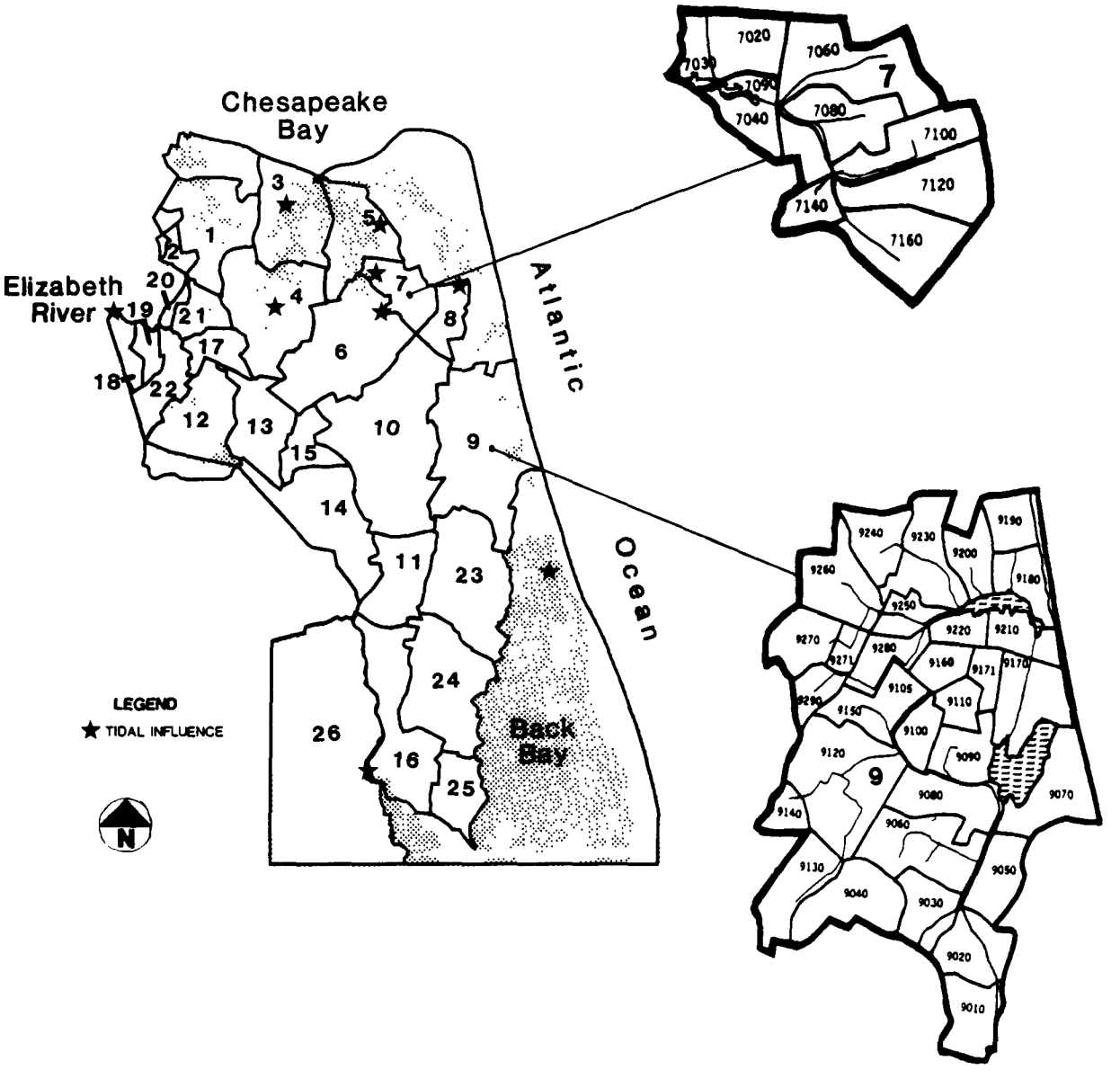


Figure 3. Location of Tidal Boundaries for Virginia Beach.

The variety of tidal boundary conditions in the Virginia Beach area present an interesting application of SWMM to study city-wide interactions. Because the primary drainage system for a large portion of Virginia Beach is interconnected, it is important to understand the operation and sensitivity of the entire interconnected drainage system under a variety of scenarios in order to provide good master plans.

An analysis of tidal conditions and typical surge events with corresponding rainfall was performed as the first step for delination of tidal influence zones. The highest tides at four long-term control tidal stations near Virginia Beach are shown in Table 1; summary information on Norfolk, Virginia (adjacent to Virginia Beach) rainfall occurring at the time of the highest tides is shown in Table 2. Comparison of these tables shows that all high tide events were accompanied by rainfall. Table 3 lists the temporal distributions of each of the rainfall events given in Table 2 for which hourly data are available. As can be seen from the tables, no consistent rainfall pattern can be generalized because of the random nature of each event. For example, some storms are fully advanced while others are delayed; in addition, most events are multi-peaked. From Table 2, the average duration of rainfall events associated with the highest tides near Virginia Beach is 16.5 hours and the mean total rainfall is 2.3 inches. Thus, the average rainfall intensity is approximately 0.14 inches per hour; this intensity relates to a 16-hour storm having a return period of just under 2 years when compared to the Virginia Department of Highways and Transportation (VDH&T) published intensity-duration-frequency curves for Norfolk, Virginia. Based on this analysis, the joint probability method would be used with conditional probabilities to account for a surge/rain coincident event. For example, the conditional probability of occurrence of a 50-year return period tide (0.02 probability) with a 50-year return period rainfall event that would actually be even smaller than $0.02 \times 0.02 = 4.10^{-4}$; that is, extremely small. Rather, surges should be combined with rainfall amounts that usually occur during such storms. Using the joint probability method for stormwater planning analysis could lead to over-design of stormwater control facilities.

An example of the tidal reach delineation methodology previously presented is shown for Watershed 7 (see Figure 3), which drains into the Chesapeake Bay. Watershed 7 has been almost completely developed into single family residential land use. The most upstream subwatersheds drain into a small lake controlled by a weir; just downstream of this lake is a series of box culverts. A schematic of the model setup is shown in Figure 4.

To delineate the (channel zone) area of Watershed 7 subject to tidal fluctuations, a typical astronomic tide was applied as the downstream boundary condition for SWMM analysis. For one scenario, this tide was determined from the Norfolk, Virginia tide gage for September 27, 1956 (see Figure 5). A second scenario applied a constant 1.83 foot-tide (i.e., high tide for 9/27/56) as the downstream boundary condition. The timing of the astronomic tide was set such that the peak runoff coincided with high tide in the first scenario and with low tide in another. In all scenarios, the initial water level throughout the drainage system was set at 1.83 feet. A 25-year, 24-hour Soil Conservation Service (SCS) type II design storm was applied to the watershed in the RUNOFF block of SWMM. Figures 6 and 7 show that the high water mark at most locations was virtually the same for the

TABLE 1. HIGHEST SURGES AROUND VIRGINIA BEACH, VIRGINIA
(ELEVATIONS IN FEET NGVD)

STORM NAME	DATE	NORFOLK TIME	NORFOLK ELEV	HAMPTON ROADS TIME	HAMPTON ROADS ELEV	CHESAPEAKE BAY‡‡‡‡ TIME	CHESAPEAKE BAY‡‡‡‡ ELEV	VA BEACH TIME	VA BEACH ELEV
ST #4	09/19/28	N/A ‡‡‡	N/A	00:00	4.52	N/A	N/A	N/A	N/A
ST #8	08/23/33	N/A	N/A	09:00	7.22	N/A	N/A	N/A	N/A
ST #13	09/16/33	N/A	N/A	18:00	5.32	N/A	N/A	N/A	N/A
ST #13	09/18/36	10:18	7.03	09:48	5.92	N/A	N/A	N/A	N/A
	10/05/48	11:42	4.93	12:00	4.62	N/A	N/A	N/A	N/A
	04/11/56	22:06	6.03	22:00	5.52	N/A	N/A	N/A	N/A
FLOSSY	09/27/56	02:12	5.43	02:00	5.12	N/A	N/A	N/A	N/A
	10/06/57	08:30	5.23	08:00	4.82	N/A	N/A	N/A	N/A
	10/21/58	17:36	4.73	17:18	4.52	N/A	N/A	N/A	N/A
DONNA	09/12/60	06:24	5.33	06:12	5.12	N/A	N/A	N/A	N/A
	10/21/61	19:54	4.63	19:00	4.42	N/A	N/A	18:00 ‡	4.12 ‡
	03/07/62	10:24	6.83	10:00	6.42	N/A	N/A	13:06	4.22
DORA	09/13/64	06:06	4.93	15:48	4.82	N/A	N/A	N/A	N/A
	10/14/77	10:12	4.86	09:42	4.62	08:54	5.87	N/A	N/A
	04/27/78	00:30	5.87	00:00	5.61	23:00 ‡‡	6.24	N/A	N/A
	10/25/82	04:36	5.38	04:00	5.10	03:18	5.81	N/A	N/A
GLORIA	09/27/85	04:48	5.03	04:54	4.25	05:00	6.10	N/A	N/A
CHARLEY	08/17/86	N/A	N/A	22:00	4.52	02:24	5.37	N/A	N/A

‡ High tide of 4.6 feet NGVD recorded 06:00 10/22/61
‡‡ High tide occurred on 4/26/78
‡‡‡ N/A indicates data not available
‡‡‡‡ Elevations in feet-MLW (gage not tied into NGVD)

TABLE 2. RAINFALL ACCOMPANYING HIGHEST SURGES
VIRGINIA BEACH, VIRGINIA

STORM NAME	START		END		STORM DURATION (HRS)	TOTAL RAIN (IN.)
	DATE	TIME	DATE	TIME		
ST #4	09/18/28	N/A	09/19/28	N/A	N/A	3.57
ST #8	08/22/33	N/A	08/20/33	N/A	N/A	1.31
ST #13	09/15/33	N/A	09/16/33	N/A	N/A	1.59
ST #13	09/17/36	N/A	09/18/36	N/A	N/A	4.06
	10/04/48	22:00	10/05/48	12:00	15	1.90
FLOSSY	04/11/56	04:00	04/12/56	06:00	27	1.85
	09/26/56	14:00	09/27/56	08:00	19	2.57
	10/05/57	21:00	10/06/57	10:00	14	2.10
	10/21/58	13:00	10/21/58	00:00	12	2.25
DONNA	09/11/60	23:00	09/12/60	08:00	10	3.81
	10/21/61	18:00	10/22/61	09:00	16	0.53
	03/06/62	18:00	03/07/62	07:00	14	0.79
	09/13/64	01:00	09/13/64	23:00	23	4.73
	10/14/77	07:00	10/14/77	22:00	15	1.10
	04/27/78	06:00	04/27/78	18:00	13	0.24
	10/24/82	18:00	10/25/82	10:00	17	2.28
GLORIA	09/27/85	13:00	09/28/85	06:00	18	5.65
CHARLEY	08/17/86	10:00	08/18/86	04:00	18	1.08
				MEAN	16.5	2.30

NOTE: ONLY DAILY RECORDS AVAILABLE PRE-1948

TABLE 3. HISTORICAL STORMS ACCOMPANYING HIGHEST SURGES
(RAINFALL IN HUNDREDTHS OF INCHES)

	STORM DATE													
TIME	10/48	04/56	09/56	10/57	10/58	09/60	10/61	03/62	09/64	10/77	04/78	10/82	09/85	08/86
1	1	5	1	4	1	9	1	4	21	2	2	5	34	0.1
2	1	8	3	3	17	13	20	4	12	14	3	3	16	0.1
3	3	6	1	1	24	25	6	8	29	45	4	0	33	1
4	5	8	1	7	21	87	7	8	17	9	1	10	11	15
5	8	4	12	4	5	111	5	8	28	0	0	21	72	14
6	8	6	25	32	5	55	0	8	7	0	1	9	23	7
7	8	5	9	27	37	43	1	8	14	6	1	12	5	10
8	18	7	26	16	49	13	1	11	17	1	4	12	7	13
9	19	5	27	13	13	18	1	10	35	1	4	14	14	0
10	19	6	32	17	10	7	0	3	34	0	2	14	26	4
11	12	2	17	7	41		0	2	12	4	1	24	33	6
12	31	3	17	22	2		4	2	49	8	0	43	5	2
13	33	11	24	43			3	2	52	11	1	9	36	1
14	19	3	25	14			2	1	44	5		16	60	7
15	5	0	18				1		27	4		11	95	10
16		22	1				1		19			14	78	10
17		21	12						13			11	16	7
18		14	5						14				1	1
19		11	1						15					0.1
20		3							6					
21		8							2					
22		13							3					
23		5							3					
24		5												
25		2												
26		1												
27		1												
28		0												
50		0												
60		0												
70		0												
80		0												
90		0												
110		0												
120		0												
TOTAL	190	185	257	210	225	381	53	79	473	110	24	228	565	108

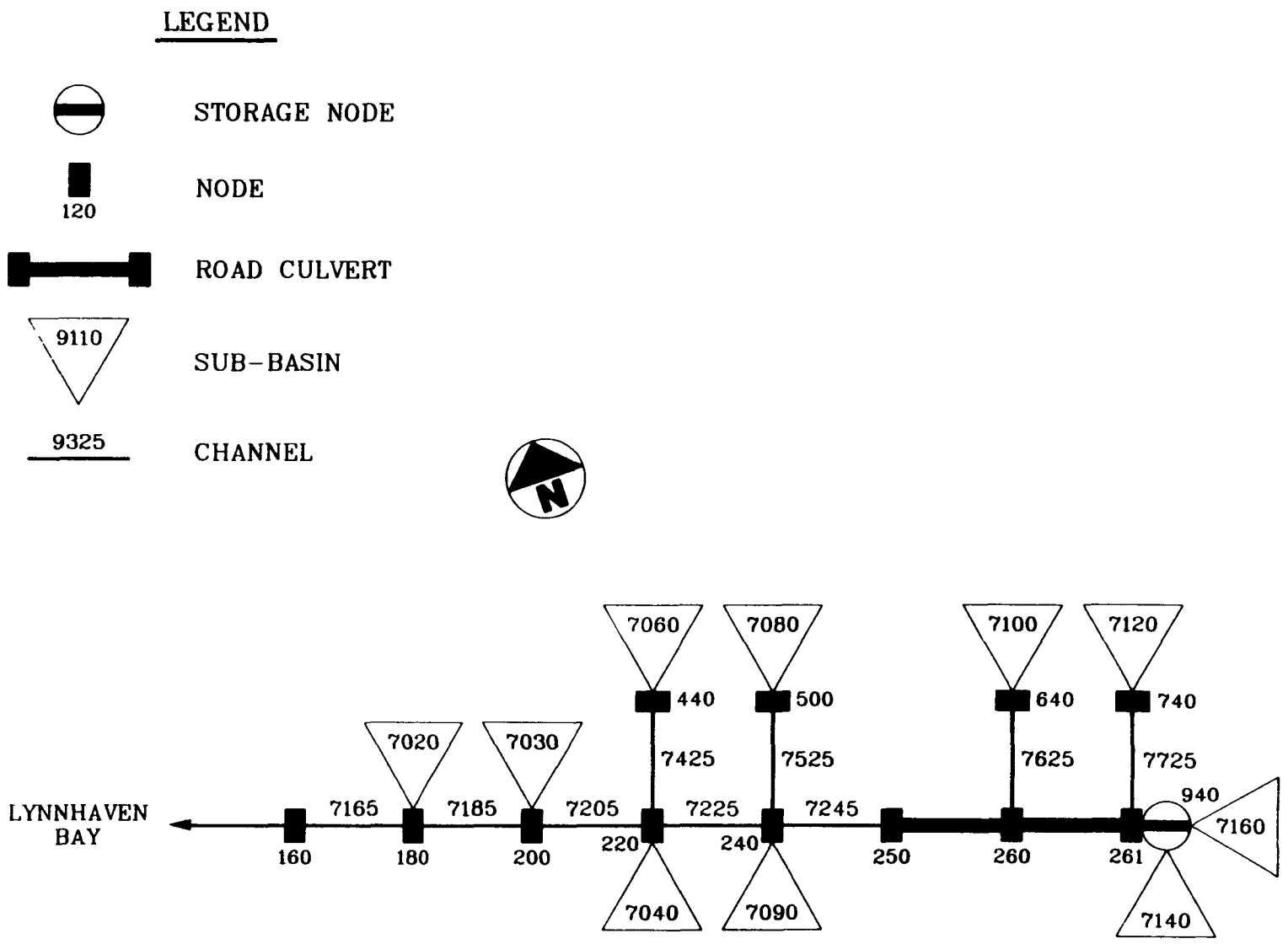
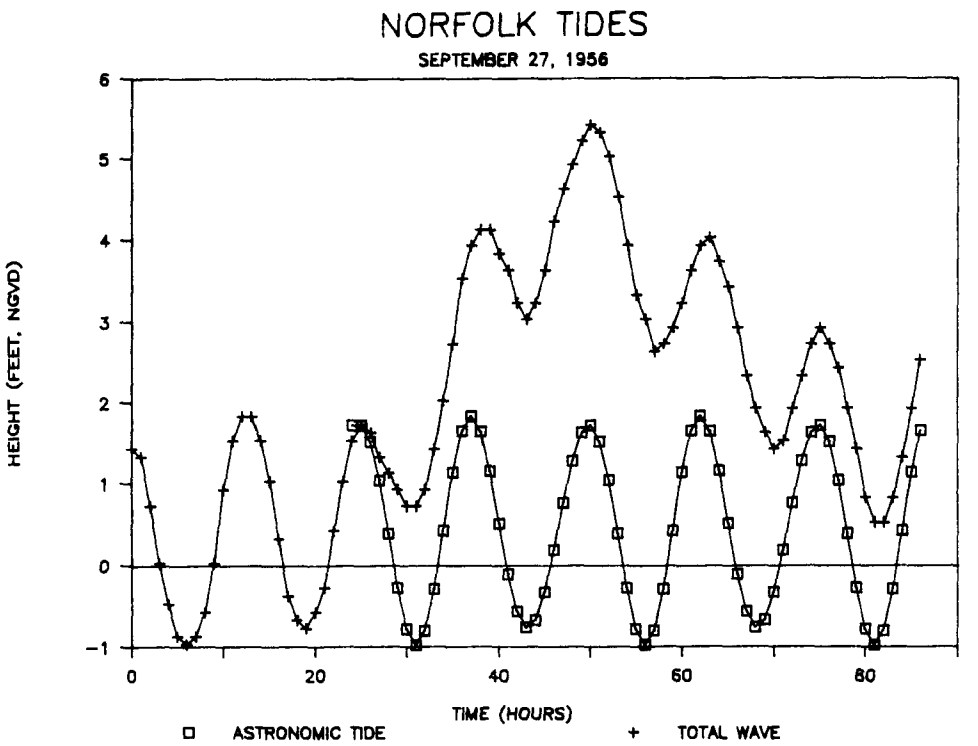


Figure 4. Model Representation for Watershed 7.



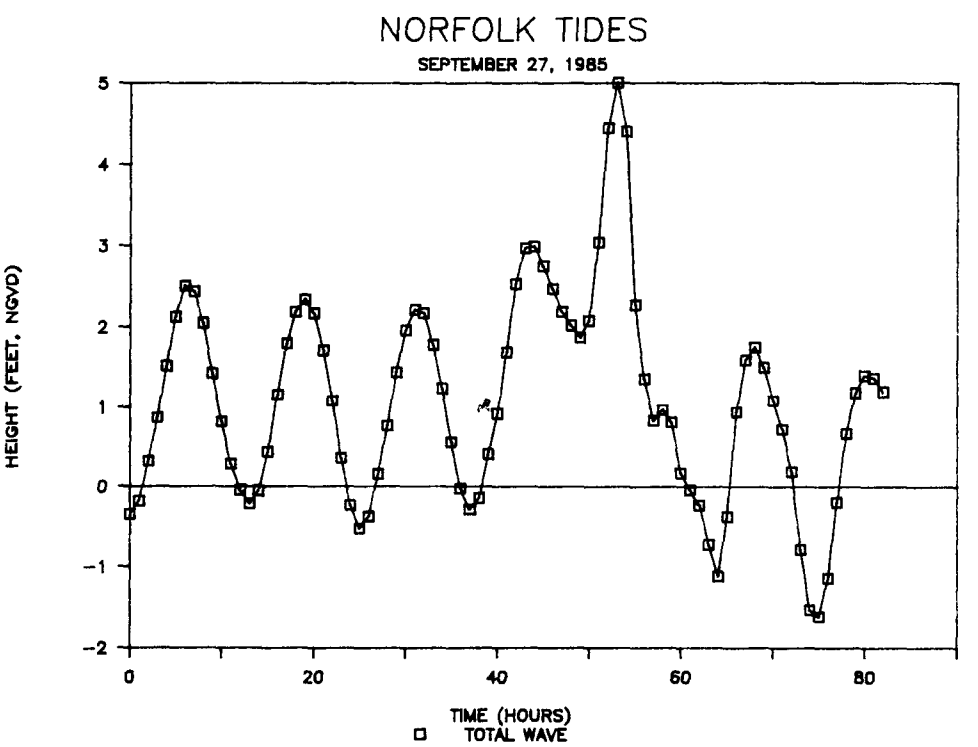


Figure 5. Tides Used In Case Study.

Figure 6
WATERSHED 7 - MAXIMUM WATER SURFACE ELEVATIONS
25-Year,24-Hour SCS Design Storm

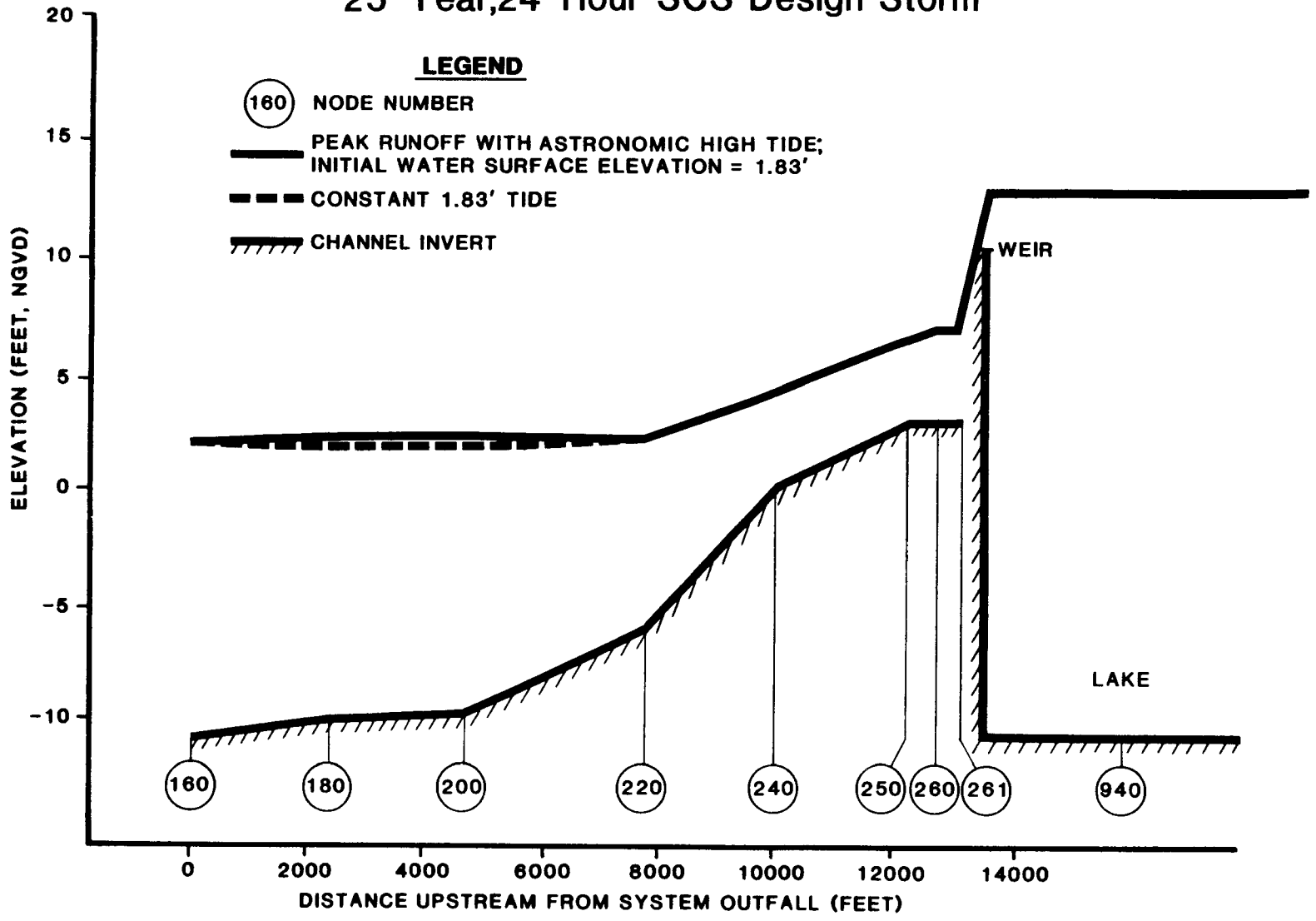
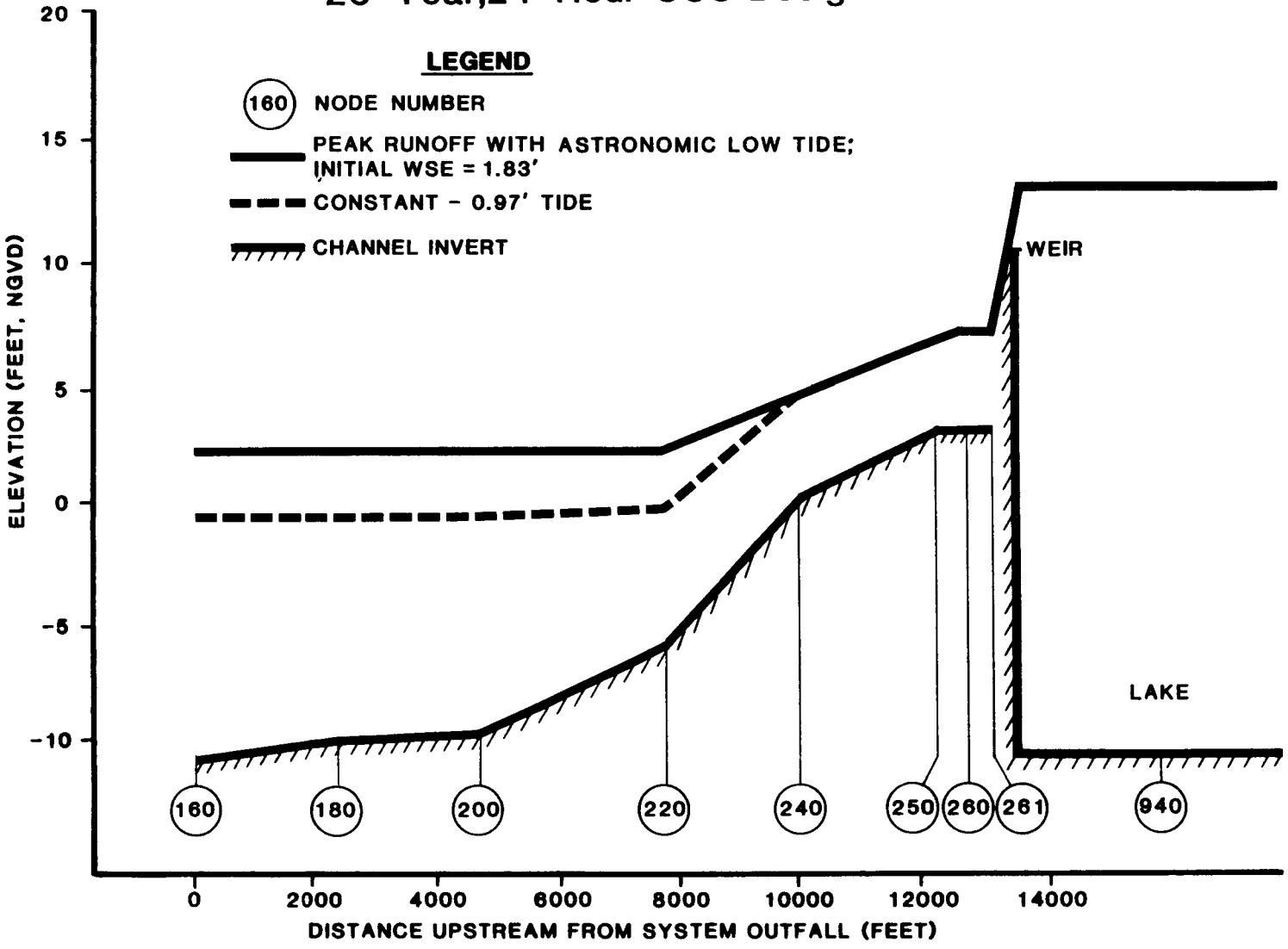


Figure 7

WATERSHED 7 - MAXIMUM WATER SURFACE ELEVATIONS

25-Year,24-Hour SCS Design Storm



astronomic tide and the constant tide scenarios. In addition, a plot of the high water surface elevations over time for Junction 220 (see Figure 8) shows the high water mark to be the same for a constant tide and for a time-varying tide. That is, for purposes of determining the envelope of high water marks along the tidal reaches of the watershed, it is sufficient to set the downstream boundary condition at high tide.

The transition zone influenced by occasional storm surges was delineated by applying the September 27, 1985 storm (Figure 5) as the downstream boundary condition for SWMM analysis. This storm, also known as Hurricane Gloria, was chosen because it produced a high storm surge and was also accompanied by 5.65 inches of rainfall, making it one of the most severe storms to hit the Virginia Beach area. Both the RUNOFF and EXTRAN blocks of SWMM were used with actual rainfall and tide data; the peak rainfall occurs almost simultaneously with the storm surge for this hurricane. A second scenario applied a 2.34 foot constant boundary condition to the system; this elevation was chosen because it was shown to be the typical high astronomic tide for several days preceding the storm. Figure 9 illustrates that Junction 250 (i.e., the downstream end of the box culvert system) was the limit of the zone affected by the storm surge. In addition, a plot of the water surface elevations over time for Junction 250 (see Figure 10) shows the high water mark to be the same for both a constant tide and for a time-varying tide.

Thus, it can be concluded that Watershed 7 is relatively insensitive to the timing of the tide and a constant tidal boundary may be used for analysis of stormwater management alternatives. This represents a significant simplification. However, for larger systems such as the interconnected citywide drainage system in Virginia Beach, such a simplification cannot be made.

Analysis of other watersheds subject to open water boundary conditions such as the Chesapeake Bay and its estuaries was completed following this methodology. The watersheds which are bounded by the landlocked Back Bay in the southeast corner of the City deviate from this methodology, however, because the water levels in Back Bay are influenced primarily by wind set-up. In the absence of astronomic tides and/or surges, that boundary condition is established by estimating the local wind set-up. Factors affecting wind set-up are: wind speed, fetch length, water depth, and duration. For a large, deep body of water, set-up increases in proportion to the square of wind speed (i.e., wind energy), and to the fetch length. Set-up also increases with duration up to the steady-state, fully arisen state. The time to steady-state set-up is longer for larger fetches. For closed bodies of water, the wind set-up is limited by the size of the basins (limited fetch), and by depth. The maximum sustainable set-up is reached relatively rapidly.

Careful examination of the Back Bay system reveals that there are principally two bodies of water: namely, North Bay/Shipps Bay in the north; and Redhead Bay/Back Bay in the south. These water bodies are separated by Long Island and linked through stable and deep channels at Great Narrows (Figure 11). The northern system has a north-south fetch length of about 6 miles and an average depth of 3.5 feet. The southern system has an approximate fetch of 12 miles and on average depth of 4.4 feet. For

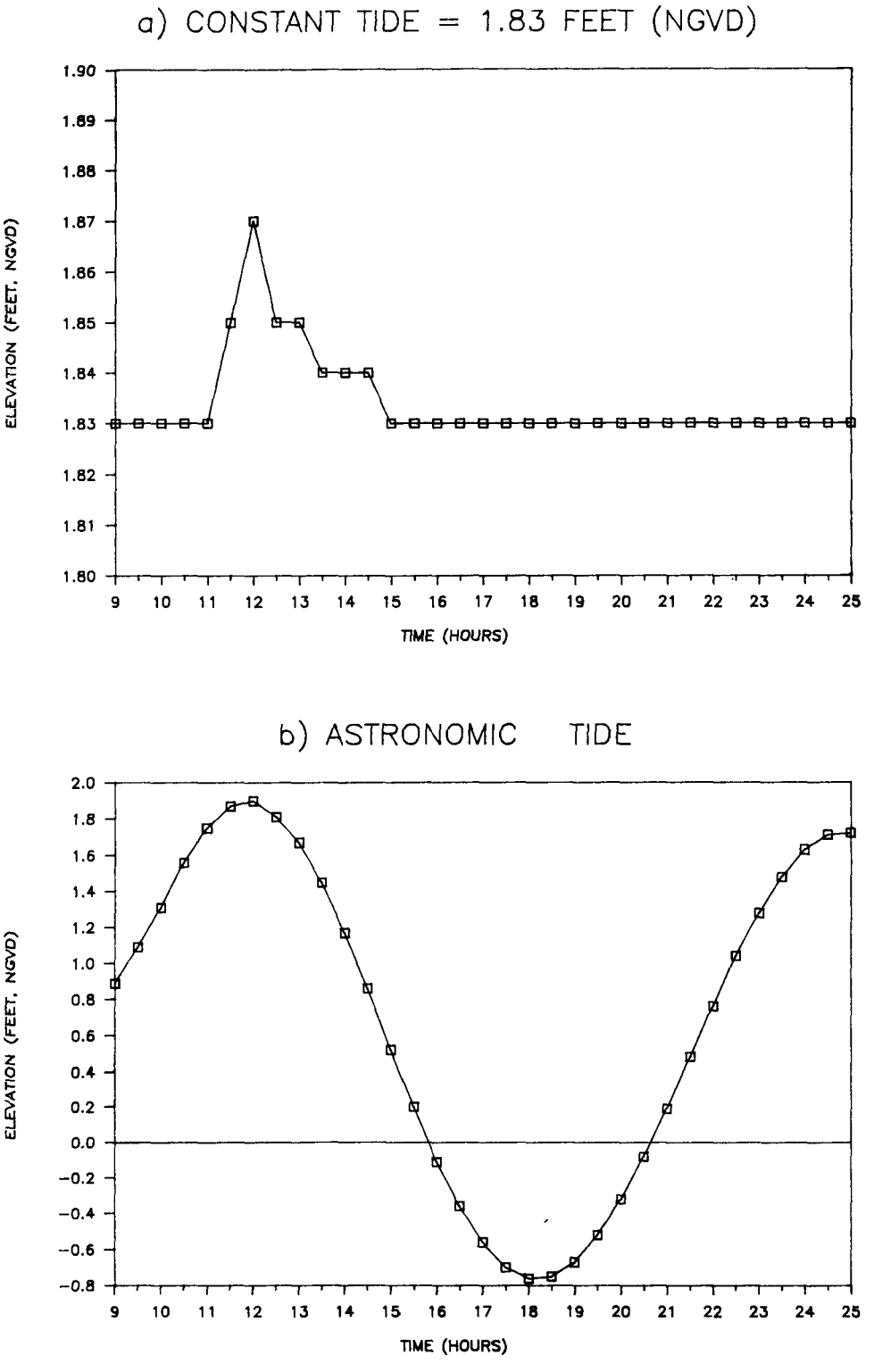
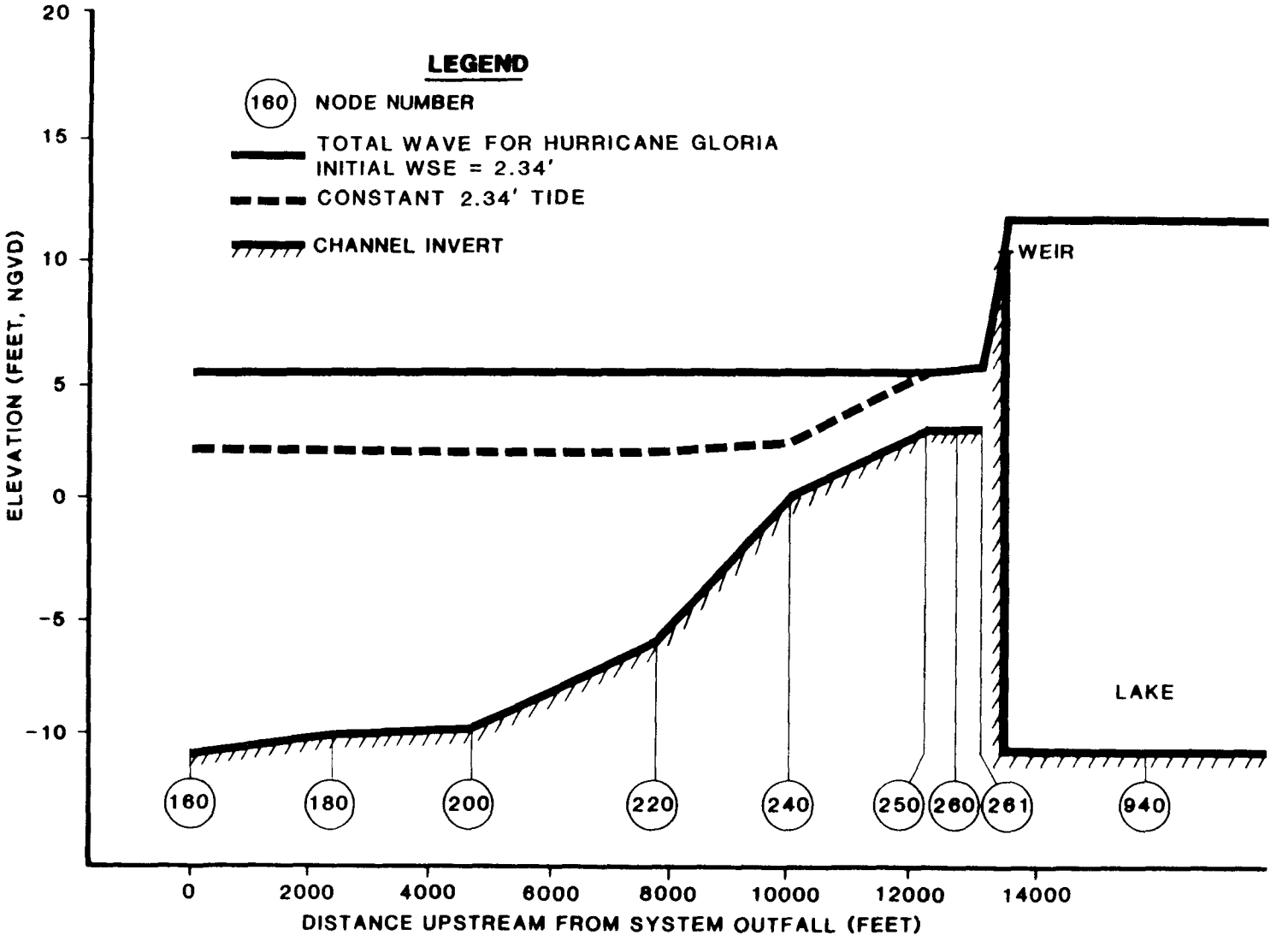


Figure 8. Effect of Constant vs. Time-Varying Astronomic Tidal Boundary Condition at Node 220 in Watershed 7.

Figure 9
WATERSHED 7 - MAXIMUM WATER SURFACE ELEVATIONS



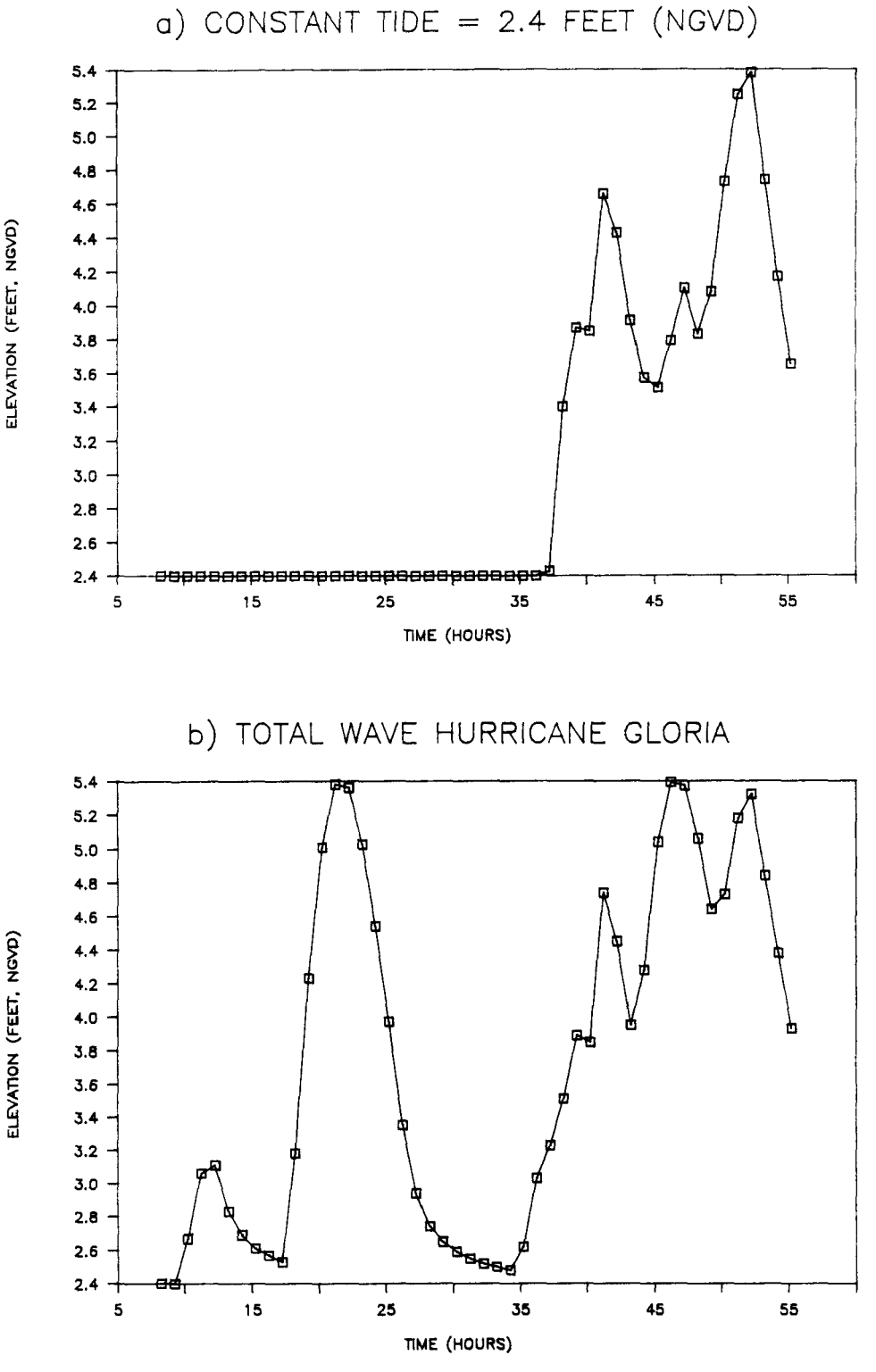


Figure 10. Effect of Constant vs. Time-Varying Storm Surge Boundary Condition at Node 250 in Watershed 7.

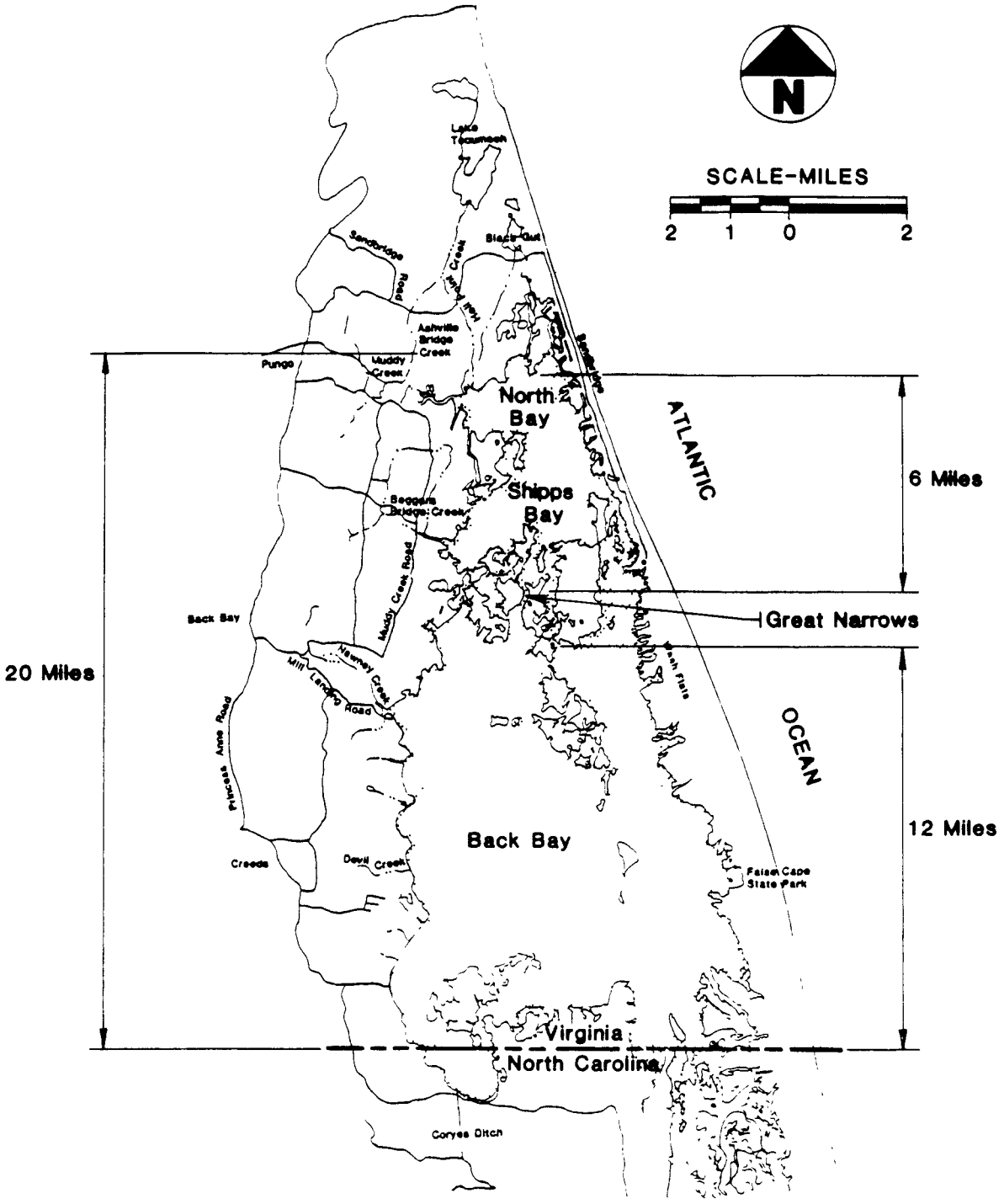


Figure 11. Major Wind Set-Up Zones of Back Bay, Virginia.
(City of Virginia Beach, 1984)

comparative purposes, Table 4 shows estimates of the wind set-up for each subsystem separately.

TABLE 4. SIGNIFICANT WAVE SET-UP HEIGHT
(Bretschneider, 1966)

Body of Water	Fetch Miles	Depth, ft	Wind	Significant height, ft 20	30	40	60	80 m/hr
North/Shipps Bay	6	3.5		<1	1.2	1.5	1.9	2.2
Redhead/Back Bay	12	4.4		1.2	1.6	1.8	2.2	2.6

Wave set-up in both above subsystems is depth-limited rather than fetch-limited (i.e., basins with the same size but deeper would sustain a higher set-up). The most important finding is that the deeper Back Bay can sustain a higher set-up such that a 0.5-ft gradient can exist between North Bay and Back Bay. Sustained flows through Great Narrows can therefore cause higher elevations in North Bay if sustained winds prevail over many hours. Thus, the following combination of boundary and initial conditions was used: a flat 2-ft level representing a frequent (at least once a year) highwater level in North Bay; and a 3-ft level representing more extreme conditions. Because the wind set-up is relatively insensitive to wind speed, and because storms can last long enough (2-3 days) to fill the entire system, tidal boundary conditions should be set at 2 ft msl for North Bay/Shipps Bay and 3 ft msl for Redhead Bay/Back Bay.

Watershed 9 (Figure 3) is an example of a watershed with Back Bay as the downstream boundary condition. Back Bay was modeled as a node with a constant water surface elevation of 2 ft msl; this elevation is likely to occur about once a year and was thus selected for the master plan analysis for this basin.

CONCLUSIONS

Stormwater management for coastal communities requires consideration of the interaction of inland and coastal causes of flooding. Dealing with the probability of their joint occurrence is only one part of the problem, as is the incorporation of "level-as-a-function-of-time" boundary conditions in SWMM. Rather, the questions to be addressed are:

(1) delineation of zones exclusively influenced by inland runoff;

(2) delineation of zones influenced by the astronomic tide;

(3) determination of zones occasionally influenced by surges; and

(4) development of measures to alleviate inland flooding which do not exacerbate coastal flooding conditions.

We have presented a methodology that is practical, reliable, implementable, and generally applicable to most coastal communities along the Atlantic

and Gulf Coasts. An example of application was shown for two typical watersheds of Virginia Beach, Virginia. A city-wide SWMM network was also developed to study drainage system interactions. A modified version of SWMM which includes multiple boundary conditions was used to accurately model the diverse boundary conditions for this system. Work with the citywide model is ongoing; results will be presented in further publications.

The work described in this paper was not funded by the U.S. Environmental Protection Agency and therefore the contents do not necessarily reflect the view of the Agency and no official endorsement should be inferred.

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WASTELOAD ALLOCATION FOR CONSERVATIVE SUBSTANCES

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ABSTRACT

The Environmental Protection Agency is implementing its third round strategy for National Pollution Discharge Elimination System permits. A primary goal is to develop permits which protect toxics criteria in water quality standards. This requires a wasteload allocation (W. A.) that mathematically predicts the amount of substance which may be allowed in an effluent without violating in-stream numerical criteria.

While wasteload allocation methods for conventional pollutants (oxygen demanding substances, for example) are well established and widely used, W. A.'s for conservative substances (toxic metals, for example) are only now being considered. The most common W. A. for conservative substances uses the assumption that the pollution is mixed uniformly across a stream. Since most states standards require that a zone of passage be maintained across a stream, the use of the uniform mixing assumption may result in hundreds of miles of the nation's waters being in violation of water quality standards.

A W. A. has been developed which protects the zone of passage while retaining the desirable features of the mass balance assumption: a minimum of input data required and ease of computation. The development of the W. A. is discussed, the assumptions upon which it is based are examined, and the analytical nature of the W. A. explained.

WASTELOAD ALLOCATION FOR CONSERVATIVE SUBSTANCES

INTRODUCTION

The EPA is now issuing their "third round" of NPDES permits to confirm that aquatic life is being adequately protected on a site-specific receiving stream basis, and the need for a viable wasteload allocation (W.A.) for conservative substances is obvious. A conservative substance remains in the water column and does not undergo chemical alteration. A viable wasteload allocation will yield permit limits which protect instream water quality standards for many toxic substances.

REQUIREMENTS WHICH SHOULD BE MET BY A VIABLE WASTELOAD ALLOCATION

An economical wasteload allocation should use input parameters which may be obtained without the necessity of data collection specifically for allocation purposes. State and federal permitters do not have the resources to perform special measurements every time a permit is drafted. The main advantage of the mass balance allocation, which incorporates an assumption of complete mixing of effluent in the receiving stream, is that it requires only the background concentration, C_B; the stream flow, Q_u; the effluent flow, Q_E; and the water quality standard, C. If C_B is unknown it may be assumed zero. Q_u is either the low flow value obtained from USGS analyses or the minimum flow at which numerical water quality standards apply. C is the concentration of the conservative substance allowed in the receiving stream. Most permitters are experienced in obtaining these input parameters. Furthermore, there is no reason to believe that a more resource intensive wasteload allocation will yield more accurate permit limits on a routine basis.

Because it is undesirable to require data collection for an allocation method which will be routinely used, the dispersion equation (upon which an allocation for conservative substances is based) must be analytical, rather than empirical. An empirical model requires field measurements for calibration each time it is used. Wasteload allocations for conventional parameters (such as dissolved oxygen) are empirical and, therefore, very resource intensive.

The W. A. should not be based on a premise which leads to water quality standards violations. This is not the case for the mass

balance allocation in states which prescribe a zone of passage. A simplistic depiction of the zone of passage is presented in Figure 1. Usually it consists of a fraction of the flow volume or cross sectional area in the receiving stream. Since the mass balance allocation is based on the assumption of complete mixing, the standards will not be protected in those portions of the zone of passage where complete mixing has not occurred. Therefore, the wide use of mass balance allocations creates the potential for standards violations in hundreds of miles of the nation's waters.

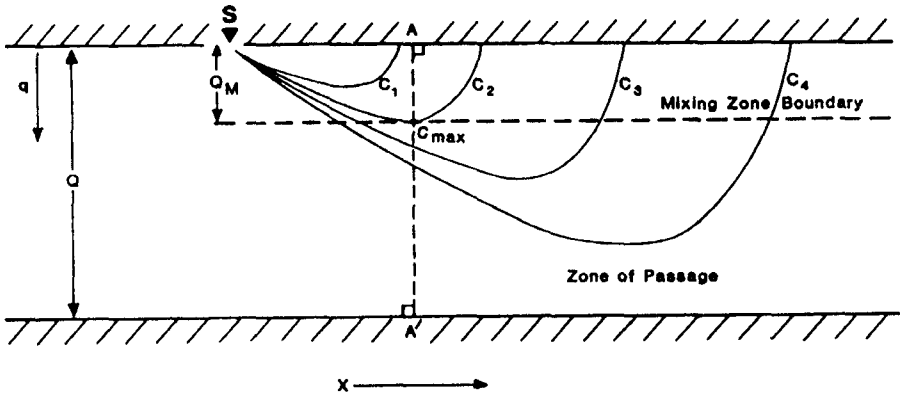

Figure 1. Mixing zone, zone of passage and plume dispersion in a receiving stream.

Since numerical standards are enforced only in the zone of passage, a regulatory mixing zone, where numerical criteria are not applicable, is created (Figure 1). Any resemblance between the regulatory mixing zone and the mixing zone created by dispersion of a conservative substance in a receiving stream is purely coincidental. The dispersion is represented in Figure 1 by isopleths of concentration, with C_1 being the maximum and C_4 the minimum concentration depicted. The maximum concentration on the boundary between the regulatory mixing zone and the zone of passage is C_2; and this point is labeled C_{max}.

DERIVATION OF THE WASTELOAD ALLOCATION EQUATION

The law of conservation of mass may be used to develop a wasteload allocation which will protect the zone of passage. In Cartesian coordinates, conservation of mass may be expressed as (1)

$$\frac{\partial \Theta}{\partial t} + \frac{\partial (\Theta Wx)}{\partial x} + \frac{\partial (\Theta Wy)}{\partial y} + \frac{\partial (\Theta Wz)}{\partial z} = 0 \qquad \text{(A)}$$

where Θ (x, y, z, t) is the instantaneous concentration of the conservative substance in the receiving stream, and W_x, W_y and W_z are instantaneous stream velocities in the x (downstream along the bank), y (vertical) and z (transverse) directions.

If assumptions of questionable validity are used in the derivation

of a W. A. procedure, then a verification program which would tax the resources of state and federal agencies would be required before the W. A. could be used for NPDES permitting activities. For this reason, (A) was chosen as the premise upon which the allocation is based. A rather complicated set of assumptions is necessary to obtain a useful W. A..

The desired solution to (A) requires the following assumptions:

1. Mass is conserved

   The pollutant does not change form chemically or volatilize during travel from S to C_{max}. The pollutant is neutrally buoyant during this travel time, so it does not settle out of the water column.

2. Steady state conditions exist.

   The effluent flow, effluent concentration and mean ambient flow must remain constant for a longer period than the travel time from the source S to the point of maximum concentration on the mixing zone boundary (C_{max} in Figure 1).

3. No persistent transverse currents.

   While random transverse currents (in the form of turbulence for example) are necessary for dispersion at the rate observed, no large whirlpools which create a persistent transverse current can be tolerated.

4. Complete vertical mixing

   In shallow streams, vertical mixing occurs within a few hundred feet of the discharge (1). If the fraction of the flow allocated to the zone of passage is sufficiently small, C_{max} will be far downstream of the point of complete vertical mixing.

5. Concentration is half-normally distributed in the transverse direction.

   Yotsukura and Sayre noted that in the Natural coordinate system concentration distributions are normal in the transverse direction (1).

6. Negligible reflection from the far bank

   If the fraction of flow allocated to the zone of passage is sufficiently large, then only a small fraction of the mass of the conservative substance will have even reached the far bank at C_{max}.

7. Stream flow remains constant between S and C_{max}

   There can be no flowing tributaries or significant water withdrawal between S and C_{max}. Only larger tributaries flow during critical conditions.

8. The discharge is a point source located at the bank.

   Most discharges are via pipes which project a negligible distance into the stream and do not have enough velocity to produce a transverse flow at C_{max}.

9. Stream depth and velocity change gradually.

   This assumption is more valid for tranquil valley streams than for turbulent mountain streams. Fortunately, there are relatively few discharges to turbulent streams.

10. The background concentration is constant

    No sources or sinks of pollution exist between S and C_{max}

11. The dispersion coefficient is constant in the vicinity of C_{max}.

    While the dispersion coefficient is not constant close to the source, if the fraction of the flow allocated to the zone of passage is small enough, then C_{max} will occur far enough downstream so that the assumption is valid.

Using these assumptions, an analytical solution to (A) may be obtained (2):

$$c = \frac{\sqrt{2}\ W}{\sqrt{\pi}\ \sigma} \exp\left(-\frac{q^2}{2\sigma^2}\right) + C_B \qquad \text{(B)}$$

where c is the steady state concentration, W is the wasteload (the product of the effluent flow and concentration, i.e. $W = C_e\ Q_e$ where C_e is the effluent concentration) σ (x) is the concentration standard deviation in the transverse direction, q is the stream flow between the injection bank and the point in the stream where the concentration is c (Figure 1 shows that at the far bank q=Q, the total flow in the stream and at the mixing zone boundary $q = Q_m$). Equation (B) is underspecified because both c and σ are unknown.

The solution (B) yields the concentration of the conservative substance at any point in the receiving stream. However, for wasteload

allocation purposes, the only point at which the concentration must be known is at C_{max}. Figure 1 shows that if the standard is not exceeded at C_{max} it will not be exceeded anywhere in the zone of passage, because at every other point the concentration is less than C_2.

The concentration distributions along the mixing zone boundary and along the transverse cross section A, A' are displayed in Figures 2a and 2b, respectively. Figure 2a shows that the gradient of the concentration distribution is zero at the point of maximum concentration (3). Since the only other variable in (B) which is dependent on x is σ, (B) may be differentiated w/r to x to obtain $\sigma = Q_m$. Since Q_m is the mixing zone specified in the water quality standards, our closure problem is solved. Substitution for σ and defining $C' \equiv c - C_B$ in (B)

$$W = C_e Q_e = 2.066 C' Q_m \qquad (C)$$

Since $c = C_{max}$ when $\sigma = Q_m$, if C_{max} is set equal to the water quality standard, then (D) yields a wasteload allocation which protects the zone of passage. The allowable wasteload depends upon the water quality standard, the background concentration, the fraction of the flow allowed for a zone of passage and the stream flow. The method described by (C) will be called the mixing zone wasteload allocation.

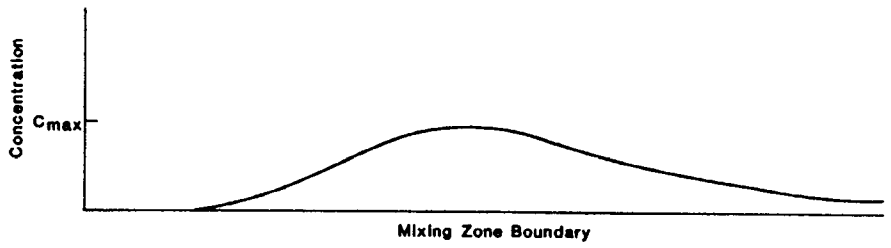

Figure 2a. Concentration on the mixing zone boundary.

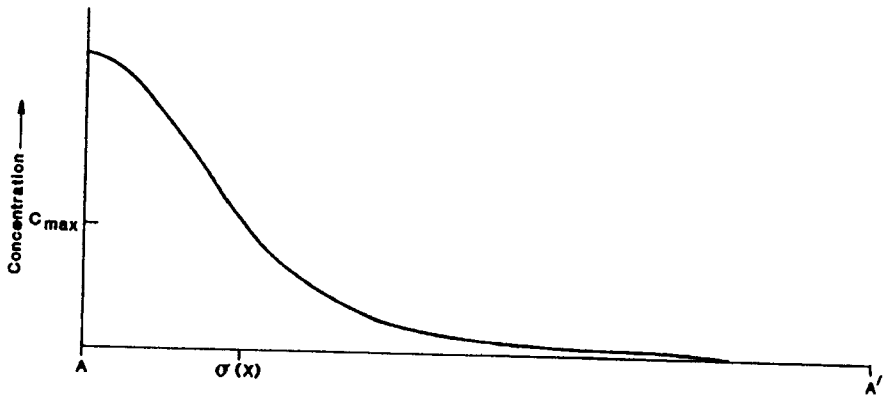

Figure 2b. Concentration on cross section A, A'.

ANALYSIS OF THE MIXING ZONE WASTELOAD ALLOCATION

If C_B is large in relation to the water quality standard, C_e may be very small or even negative. This is caused by substituting the water quality standards for c in (C). For water quality management purposes, C_e should never be required to be smaller than the standard.

Many states reserve three quarters of the stream flow for a zone of passage. In this case $Q_m \cong \frac{Q_u + Q_e}{4}$.

(C) may be rearranged by substituting for Q_m and setting $C_B = 0$ to obtain

$$\frac{C_e}{C_{max}} = .5165 \frac{1 + Q_e^*}{Q_e^*} \qquad \text{(D)}$$

where $Q_e^* \equiv Q_e/Q_u$ and C_{max} is the water quality standard.

Figure 3 shows C_e/C_{max} plotted against Q_e^*. When Q_e^* is large (effluent flow is large in comparison with the stream flow), the effluent concentration (C_e) allowed by the mixing zone allocation (solid line) is small. When the effluent and stream flows are the same size ($Q_e^* = 1$), the effluent is required to nearly meet water quality standards ($C_e/C_{max} \rightarrow 1.$). When the effluent is much less than the stream flow ($Q_e^* << 1$), then C_e is allowed to be much greater than the standard

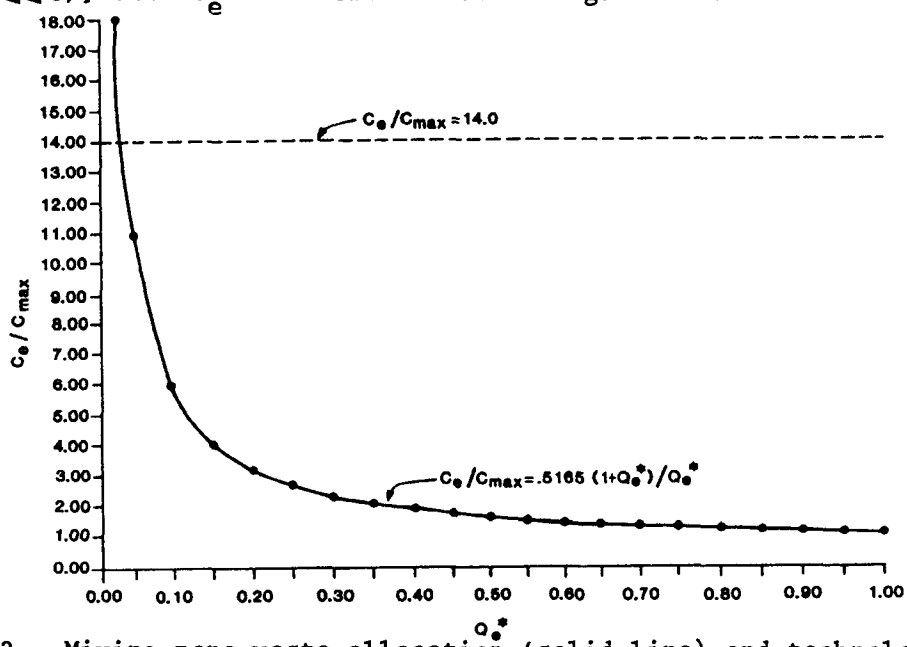

Figure 3. Mixing zone waste allocation (solid line) and technology based permit limit (dashed line). Q_e^* is the ratio of the effluent discharge flow to the upstream dilution flow. C_e/C_{max} is the the ratio of the effluent concentration to the water quality standard.

($C_e/C_{max} >> 1$). The mixing zone allocation accounts for the dilution capacity of the stream by allowing a higher effluent concentration when the dilution capacity is greater.

If the dilution capacity is extremely large ($Q_e^* >>> 1$), then the mixing zone wasteload allocation allows the effluent concentration to be almost unlimited. In this case, a technology based limit is required. In Figure 3, a technology based limit is represented by a dashed line. Because it does not depend upon dilution capacity, C_e/C_{max} is constant. For water quality management purposes, the more stringent of the technology based or water quality based permit criteria should be used to limit effluent concentration. In Figure 3, if, $Q_e^* < .025$, a technology based permit is appropriate. Otherwise, the permit limit obtained from the mixing zone wasteload allocation should be used.

It may be shown that the mixing zone allocation always yields a more stringent permit limit than the mass balance allocation does. If $C_B = 0$, the mixing zone allocation may be written as (2)

$$\frac{C_e}{C} = \frac{1 + Q_e^*}{Q_e^*} \qquad \text{(E)}$$

A comparison of (D) and (E) reveals that the mixing zone allocation is nearly twice as stringent as the mass balance allocation.

CONCLUSIONS

The mixing zone allocation is as simple to use as mass balance, but does not allow standards violations. It is easily incorporated into a microcomputer program, which, depending on state standards and regulations, may incorporate technology based permit limits and other wasteload allocations. In Oklahoma, a microcomputer accepts input data which does not require field collection, determines the type of permit required, develops permit limits, and writes a portion of the rationale for the discharge permit.

Field validation of the mixing zone wasteload allocation method is not necessary, because the assumptions upon which it is based are valid for a natural stream. This is fortuitous since the location of C_{max} is unknown and the concentration is changing rapidly in the transverse direction in its vicinity (Figure 2b).

The work described in this paper was not funded by the U. S. Environmental Protection Agency and therefore the contents do not necessarily reflect the views of the Agency and no official endorsement should be inferred.

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THE USE OF DETAILED COST ESTIMATION FOR DRAINAGE DESIGN PARAMETER ANALYSIS ON SPREADSHEETS

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ABSTRACT

Extensive research in the field of sewer system design has produced many excellent computer programs based on optimization and simulation techniques. The use of these programs, however, has been limited because they require specialized skills in programming and applied mathematics. This paper describes the computerization of the current Florida Department of Transportation (FDOT) drainage design procedures with the use of spreadsheets. By replicating the current manual procedure, the spreadsheet permits the engineer to perform calculations in a familiar format. The spreadsheet design procedure allows the user to vary pipe sizes and slopes manually in a trial and error method while monitoring the system constraints. The spreadsheet also automatically provides a detailed cost estimate of the current system based on the FDOT itemized drainage cost database. The template will update all system calculations of pipe flow capacities, velocities, pipe elevations, and system cost estimates with a change in a pipe size or slope.

The advantages of the spreadsheet design procedure over currently used techniques are discussed. The advantages of using a detailed cost estimate over a functionalized cost estimate, which may depend on one or two parameter values, are also discussed. Savings induced by a redesign and cost estimate using the spreadsheet template on the FDOT Thomasville Highway project are presented to illustrate the method.

INTRODUCTION

Research in the field of sewer system design has brought steady improvements in design procedures through the use of computerized simulation and optimization models. The use of these new procedures for actual designs, however, often lags far behind. Because of precedent and time and money constraints, many designers continue to use established design procedures. Often the only considerations of system cost in the design process is through

use of rules of thumb and post-design cost reviews. Even while remaining within the guidelines of a design procedure, many combinations of pipe sizes, pipe depths, and structure types may be feasible for a working system. The cost of these feasible systems, however, may vary greatly. If the design calculations are performed manually, the number of alternatives evaluated may be severely limited. The intent of this paper is to present a methodology whereby design procedures are improved and computerized in a manner that will be accepted by practicing designers. This process will necessarily involve incremental changes in the current design procedures.

The spreadsheet provides a computerized environment where these incremental changes may take place. In a spreadsheet, a design calculation procedure may be organized into a format that is familiar to engineers who use the analogous hand calculation procedures. Also the input and output of data is done in a natural manner. These features enable the computerization of hand calculation procedures to be done relatively easily on spreadsheets.

For this paper a spreadsheet replication of the Florida Department of Transportation (FDOT) highway storm drainage design procedures was created. The Lotus 1-2-3 spreadsheet package was used to computerize the FDOT design procedures so as to duplicate their design calculations as closely as possible. The spreadsheet procedure, however, offers many advantages over the hand calculations. Most importantly, computerization allows the calculations to be performed many more times and encourages improvement of the design by trial and error. Also, an automatic cost estimation scheme has been included in the spreadsheet. A change in the system design will produce a change in the cost estimate of the design. With this ability, the user may develop a heuristic algorithm to proceed through the system design. A spreadsheet design procedure also allows the user to transfer any personal style or techniques which may have been used in performing hand calculations to the computerized procedure.

LITERATURE REVIEW

The history of computerized optimization of sewer systems dates back two decades to papers by Liebman (1) and Holland (2). Since this beginning, optimization algorithms have been used which made simplifying assumptions in order to solve the sewer design problem. Liebman's linear programming algorithm only dealt with the network layout, and Holland's nonlinear algorithm could not handle discrete pipe sizes. The most common simplification in these techniques is the use of a cost estimation function. Many of the dynamic programming models use functions to determine cost as a function of pipe size and invert depth (e.g. Zepp and Leary (3), Merritt and Bogan (4)). In each of these optimization techniques much time is spent on defining the assumptions so that errors are minimized. System constraints must also be carefully defined. The definition of these constraints is a difficult job and Merritt and Bogan (4) admit that, "It is unlikely that any optimization method achieves a true optimum when the full scope of a real world setting is considered."

A survey of water resources personnel performed by Austin (5) concludes that a lack of models that represent "real world" situations was a common complaint of model users. This survey also showed that simulation models are much more widely used than optimization models. Simulation models are helpful, but are awkward for system design because of their batch run format. A reliable simulation model used for verification of a simple optimization technique however, would be very valuable for system design. Spreadsheets have been used successfully in a wide range of water resources applications from stormwater permitting and groundwater modeling (Hancock (6)) to model preprocessing (Miles and Heaney (7)). Hancock developed a decision support system for following stormwater discharge permitting procedures on a spreadsheet. Spreadsheets have been very useful for database needs and have extensive calculation and programming capabilities.

BACKGROUND OF STUDY

This study began with the intention of designing an experiment which would determine whether the use of computerized models in designing stormwater drainage systems could be economically justified. In order to conduct such an experiment, past projects must be analyzed and redesigned using new procedures. The search for past projects led to the Florida DOT.

The Florida DOT has drainage design procedures which are very well documented in their Drainage Manual (8). They also have a large number of past project designs available in blueprint form as well as planning calculation form. The most extensive database of their past projects, however, is found in their cost estimating department. An itemized unit cost which is based on average bid prices from past projects is available for each highway construction item. This database is updated every six months and the item numbering system allows drainage related items to be determined easily.

The FDOT Drainage Manual lists a mainframe Fortran program called "Draino" (PEGDRG32) as available to assist in drainage design. The program uses a heuristic algorithm that minimizes pipe costs of a drainage system. This program is seldom used, however, because of its difficulty in handling real world problems and its tedious data input procedure. Users find it difficult to define problem constraints and the program makes the assumption that all pipes are designed to flow full.

CURRENT DRAINAGE SYSTEM DESIGN PROCEDURE

Presently, Florida Department of Transportation (FDOT) personnel perform most of their drainage calculations using the worksheet shown in Figure 1. Explanations of a few of the entries needed on this tabulation form are given in an excerpt from the FDOT Drainage Manual shown in Figure 2. These calculations are most commonly performed by hand or with a nomograph. Repetitive hand calculations are subject to error and do not encourage the engineer to "push the limits" of the design criteria to find an optimal design. The tabulation form procedure also does not explicitly include system cost as a design criterion. The tabulation form has become the accepted practice for highway

B.I. No.

STATE OF FLORIDA DEPARTMENT OF TRANSPORTATION

STORM DRAIN TABULATION FORM

SHEET NO.

DATE _____ PROJECT NO. _____ ROAD _____ COUNTY _____ BY _____ OF ___

LOCATION OF UPPER END			STRUCTURE NO.	TYPE OF STRUCTURE	TYPE OF LINE	LENGTH (FT.)	DRAINAGE AREA (ACRES)			TIME OF CONCENTRATION (MIN.)	TIME OF FLOW IN SECTION (MIN)	INTENSITY	TOTAL (CA)	TOTAL RUNOFF (CFS)	INLET ELEVATION (FEET)	ELEV. OF H.G. / CROWN ELEV. / FLOWLINE ELEV.			DIAMETER (IN)	SLOPE (%)	VELOCITY (FPS)	CAPACITY (CFS)	NOTES: ZONE -FREQUENCY YR; n=.012 CONCRETE PIPE & CULVERTS; n=.024 CORRUGATED METAL PIPE	
STATION	DIST	S					INCREMENT	SUB TOTAL	SUB-TOTAL (CA)							UPPER END	LOWER END	FALL (IN FEET)						REMARKS
3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19 / 21 / 23	20 / 22 / 24	25 / 26	27	28	29	30		31

Figure 1. Recommended Storm Drain Tabulation Form

14. Time of Flow in Section (min)

This is the time it takes the runoff to pass through the section of pipe in question; it depends on the velocity as well as the condition of flow (i.e., gradient or physical flow time based on proper condition and velocity).

15. Intensity

Intensity values are determined from one of the 11 intensity-duration-frequency (IDF) curves developed by the Department and presented in Chapter 5 of this volume. Intensity depends on the design frequency and the time of concentration.

16. Total (CA)

The total CA is the sum of the subtotal CAs.

17. Total Runoff (cfs)

Total runoff is the product of the intensity and the total CA, less inlet bypass and exfiltration.

Figure 2. Excerpt from FDOT Drainage Manual (8) showing descriptions of tabulation form entries.

drainage design. Designs obtained with the use of a computer model or alternative method must be compared to the tabulation form design. With the present time and budget constraints, the prospect of extra work is a deterrent to the use of modeling. Therefore, any improvement in design procedures must also include an improvement in the efficiency of the time spent on the design. The use of a spreadsheet in performing these design calculations may help to increase this efficiency.

PROPOSED SPREADSHEET DESIGN PROCEDURE

A spreadsheet template has been created that replicates the design procedures used by the FDOT on their tabulation forms. The spreadsheet template is divided into four areas: input area, initialization area, interactive design area, and database. Each of these areas will be described in the following sections.

DATA INPUT AREA

Information is input into the spreadsheet much like it would be written onto the FDOT tabulation forms (see Figure 3). This information includes the pipe identifications (to and from nodes), pipe lengths, ground elevation at nodes, and peak flow values. If the Rational Method is used to calculate peak flow values, then drainage areas, runoff coefficients, times of concentration, and a design frequency must also be entered. The spreadsheet can determine the storm intensity from a time of concentration and design frequency by using the FDOT intensity-duration-frequency curve regression equations (FDOT Drainage Manual 1987). The user must also include the type of manhole or inlet at each node if this cost is to be contained in the cost estimate used for system optimization. These costs are important in the cost estimation because structural costs are given as a function of depth in the FDOT database. A survey station number and the type of line (main, stub, etc.) may also be included in the input area for user convenience.

```
D9: [W8]                                                                MENU
Start [Initialize] Recalculate  Optimize  Base cost  Change
Calculates initial values of pipe system parameters.
    A  B C  D        E      F     G    H    I    J     K    L    M
1         DOT Program Simulator
2         Input area                 Enter values only in unprotected
3                                    areas
4
5   Pipe                        Type      Len.  Incre.          Min
6   ID                   Struc. of   Len. to    area  Runoff    t.c.
7   Fr To  Station Struc. Spec.  Line (ft) outlt (acr) Coef.     (min)
8   ---------------------------------------------------------------
9         :
10  62 61 :1131+00 Inlet  E      M     365  3275  3.80 C1=  0.45    10
11        :                                     0.00 C2=   0.8
12        :                                     0.00 C3=  0.25
13  61 60 :1127+58 Inlet  G      M     150  2910 22.20             12
14        :                                     0.00
15        :                                     0.00
16  60 58 :1126+48 Inlet  J-3    M     190  2760  0.00              0
17        :                                     2.10
18        :                                     2.00
19  59 58 :1124+58 Inlet  P-2    S      76  2646  0.00             10
20        :                                     0.80
                                          CMD       CALC
```

Figure 3. Spreadsheet data input area with menu.

INITIALIZATION AREA

Once the input data has been entered into the storm drainage design template, the user may choose to use the pipe size and slope initialization algorithm. The initialization algorithm is analogous to the one used by the DOT "Draino" program (8) and described here. It is not required, however, to perform the initialization algorithm in the spreadsheet design procedure. The algorithm first determines the lowest allowable hydraulic grade line elevation in the system. The minimum slope from this point to the outlet is then calculated and all pipes downstream from this point are assigned this slope. All pipes upstream from this point are assigned the ground slope. Initial pipe sizes are then calculated with respect to these slopes and the design flow. A macro program may now be executed to assist the user in moving the initial pipe sizes to the design area.

The flow process in this template is controlled by a Lotus 1-2-3 macro program. Menus have been created which are much like the menus that execute the Lotus 1-2-3 commands. These menus, also shown in Figure 3, allow the user to move easily throughout the spreadsheet and also begin the execution of macro programs. The Lotus menus also provide a brief description of each macro choice to which the cursor is moved. The initialization macro is one of the algorithms which may begin by calling the menu and choosing the desired macro.

INTERACTIVE DESIGN AREA

The design area in the template is constructed as shown in Figure 4. This area is designed to keep the most important system design parameters showing on the screen. Lotus 1-2-3's ability to create column and row titles is used here. Titles can remain at the top of the screen while the cursor is moved down to view pipes lower in the network (below row 20). Similarly, the three columns to the far left of this area (A, B, and C) are also titles and will always remain on the screen. This feature enables the user to always know the current pipe identification.

The optimization of the drainage system is based on varying a pipe size and slope while monitoring the resulting calculations such that they remain within designated criteria. For example, a pipe may be reduced by one size and then checked to make sure that its flow capacity exceeds the required design flow. If it does not, the user may then try to increase the slope of the pipe to increase the flow capacity while verifying that the velocity does not exceed the maximum and that the minimum cover is kept above the pipe crown. This feature allows the engineer to employ personal strategies in a system design. Excavation can be minimized by following the ground slope as closely as possible, or steeper grades and smaller pipes can be used in the upstream regions of the network.

The user may also easily define system constraints in this area. For example, if an existing utility constrained a pipe invert to be at a 100 foot elevation, then the cell formula which had previously calculated the invert

```
BN9: [W6] +INVERT+DIAMETER/12                                          READY

    A  B CBH  BI  BJ   BK   BL    BM    BN    BO    BP   BQ  BR   BS    BT
1                                          Old system cost= $  108112
2                                          New system cost= $  108112
3                                          Pipe material = rcp III
4         T                Guttr/Crown Crown            D
5  Pipe   Y Tot. Full      Grate Invrt Invrt Grnd   Pipe R   Pipe
6  ID     P Flow Flow Vel Elev. Upper Lower Slope Slope O   Dia. Pipe Struc
7  Fr To  E cfs  cfs (fps) (ft)  (ft)  (ft)        S   P   (in) Cost Cost
8  ---------------------------------------------------------------------
9         :                     144.9 133.5
10 62 61 :M 8.87 12. 10.0   148 143.6 132.3 0.032 0.031 1.5  15 6872. 4360
11        :
12        :                     133.3 130.9
13 61 60 :M 56.6 63. 12.9   136 130.8 128.4 0.003 0.016   0  30 7807. 1734
14        :
15        :                     130.9 127.5
16 60 58 :M 66.3 67. 13.8 135.5 128.4 125.0 0.008 0.018   0  30 9889. 2785
17        :
18        :                     130.8 127.7
19 59 58 :S 4.35 11. 9.56 133.8 129.5 126.5     0  0.04 1.5  15 1431. 1691
20        :
                                                     CALC
```

Figure 4. Spreadsheet interactive design area.

elevation could be replaced with the constant 100. If the constraint limited the invert elevation to be greater than 100 feet, then the previous cell formula could be replaced with a conditional formula which gives the constant value of 100 if the calculated value is less than 100. In this manner, the spreadsheet provides a simple method for defining individual system constraints.

Cost Estimation

The optimization of the drainage system is performed primarily to save on costs while not decreasing the reliability. Since a reduction in size of almost any pipe will reduce the reliability in some way, the optimization should be an effort to conform more precisely to the defined criteria (e.g. to withstand runoff from the 25 year storm). The procedure would, in effect, minimize the costs of meeting the prespecified standards.

The first step in reducing the cost of a system is to acquire a feel for the distribution of cost within the system. If a detailed cost estimate of a system is known during the planning stage, then the search may concentrate on the areas of most probable savings. For example, more effort should be spent on reducing a 200 foot 72 inch diameter pipe by one size than on reducing a 50 foot 18 inch diameter pipe.

A detailed cost estimate of a drainage system includes quantities and unit costs of installed pipes, inlets, and manholes. The spreadsheet template can update the system cost to reflect changes in the system design by using DOT itemized average bid data. With this ability, the user may quickly find the design areas in the system with the largest potential savings and may easily define tradeoffs between parameter refinement and system cost.

The spreadsheet provides an escape from functionalized cost estimates which may depend on one or two parameters. The most common sewer cost estimation functions give cost as a function of pipe diameter (Grigg and O'Hearn (9), Arnell (10)). Others include parameters such as invert depth and flow (Tyteca (11), Han et al. (12)). If used as design criteria, these functions may deceive the user into believing that a parameter value is of more or less importance than it actually is. The itemized cost estimation allows the user to see the true relationships between parameter values and cost and to optimize the system accordingly.

Hydraulic Design

In addition to cost information, the design area of the spreadsheet template provides automated calculations for the pipe flow capacity at the given slope, the velocity at the design flow, and the crown and inlet elevations at each structure. For each refinement in a pipe slope or size, the resulting calculations are performed throughout the remainder of the system. This feature encourages a trial and error approach to the system refinement since the user is not required to perform extra calculations if the refinement proves faulty. The template also provides a series of automated criteria checks which will alert the user to a criterion violation in the system which was the result of parameter change.

The automation provided in many of the steps in this design template is not required in order to perform the calculations. It is often the case that the design may be performed more easily without automatic criteria checks. This is especially true when a large (greater than 20 pipes) drainage system is being designed. The recalculation time of the template increases proportionately to the number of pipes in the system. A long recalculation time hinders the desired trial and error approach to the system design. To solve this problem, a small macro program has been written that will recalculate only the parameters for the pipe that is presently being optimized. This procedure has the disadvantage that the automated criteria checks are disabled, but the recalculation time is drastically reduced. This procedure actually simulates the hand calculation procedure very closely by moving up the system pipe by pipe, but encourages a fine-tuning of the system parameters.

The template does not yet include a procedure for calculating the hydraulic grade line, but work on this addition is currently underway. The template does, however, correct velocities in pipes that are not flowing full at design flow. This correction is done using a method developed by Christensen (13). The method was derived by a Fourier analysis of experimental data relating the depth in a partially full pipe to the water velocity and flow in that pipe. Since the design flow and the full flow of the pipe will be known in the design, the partial flow depth and velocity may be calculated using the iterative method developed by Christensen.

Database Area

The database area of the spreadsheet template contains the FDOT itemized unit costs for all drainage items. These unit costs are extracted from the database and used elsewhere in the spreadsheet with the Lotus lookup table function. The database area also contains the regression coefficients for the FDOT intensity-duration-frequency curves. These curves give storm intensity to be used in the Rational Method given the duration, frequency, and zone number of the location. The spreadsheet template automatically updates the intensities obtained from the regression curves given a design change in the storm frequency or in the time of concentration of a subcatchment.

THOMASVILLE HIGHWAY CASE STUDY

Thomasville Highway was a reconstruction project which consisted of widening a two lane rural highway into a four lane urban highway in Tallahassee, Florida. The project was carried out by the FDOT in three phases over a span of six years.

Several characteristics of the Thomasville Highway project made it a desirable case study. The topography of the area showed significant relief for Florida (maximum of 3% grade) and so there existed the potential for a pipe size and slope tradeoff in the system design. The availability of the project blueprints and drainage planning calculations allowed an analysis of the design procedures. Since the drainage design calculations were given on the tabulation forms, a direct comparison to the spreadsheet design procedure could be made. Each project phase consisted of an independent system that emptied to a single outfall. Each of the phases also involved similar general geographic, topographic and urban development characteristics. These characteristics were deemed desirable for the evaluation of the effectiveness of using detailed cost estimation as opposed to cost functions.

The evaluation of cost estimation requirements for design involved the analysis of pipe length and pipe cost distributions. A plot of actual design pipe lengths for each pipe diameter, expressed as a percentage of the total pipe length, for each project phase reveals dissimilar distributions. Analogous plots for pipe cost distributions also are dissimilar. Figures 5a&b show a large percentage of pipe length and cost centered in smaller diameter pipes (18-36 in. dia.). For phase 3516, Figures 5c&d reveal a bimodal distribution of pipe length with no intermediate pipe sizes (42-56 in. dia.) required. However, the cost distribution for this phase is unimodal with 75%

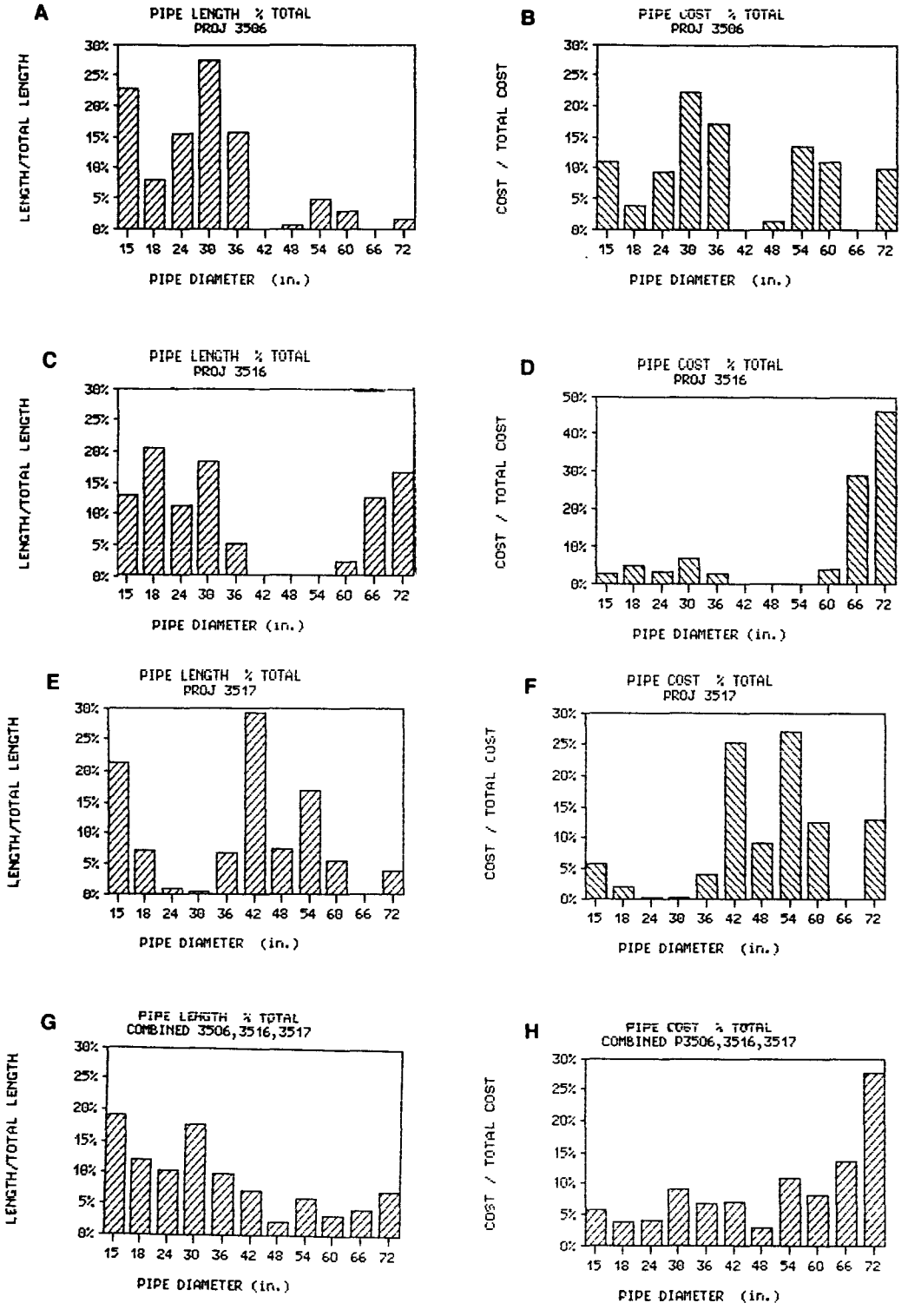

Figure 5. Pipe length and cost distributions for Thomasville Highway Project.

of the total cost associated with the large diameter pipe. This suggests the possibility of large savings through reduction of pipe size for large diameter sections. A third pattern of length and cost distribution is shown in Figure 5e&f for phase 3517 with length and cost being centered in intermediate diameter pipe. When the distributions for the overall project are generated (Figures 5g&h) the pipe length distribution appears lognormal with larger percentages in the smaller diameter pipes and a decreasing percentage as pipe diameter increases. The cost distribution for the overall project reveals the inverse relationship between cost and length distributions. These combined distributions smooth out the dissimilarities between each of the individual phase distributions.

The well behaved distributions for the overall project, if taken alone, would not reveal the significance of cost estimation and cost feedback in the design process. The dissimilarities of the distributions for separate project phases can possibly be related to distance and slope from the outfall to inlets where large areas contribute inflows to the system. Cost functions based on and requiring such data appear more difficult to use at the design level than straightforward detailed system costing. Each independent system within the overall project requires a different design emphasis in order to incur savings recognized by a detailed cost estimation. Without the use of computer assistance, a detailed cost estimation as a part of the design decision process would be a very labor intensive task. The spreadsheet design template, however, easily incorporates the detailed costing into the design procedure.

Cost estimates for each of the three sections of the Thomasville Highway were performed using the current FDOT itemized unit costs. A redesign of phase 3517 using the spreadsheet design procedure produced a savings of over 10% in pipe costs over the original design. The pipe costs are estimated to be about 75% of the total drainage system costs.

CONCLUSIONS

1. Optimization and simulation methods are being used very little in the design of storm drainage systems because of a lack of understanding of their techniques and a difficulty in defining real world problem constraints. Also the time and effort needed to run computer programs are often difficult to justify when working with limited time and money. A spreadsheet design technique which is simply a modified version of presently used design procedures will be more readily accepted by drainage engineers.

2. The detailed cost estimates provided by the spreadsheet allow the engineer to directly examine the components of the system which will incur the most savings. Itemized cost databases have the advantage over cost functions of being updated as a response to current item costs while cost function update requires a reanalysis of cost data.

3. Spreadsheet drainage design allows for easy definition of system constraints and allows for engineering judgment which is not easily programmed into a computerized algorithm.

4. Users feel comfortable with a solution obtained by fine-tuning the system by trial and error because they have performed the calculations. Users quickly get a feel for relationships between system parameters and cost while directing the procedure toward a solution which meets both engineering and design budget criteria.

ACKNOWLEDGMENTS

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CORRECTIVE PHOSPHORUS REMOVAL FOR URBAN STORM RUNOFF AT A RESIDENTIAL DEVELOPMENT IN THE TOWN OF PARKER, COLORADO

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ABSTRACT

This paper discusses runoff water quality management for a residential development in the Town of Parker, Colorado. The project is located in Cherry Creek drainage basin, southeast of Denver, where reduction of non-point pollutants, particularly phosphorus, is required for the development. The primary intent of this program is to reduce phosphorus loading and eutrophication of Cherry Creek Reservoir. Facilities include two detention ponds, storm sewers, grass-lined channels, a combined detention and sedimentation basin, pump station and an irrigation system. The system would comply with "Criteria for the Control of Erosion and Non-Point Source Pollution", a runoff water quality enhancement guideline for the Town of Parker.

The irrigation system, which was already needed for the development, is felt to be a preferable phosphorus removal system over present guidelines which suggest constructing a filtration system.

The soils are conducive to infiltration. An underdrain system was proposed to augment treatment during wet periods and to provide monitoring opportunities. It is perceived that the filtration system will have significant maintenance problems because of sediment accumulation on the filtration bed and reliability/performance problems with actual phosphorus removal because filtration cannot remove dissolved phosphorous. Other than processes involving chemical treatment, it is more reliable to involve treatment where a soil column and plant uptake of phosphorus is involved. The system has other benefits such as reducing the need for groundwater for irrigation needs.

BACKGROUND

The Town of Parker is in the Cherry Creek basin, which drains to Cherry Creek Reservoir. Cherry Creek Reservoir is located southeast of Denver. The reservoir was determined to be slightly eutrophic in the National Eutrophication Survey (Ref. 1) (NES) conducted by U. S. Environmental Protection Agency between

1972 and 1975. In 1984, the Cherry Creek Reservoir Clean Lake Study was conducted by Denver Regional Council of Governments (DRCOG) to establish water quality goals and standards related to eutrophication as well as recommend treatment levels to achieve those goals and standards.

A total phosphorus standard of 0.035 mg/l for Cherry Creek Reservoir was adopted by the Colorado Water Quality Control Commission (CWQCC) in September, 1985 (Ref. 2). In order to maintain this phosphorus standard in Cherry Creek Reservoir, the annual load of total phosphorus has to be reduced. Non-point stormwater runoff was estimated as the major contributor (77%) of phosphorus to the reservoir. Therefore, water quality control measures were called for which would be capable of removing 50% of the total phosphorus load for non-point storm water runoff for all the developments in Cherry Creek basin.

The Town of Parker had the firm of HydroDynamics prepare a manual entitled "Criteria for the Control of Erosion and Non-point Source Pollution" (Ref. 3). In order to achieve the desired goal, performance standards are cited (Table 1) for various types of development. These are based on the regional studies (Ref. 3).

TABLE 1: PERFORMANCE STANDARDS FOR PHOSPHORUS REMOVAL FROM DEVELOPED LANDS

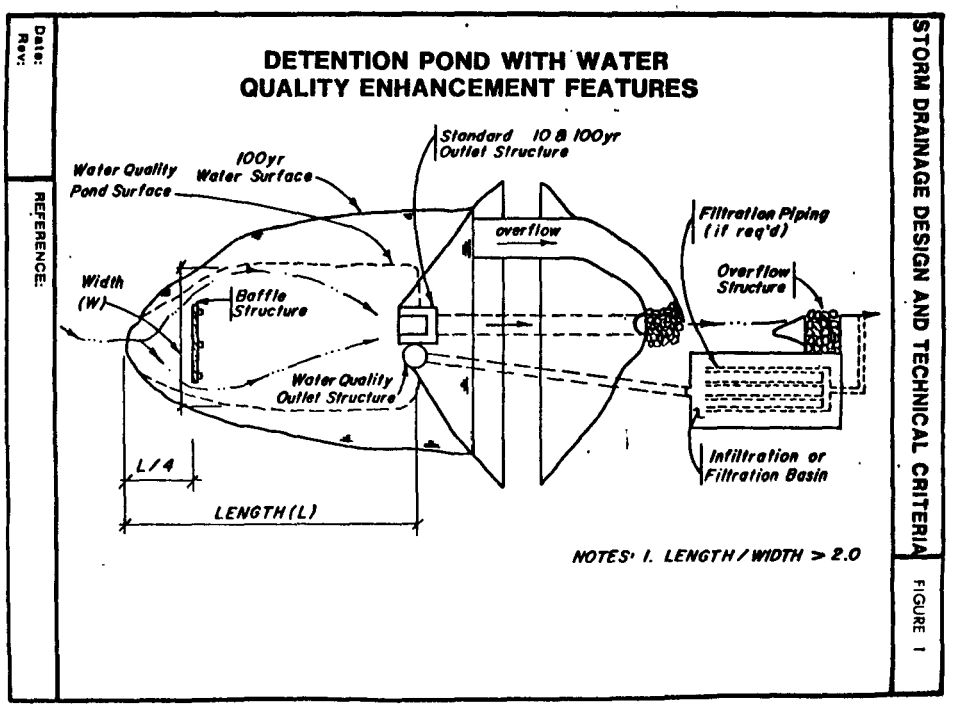
Land Use	Phosphorous Removal
Residential	45%
Commercial	70%
Public Areas	50%

Control measures such as retention, filtration, infiltration, and wetland application are discussed. General efficiency factors are given in Table 2.

TABLE 2: MITIGATION MEASURE EFFECTIVENESS FOR REDUCING TOTAL PHOSPHOROUS (Ref. 3)

Measure	Reduction Efficiency
Retention	25%
Infiltration	90%
Filtration	50%
Wetland Application	75%

Retention is basically storage, sedimentation and very slow release. Infiltration generally includes methods to utilize the native soil capability. This method is preferred, but concerns are expressed (Ref. 3) as to the inherent loss of area where this can take place because of development. Filtration refers to a sand bed filter, which is typically downstream of a retention basin. Figure 1 illustrates a schematic of the pond and filtration basin. Figure 2 depicts the pond outlet and Figure 3 the filter drain layout. (Figures copied from Ref. 3). Wetlands application refers to flow through wetlands where both sedimentation and phosphorous removal take place as the plants trap and utilize phosphorous.



Tables 3, 4, and 5 are the Reduction factors provided by retention, filtration, and infiltration methods for various amounts of runoff treated (Ref. 3). As the amount treated increases the reduction factor approaches the efficiencies cited in Table 2. A relative removal efficiency is used to adjust for higher or lower removal efficiencies, such as might be documented by pilot tests.

$$\text{Relative Removal Efficiency} = \frac{\text{Corrected Removal Efficiency (pilot test or other data)}}{\text{General Removal Efficiency (Table 2)}}$$

Thus the effectiveness can be computed for each type of development receiving a given treatment by multiplying the area treated (in terms of percentage of the study area) by the reduction factor and the relative removal efficiency. the cumulative total of all treatments is then the overall effectiveness.

PROJECT DESCRIPTION

Total drainage area of the proposed project was 358.5 acres. An initial drainage plan was prepared by another consulting firm. The primary facilities included three detention ponds, storm sewers, grass-lined channels and a sand/gravel filtration bed below the lower pond (Pond No. 3) in compliance with standard recommendations. The volume for water quality treatment was provided in Pond No. 3. Figure 4 illustrates the basin boundaries and a sketch map of the storm runoff control facilities for the development proposed in the drainage plan. The key facilities are described below:

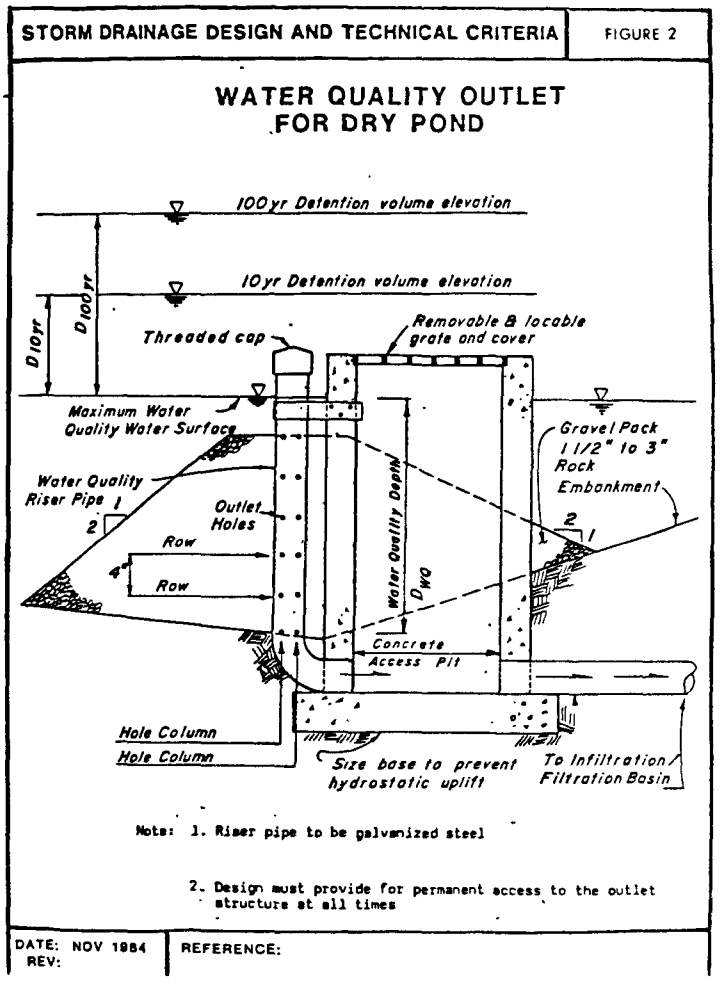
(1) A storm sewer and grass-lined channel conveyance system which directs the majority of runoff from more common events to Pond 3. Pond 2 is bypassed by this conveyance system except during larger storm events.

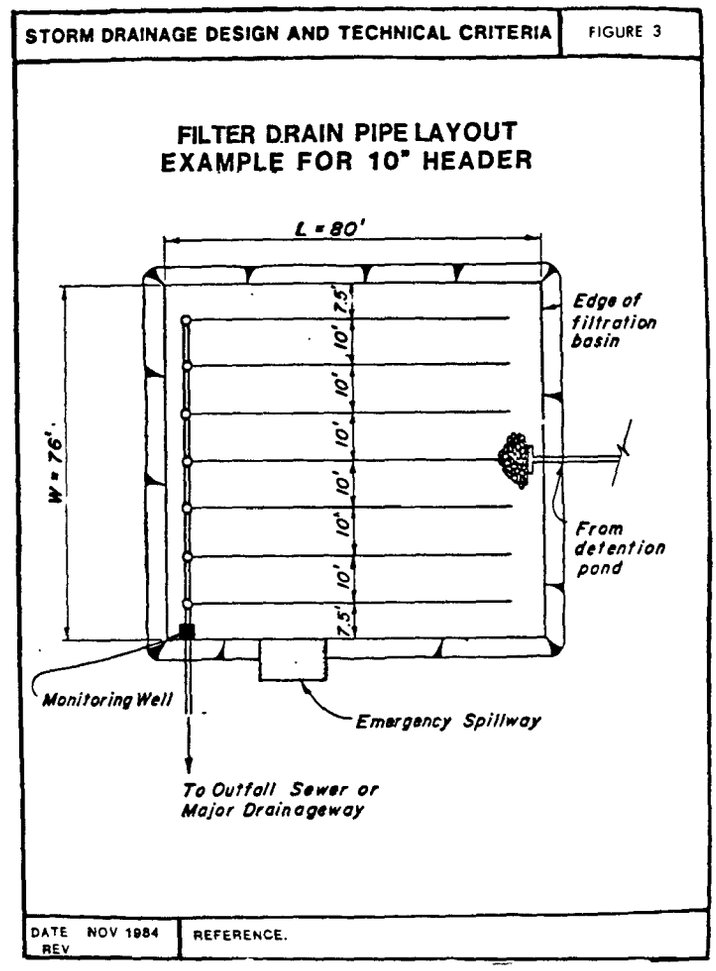
(2) Provision of 6.2 acre-feet of water quality storage at Pond 3 which is one-half inch of runoff from the impervious surfaces.

(3) An outlet structure for Pond 3 which incorporates coarse removal of debris and sediment.

(4) A filtration basin, which is to be constructed according to Parker and Douglas County Guidelines with imported sand and gravel filtering medium and perforated PVC pipe drains.

The primary intent of the treatment process is to remove phosphorous. However there is little performance history of such non-point phosphorus removal facilities. It is perceived that there will be significant problems with maintenance because of





sediment accumulation on the infiltration bed and the effectiveness for phosphorous removal. Specifically the removal of dissolved phosphorous in a sand media filter was questioned. Therefore, the water quality management alternative studied here was oriented toward retention, sedimentation and subsequent treatment utilizing irrigation. The effort specifically addressed on-site soils characteristics and open spaces plans conducive to such a treatment scheme. A key reason for utilization of this system is phosphorous removal efficiency. Basically irrigation such as this would be a land treatment system capable of 97% to 99% removal efficiency (Ref. 4).

ON-SITE SOIL INVESTIGATION

Most of the soils in the area of Pond No. 2 are classified as Newlin gravelly sandy loam or the Newlin-Santanta Complex by the Soil Conservation Service. The Newlin gravelly sandy loam has reasonably high, hydraulic conductivity characteristics with permeability of 0.63 to 2.0 inches per hour at the upper layer and 6.3 to 20 inches per hour at the lower layer.

The permeability of Santanta loam varies from 0.63 to 2.0 inches per hour for both layers.

Table 3: Residential Development: Mitigation Measure Effectiveness (Ref. 3)

Inches Treated	Total Phosphorus Reduction Factor		
	Retention	Filtration	Infiltration
0.00	0.000	0.000	0.000
0.02	0.012	0.024	0.044
0.04	0.026	0.052	0.094
0.06	0.040	0.080	0.145
0.08	0.053	0.108	0.194
0.10	0.065	0.130	0.234
0.15	0.089	0.178	0.253
0.20	0.112	0.223	0.401
0.25	0.132	0.263	0.473
0.30	0.153	0.307	0.551
0.35	0.169	0.338	0.607
0.40	0.182	0.364	0.654
0.45	0.194	0.389	0.700
0.50	0.207	0.415	0.746
0.55	0.213	0.426	0.766
0.60	0.219	0.438	0.787
0.65	0.224	0.449	0.807
0.70	0.230	0.411	0.828
0.75	0.239	0.478	0.859
0.80 or more	0.248	0.496	0.893

Note: Intermediate values may be determined by linear interpolation

A subsurface drainage system under Pond 2 could be expected to accept from 6 to 12 inches of water depth for the first day. For the second day, the intake rate may decrease by half and again by half the third day.

Soils in the area of Pond 3 and downstream of Pond 2 consist of Sampson loam. The permeability of the upper layer varies from 0.2 to 0.63 inches per hour according to the Soil Conservation Service (Ref. 5). Obviously, these soils are much slower draining than the Newlin, gravelly, sandy loams or even the Santanta loam complex.

Because of the excellent subsurface conditions for Pond 2, it was first considered that water quality volume and treatment by infiltrating be provided in Pond 2. However, it was subsequently learned that a soccer field was planned, which would dictate dry conditions soon after a rain storm. Also, it was desirable to keep sediment accumulations to a minimum in order to maintain infiltration rates. Therefore, it appears more practical to direct the majority of site runoff to Pond 3. Thus, only surplus runoff during major events will be spilled into Pond 2 directly.

The Pond 2 area characteristics for irrigation and infiltration treatment are good, and thus the study indicated its use as a primary treatment area.

The area that requires irrigation in the development was estimated at 17.4 acres. The 6.2 acre-feet of water quality volume stored can provide a 2 days supply for normal irrigation in this area. This irrigation water in area of Pond 2 could be collected by the underdrains that would drain directly into Cherry Creek. Most

Table 4: Grassland/Open Space: Mitigation Measure Effectiveness (Ref. 3)

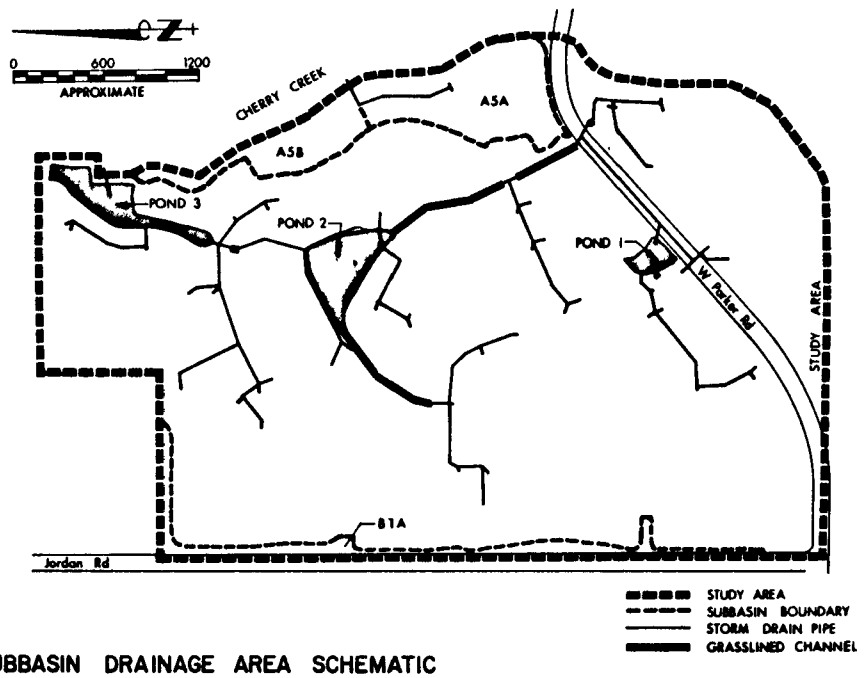
Inches Treated	Total Phosphorus Reduction Factor		
	Retention	Filtration	Infiltration
0.00	0.000	0.000	0.000
0.02	0.037	0.078	0.133
0.04	0.155	0.316	0.555
0.06 or more	0.227	0.457	0.806

Note: Intermediate values may be determined by linear interpolation

Table 5: Commercial Development: Mitigation Measure Effectiveness (Ref. 3)

Inches Treated	Total Phosphorus Reduction Factor		
	Retention	Filtration	Infiltration
0.00	0.000	0.000	0.000
0.02	0.039	0.076	0.138
0.04	0.065	0.130	0.234
0.06	0.076	0.152	0.274
0.08	0.087	0.174	0.313
0.10	0.098	0.196	0.352
0.15	0.114	0.227	0.410
0.20	0.127	0.254	0.458
0.25	0.139	0.277	0.499
0.30	0.149	0.297	0.535
0.35	0.158	0.315	0.568
0.40	0.164	0.329	0.592
0.45	0.171	0.342	0.615
0.50	0.177	0.354	0.638
0.55	0.181	0.363	0.653
0.60	0.185	0.371	0.667
0.65	0.191	0.381	0.685
0.70	0.196	0.390	0.702
0.75	0.199	0.397	0.713
0.80	0.202	0.403	0.725
0.90	0.209	0.410	0.750
0.95	0.212	0.417	0.763
1.00	0.218	0.431	0.776
1.20	0.225	0.450	0.809
1.40	0.237	0.465	0.836
1.60	0.238	0.476	0.856
1.80	0.241	0.482	0.868
2.00 or more	0.244	0.489	0.880

Note: Intermediate values may be determined by linear interpolation



SUBBASIN DRAINAGE AREA SCHEMATIC

McLaughlin Water Engineers, Ltd. FIGURE 4

of the phosphorous and much of the nitrogen will be removed by the soil profile. Plant uptake should be able to remove a high percentage of each element.

A subsurface drainage system under Pond No. 3 would drain water from the pond slowly, probably too slowly to add much to the volume of irrigation water. However, it would be desirable to be able to dry up the pond and the soil profile under the pond in between rainfalls. This would allow machinery to work in the pond to scrape off the sediment and also to cut grass or other vegetation.

CONCEPTUAL ALTERNATIVE SYSTEM FOR RUNOFF WATER TREATMENT

Based on the soils analyses and review of the proposed drainage facilities in Master Drainage Planning, an alternative runoff water quality management system was developed. As depicted in Figure 4, all frequent runoff events will find their way to Pond 3, except sub-basins A5A, A5B, and B1A. Sub-basins A5A and A5B (23.9 acres, or 7%), would usually discharge to the overbank flood plain meadow along Cherry Creek. This is similar to wetland application because this area is to be dedicated as an open space park, runoff will filter through the vegetation and soils there and thus be treated. Sub-basin B1A (11.1 acres, or 3.0% of the developable land) is basically Jordan Road and drains to a drainageway outside the development boundary.

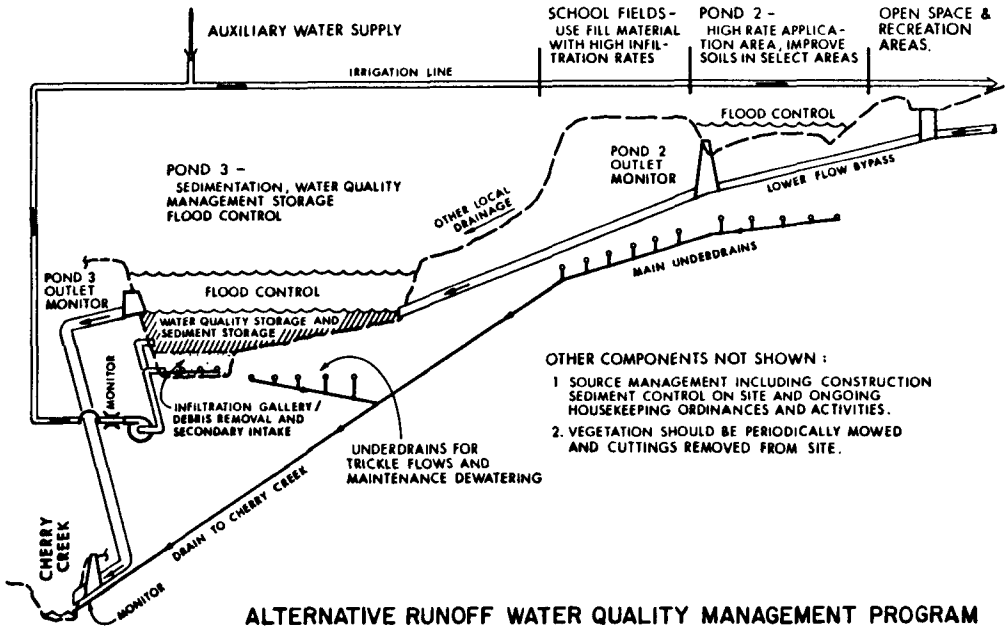
Runoff from the remaining 323.5 acres (90%) of the development will find its way to Pond 3. Based on the imperviousness of 40% for the development 6.2 acre-feet of water quality volume is required to capture 0.5 inches of runoff, which is

often regarded as an event that flushes approximately 90% - 95% of the available pollutants to the water course. In the concept proposed in Figure 5, some of this water would slowly infiltrate through underdrains to Cherry Creek, although this water could potentially be pumped to irrigation. This also helps to address water quality during the winter.

During the normal irrigation season, the majority of the water quality volume in Pond 3 will be pumped to the irrigation for the site, especially the high infiltration area around Pond 2. Auxiliary water supply would be needed during drier periods.

The irrigation system and pump station would necessitate a level of initial filtration/screening/operation at Pond 3 that would be more involved than the water quality outlet originally designated, but within reason. Pond 3 can be managed in several different modes. The pond will be drawn down by the irrigation pump station, and dewatered by the underdrains. Thus, the pond bottom could be cleaned of debris, particularly at the intake facility, and periodically disked to enhance plant growth and infiltration. Once every 2 to 10 years, depending on development and the success level of erosion-control practices, sediment accumulated in Pond 3 will have to be removed.

With major sediment control taking place in Pond 3 and the pump intake system, sediment application to the irrigated area should be minimized. It is anticipated that frequent soil aeration will be necessary, and at worst, portions of the sod in Pond 3 might have to be replaced (once every 10 to 30 years) if high infiltration rates are to be sustained.



ALTERNATIVE RUNOFF WATER QUALITY MANAGEMENT PROGRAM

McLAUGHLIN WATER ENGINEERS, LTD. — FIGURE 5

ESTIMATED SEDIMENT LOADS

Potential sediment loads during construction periods were estimated according to sampling results for Cherry Creek (Ref. 2). A potential erosion rate of 156 tons/Ac-yr. was assumed. This converts to settleable solids of 1,750 $ft.^3$/Ac-yr. Erosion control measures would reduce this rate to 430 $ft.^3$/Ac-yr. For this sediment loading rate, roughly 5,000 cubic yards per year may accumulate.

Sediment loads for developed conditions were estimated according to the study for the Cherry Creek Dam tributary area (Ref. 3) and total suspended solids estimates from DRCOG (Ref. 3). An annual sediment loading volume of 160 cubic yards/year was indicated. Assuming that all this sediment arrives at Pond 3, and that it would be practical to remove sediment when 3 inches of sediment had accumulated; a sediment storage zone of 1,500 cubic yards (approximately 1 acre foot) was called for. This data would indicate frequent sediment removal during early years of construction.

EFFECTIVENESS OF RUNOFF WATER QUALITY MANAGEMENT PROGRAM

Using the evaluation criteria of the Town of Parker, the effectiveness of this proposal is estimated at 61% which satisfies 50% (46% for this mix of development) phosphorus removal standard adopted by CWQCC in Cherry Creek basin.

Monitoring of the system during normal events should be fairly straightforward as the discharge from the underdrain system provides an absolute control point for flow and quality. The pump station provides a logical point for determining volumes and quality of water to be applied by irrigation. The overflow outlets out of each pond provide points for determining quantities of water receiving lesser treatment.

ADDITIONAL SYSTEM NEEDS

The proposed system has many components and operational needs that would be required with other on-site facilities. The discussion below highlights the additional needs or special coordination items that should be recognized and refined.

(1) Pump Station and Intake. This is a key facility to the success of the system. Although some delay can occur before application, irrigation should take place within a few days after the event. The controls would have to be tied to the irrigation controllers so that during key situations application could take place.

(2) Irrigation System. A conventional irrigation system may be utilized, except the hydraulics of the auxiliary source will have to be coordinated to make up the difference when the pump station is only providing a portion of the needs. This is not especially difficult, but needs recognition.

(3) Underdrain System. The underdrain system would use conventional agricultural practices, which are fairly economical as they are installed with trenchers and automatic pipe laying systems. The collection pipe to be routed to Cherry Creek will also be desirable to allow monitoring and prevent salt buildup.

CONCLUSIONS

This project investigated an alternative phosphorus control approach which utilized detention storage, sedimentation and irrigation of open space areas. Although three detention storages are called for only one provides water quality volume to store the sediment and the initial flush of storm runoff.

Phosphorous removal by irrigation and land treatment can be 97 to 99% effective (Ref. 4), and thus can have a higher removal efficiency than Infiltration cited in the Parker Manual (Ref. 3).

Irrigation will take place in most development and thus an opportunity exists to make efficient use of on-site soil infiltration as a preferred method and take advantage of facilities that are usually planned. It is percieved that the effectiveness of sand filtration beds as a phosphorous removal system is over-estimated in the Parker Manual (3) because dissolved phosphorous will be carried directly through, and because there will be excessive operational and maintenance costs.

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EVALUATION OF SEDIMENT EROSION AND POLLUTANT ASSOCIATIONS FOR URBAN AREAS

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ABSTRACT

The algorithms for erosion of pervious land in the USEPA stormwater management model (SWMM) are examined. Field equipment for measurement of simulated rain erosion is described. Field experiments and results are summarized. The relationship between eroded solids and rainfall energy is evaluated for different land types. Associations between fractionated eroded solids and metal concentrations are examined. Pollutographs show a relationship between eroded solids and selected metals concentrations.

INTRODUCTION

The general term 'pervious land' includes all erodible areas such as gravelled parking lots, bare industrial land, railway land, cemetaries, golf courses, parks, ravines and lawns. Clearly, these different pervious land types will have different erosion response rates to rainfall energy and different potential for pollutant (organics and trace metal) input to stormwater runoff. Urban stormwater runoff quality models typically do not consider the dynamics of erosional processes, if erosional processes are considered at all. The USEPA Stormwater Management Model (SWMM), for example, uses the Universal Soil Loss Equation (USLE) to simulate erosion for small time intervals (1-5 minute steps) . In this study the PC version of SWMM (James, 1985 (1)) is being used. The USLE was originally developed from data obtained from rural experimental soil plots in 21 states in the U.S.A. with the intention of simulating average annual soil loss in agricultrual areas (Smith and Wischmeier, 1962 (2)). The time frame to which the USLE is applied in SWMM, and the agricultural origin of some of the parameters, now transposed to an urban environment, probably constitutes a misapplication of the USLE.

Several researchers (eg. Ammon, 1979 (3)); Malmquist, 1983 (4)) have suggested that pollutant and particulate output from pervious land may be significant factors contributing to poor quality stormwater runoff. However, a literature review to date has revealed only one report (Pitt, 1985 (5)) in which pollutant outputs are quantified for pervious land other than construction sites. The purpose of this paper is to present some preliminary results from our ongoing study on the role of pervious land in stormwater runoff quality in the city of Hamilton, Ontario, Canada.

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SEDIMENT EROSION

It has been found that fine particulates (less than 4 mm in diameter, and often referred to as dust and dirt (DD) can significantly pollute stormwater runoff because: a) water turbidity is affected; and b) trace metals are often adsorbed at the particulate surface (Ammon, 1979 (3); Manning et al., 1977 (6); Ongley et al., 1981 (7) ; Pitt, 1985 (5); Sartor and Boyd, 1972 (8)). Although the toxicity of a given trace metal is species dependent (Benes et al., 1985 (9); Crecelius et al., 1982 (10); Florence, 1977 (11); Simkiss and Mason, 1984 (12)) it is time consuming and difficult to identify labile species in water/sediment samples (Chau et al., 1983 (13); 1984 (14) ; Ricci et al., 1981 (15)). Furthermore, the physical, chemical and biological processes that interact to produce the species distribution within a sample are very complex and difficult to model in any detail (Chapman, 1982 (16)). It is possible to model particulate (DD) movement within the urban system and to derive relationships between DD and selected pollutants such as phosphorus and various trace metals. Toxicity cannot be directly evaluated, but by monitoring and modelling DD buildup, redistribution and washoff processes, pollutant sources may be identified and abatement programs can be instituted and evaluated.

A mass balance approach is used to assess particulate input, storage and output for pervious/bare land, Components of the mass balance include inputs from rainfall, dry dustfall, human activities, vegetation, wind redistribution from impervious land, and outputs through water and wind erosion and biological activity. The major processes include: a) stormwater erosion output; and b) rainfall, dry dustfall and human activity input. Sampling and modelling procedures therefore concentrate on these processes.

There are four essential processes to be considered when evaluating water erosion from pervious land: a) particle detatchment by rainsplash; b) particle detatchment by overland flow; c) rill erosion/development; d) transport capacity of particles through rill and interrill areas. Each component of the erosion process should be evaluated in simple quantitative and descriptive terms to provide a fundamental understanding of particulate output from pervious land. Different types of pervious/bare land, such as lawns, golf courses, cemetaries, railway yards, gravelled parking lots and industrial yards will have different responses to similar rainfall inputs. A classification system based on process response should therefore be developed to facilitate system modelling.

Soil erosion by water in agricultural areas has basically been modelled by one of four methods: a) Universal Soil Loss Equation (USLE) or modified Universal Soil Loss Equation (MUSLE) (Smith and Wischmeier, 1962 (1); Wischmeier and Smith, 1978 (17); Williams, 1975 (18)); b) conceptual consideration for the physical processes involved (Donigian and Crawford, 1976 (19); Li et al., 1977 (20); Simons et al., 1977 (21)); c) combination of the USLE, interrill and rill erosion and routing processes (Foster et al., 1980 (22); Khanbilvardi and Rogowski, 1984 (23)); d) combination of deterministic and probabilistic techniques (Moore, 1984 (24); Rojiani et al., 1984 (25)). There are numerous problems with applying the USLE or MUSLE directly to an urban catchment on an event basis, including: a) the inability to model detachment and transport separately; b) no consideration of transported grain sizes; c) variables such as the cropping factor were developed for agricultural areas and are not meaningful in an urban environment. The (M)USLE is easy to use however, and some of the parameters in other, more complex models may be difficult to obtain for urban land. The Storm Water Management Model (SWMM) is used extensively in design and water quality studies, but the water quality subroutines clearly need improvement (Boregowda, 1984 (26); Cermola et al., 1979 (27); MacRae, 1979 (28)). The USLE is currently employed in SWMM to simulate erosion.

The USLE in SWMM does not provide any estimate of the grain size distribution of the eroded sediment. In our study, eroded particle sizes are being analysed to determine which sizes

are most important in pollutant transport and how eroded material from pervious land might affect total DD transport in the urban system. Such information would be particularly useful in the the development of nonstructural pollution abatement programs (eg. street sweeping, sewer flushing) and in the optimization of settling/holding tank hydraulic conditions (Klemetson, 1985 (29); Randall, 1982 (30)).

There are at least three methods by which eroded grain sizes can be predicted: a) empirical and quasi-conceptual deterministic models (Li et al., 1977 (20); Williams, 1980 (31)); b) regression models that consider soil characteristics (Frere et al., 1975 (32); Young and Onstad, 1976 (33)); c) laboratory disaggregation techniques (Meyer et al., 1983 (34)). Approaches (a) and (b) are most appropriate for the USLE type of erosion model . Using the predicted grain size distribution, the transport capacity for each size class could be calculated by Yalin's equation, for example.

PREVIOUS RESEARCH

It has long been known that construction and general urbanization can have a significant impact on the sediment regime of a catchment (Armstrong, 1978 (35); Burton et al., 1976 (36); Diseker and Richardson, 1962 (37); Keller, 1962 (38); Walling and Gregory, 1970 (39)). Sediment yields from construction sites may be many times greater than from nearby agricultural land and in general, progressive urbanization causes an initial rise in total sediment, which ultimately is followed by a decline as pervious land is paved (Wolman and Schick, 1967 (40)). During the 1960's and early 70's, research concerns in urban sedimentology were directed primarily at environmental effects of the eroded sediment load itself, (for example, changes in channel hydraulic geometry) rather than with respect to the various pollutants transported by the particulates (eg. Guy, 1967 (41)). However, with increasing awareness that urban runoff was a significant contributor to receiving body degradation, research programs such as the Nationwide Urban Runoff Program (NURP) in the U.S., were initiated in the late 1970's (Cole et al., 1984 (42)).

In the review of research needs on urban stormwater pollution, Heaney (1986) (43) suggested that the influence of soils, land use and season should be investigated. As mentioned above, many researchers have suggested that pollutant output from pervious land may be important, but our literature review to date revealed only one paper (Pitt, 1985 (5)) in which outputs are quantified. Pitt found that for two residential areas in Bellevue, Washington, front and back yards supplied approximately 83% of the total solids, 25% of COD, 42% of phosphates, 39% of TKN, 2% of Pb and 4% of Zn loads for 2.5-65 mm rain events. Although the total solids in this case may be relatively 'clean' of trace metals at source, they can provide a transport medium for metals absorbed during movement over impervious areas.

Many of the field and modelling techniques, as well as a general description of water erosion processes can be borrowed from the extensive work done in agricultural/rural areas. Empirical equations to predict soil loss, such as those of Musgrave or Browning and his coworkers, began to appear in the 1940's (Smith and Wischmeier, 1962 (1)). Wischmeier and Smith (1958) (44) and Wischmeier (1959) (45) devised a rainfall erosion index for general use in the United States based on the research of Laws and Parsons (1943) (46), and by 1960 the USLE had been developed. The USLE was orginally based on 8000 plot years of basic hydrometeorologic and soil loss data from experimental soil plots in 21 states. Use of the USLE and modifications to individual parameters have been discussed by Mitchell and Bubenzer (1980) (47); Smith and Wischmeier (1962) (1); and Wischmeier and Smith (1978) (17).

A great deal of research has been done on characterizing and quantifying rainsplash detachment and transport, overland flow detachment and transport and rill development (eg. Al-Durrah and Bradford, 1982 (48); Bryan, 1976 (49); 1979 (50); Emmett, 1970 (51); Evans, 1980 (52); Luk, 1979 (53); Luk and Hamilton, 1986 (54); Morris, 1986 (55); Poesen and Savat, 1981

(56)). However, Morris (1986 (55)) has pointed out several difficulties in isolating and rigorously quanitfying the individual components of the erosion process, and Kirkby (1980) (57) suggests that: a) sediment yields from rainsplash are low; and b) interrill mechanisms are so complex that at present largely empirical models for these processes are sufficient. Thus, while it is not profitable in the present context of research to model the individual erosional processes in much detail, it is useful to consider detachment and transport separately, on a storm basis. Such an approach overcomes several of the criticisms made of the USLE (Foster et al., 1980 (22); Khanbilvardi and Rogowski, 1984 (23); Kirkby, 1980 (57)) and facilitates eroded grain size distribution modelling (Foster et al., 1980 (22); Williams, 1980 (31)).

A mass balance approach has recently been used to model DD processes on impervious land surfaces with some success (Boregowda, 1984 (26); James and Boregowda, 1985 a. (58) b. (59); Novotony et al., 1985 (60)) and there would be some common processes for both pervious and impervious land. Hamilton and Chatt (1982) (61) and Tanaka et al. (1981) (62) have successfully determined particulate metal concentrations directly from filters, and have indicated that precipitation particulates may form an important part of pollutant load input to the surface. Slinn (1977) (63) has developed relationships to predict particulate scavaging during rainfall events. Dry dustfall has been examined by numerous researchers (eg. Jeffries and Snyder, 1981 (64); Malmquist, 1983 (4): Ontario Research Foundation et al., 1982 (65)) and it is now generally accepted that dustfall can be a significant component in urban pollution. However, the relationships between pervious land and dry dustfall have yet to be determined. Various empirical relationships have been developed for population and vegetation inputs to impervious land (Boregowda, 1984 (26); Prasad et al., 1980 (66)) and these are potentially applicable to pervious land, although this will have to be investigated. Erosion and transport by wind has been given similar theoretical and practical treatment as erosion and transport by water. Wind velocity profiles in fully turbulent conditions have been described by the Prandtl and von Karman equation, while Bagnold (1941 (67)) defined threshold shear velocity in terms of grain density, air density, the gravitational constant and grain diameter. He also related the rate of sand flow per unit width to wind shear velocity, grain diameter and air density. Numerous empirical equations have been developed relating erosion rates directly to wind velocity, and in 1965 Woodruff and Siddoway developed a wind erosion equation with a form similar to the USLE. Wilson and Cooke (1980) (68) note that wind erosion can be highly localized and de Ploey and Gabriels (1980) (69) examined the difficulties of measuring wind erosion. At the present time it may be worthwhile to consider wind erosion in the simplest terms for an urban area.

STUDY CATCHMENT AND DATA

The Chedoke study catchment in Hamilton is 26.8 km^2 in area of which 82.5% is pervious (Boregowda, 1984 (26)). Land use is predominantly low to medium density, single family residential, although institutional (eg. McMaster University and Medical Centre), commercial and light industrial uses are also present.

Runoff was sampled at 7 sites between May and November, 1986: 1. light industrial gravelled receiving area; 2. sewer inlet draining the paved road and lawns adjacent to site 1; 3. light industrial bare gravelled storage lot; 4. side of a railway track embankment; 5. small grassed plot at sewer overflow; 6. playing field; 7. two sites in a ravine receiving combined sewer overflow: site (a) was at the sewer outfall and site (b) was downstream in the ravine. Runoff at the sites was generated either by natural rainfall or by the rainfall simulator described below. Simulated rain closely resembled natural rainfall characteristics.

Rainfall intensity data for 1 minute intervals for the natural events was obtained using a Drop Counter Precipitation Sensor (DCPS) system installed on the roof of the Engineering building at McMaster. The DCPS system, developed at McMaster (James and Stirrup, 1986 (70)), provided reliable intensity data during the study period, the average error between

observed and calculated (from the intensity at each time step) total storm volumes being 10%. The greatest errors were recorded for rainfalls of less than 2 mm.

Total trace metal concentrations were determined by Instrumental Neutron Activation Analysis (INAA) at the McMaster Nuclear Reactor under the direction of Dr. S. Landsberger. Sample collection, storage and handling procedures were designed to limit sample contamination.

RAINFALL SIMULATOR

Rainfall simulators are often used for studying erosional processes in agricultural soils (eg. Bryan, 1970 (71); Bryan and de Ploey, 1983 (72); Bubenzer and Jones, 1971 (73); Imeson, 1977 (74); Luk, 1979 (53); Luk and Hamilton, 1986 (54)), but they have also been used for such diversified research as Karst landform development (Glew, 1976(75)), pollutant washoff on city streets (Sartor and Boyd, 1972 (8)) and assessment of linear and initial storage theories to describe the relationship between rainfall and runoff characteristics (Johanson, 1967 (76)).

Many of the simulators described in the aforementioned studies are strictly for use in a laboratory setting. Reproduction of some pervious land types encountered in an urban area, such as gravelled parking lots, bare industrial land, and railway land, would be difficult. Therefore, a rainfall simulator was needed that could be easily transported for in situ simulations, or for laboratory studies, if desired. Rainfall simulation is being used in this study to augment data obtained from natural rainfall, since a large amount of data can be collected through simulation at a time and place of the researcher's choosing, under carefully controlled conditions.

The four factors that were of greatest concern in the development and uses of the simulator were: a) ease of installation and simplicity of design, due to time and financial constraints; b) ability to simulate a range of rainfall intensities; c) even distribution of water over the sample plot; and d) reasonable imitation of the size distribution and fall velocity of naturally occurring raindrops. The simulator was developed and modified from a design proposed by Dr. S. Luk for the Geography Department at the University of Toronto. The total costs of the 2-stand simulators is $355 (Canadian dollars in 1986).

The 2-stand version can be assembled by 2 people in about 30 minutes, depending on the slope of the land. More adjustment is generally required to ensure that both nozzles are at equal elevation and that the upright galvanized steel pipe is vertical when the slope is greater than about 15%. The test plot can be expanded by adding more simulator pairs, although it may be necessary to connect a second water pressure regulator (one for each side of the test plot) if more simulator pairs are added.

The maximum test plot size to ensure an evenly distributed rain is 2 m x 2 m ($4m^2$ or 10.8 ft^2) and plots should be defined by garden edging or a wood frame, depending on the surface type. Test plots in this study were typically smaller than 4 m^2 (3.5-3.9 m^2) because the downslope plot edging is angled into a collector trough.

One stand is placed on either side of the test plot and the 1.9 cm (3/4") upright pipe should be 1 m (3'3") from the top and bottom of the plot and 0.5 m (1'8") back from the plot edge. This positioning ensures that the conical spray pattern of the individual spray nozzles overlaps on the plot to produce an evenly distributed rain.

Gerlach-type overland flow troughs were used in the simulations to collect runoff and eroded sediment at the downslope end of the plot. These troughs are constructed of PVC tubing to limit sample contamination. The troughs are 0.66 m (2'2") in length (the dimension of a typical sewer grate) and have one lip (3.5 cm or 9" wide) which projects into the soil, and an optional second lip which helps to: a) limit rainfall input to the trough; and b) catch splash-eroded particulates. In the absence of the second lip, the open top of the trough should be covered with plastic to limit rainfall input. Overland flow from the plot is routed down the trough, through a funnel, into a Nalgene collection bottle. Overland flow may be collected at a sewer inlet if the surface does not permit installation of a flow trough.

Rainfall intensities range from 52 to 87 mm hr^{-1}. The average variability of intensity between individual rain gages during a single event (as measured by the standard deviation of intensity for the gages) is about 5 mm hr^{-1}. The range of standard deviations of intensity for individual runs is 0.97 to 10.5 mm hr^{-1}. Higher standard deviations were recorded when wind velocity was high. The standard deviation of intensity is typically 2-3 mm hr^{-1} provided that calm winds prevail.

Simulated rainfall intensity is related to water pressure, which can be adjusted by turning a screw on top of the pressure regulator. Tests indicate that water pressure must be changed by 3.45-5.17 Kpa (1/2-3/4 psi) for a measureable, consistent change in rainfall intensity. Rainfall intensity decreases with increasing water pressure, being approximately 71 mm hr^{-1} at 62.06 Kpa (9psi), 62 mm hr^{-1} at 65.5-68.95 Kpa (9.5-10 psi) and 56 mm hr^{-1} at 75.84 Kpa (11 psi). Using a similar rainfall simulation system, S. Luk (pers. comm.) found that a pressure of 67.23 Kpa (9.75 psi) results in a rainfall intensity of 65 mm hr^{-1}, which suggests that results for the two systems are reproducible.

Water consumption averages approximately 1240 1 hr^{-1} (272 gal. hr^{-1}) and the difference in flow rate between the two uprights varies, but is normally around 6%.

The median drop diameter produced by the Spraco nozzle decreases in size from 810 um at 62.06 Kpa (9 psi) to 72- um at 103.42 Kpa (15 psi). Similarly, the sauter mean drop size (diameter of a droplet whose ratio of volume to surface area is equal to that of the entire spray sample) decreases from 648 um at 62.06 Kpa (9 psi) to 576 um at 103.42 Kpa (15 psi) (Spraco, 1985 (77)). These values appear to be slightly less than that produced by natural rainfall.

RESULTS

A total of 21 simulated and natural events were sampled. The simulated events were used strictly in the collection of erosion rate information. The greatest number of events (7) were sampled at site 4, a 3.26 m^2 plot with an 18% slope. Six of the seven events have a runoff and sediment record for the entire event and average sediment yield (gm m^2) for these six events was regressed against a rainfall erosivity factor (R):

$$R = EI_{30} \qquad [1]$$

I_{30} is the maximum 30 minute rainfall intensity (in hr^{-1}) for the event; E is the total kinetic energy of rainfall (ft tons ac^{-1}) for the event (from Wischmeier and Smith, 1958 (44)):

$$E = S[(916 + 331 \log Ij)\ I_j\ t_j] \qquad [2]$$

where I_j is the rainfall intensity (in hr^{-1}) for the given time interval, t_j (hours). All measurements in this study were done using the metric system and although [2] was derived from imperial units, for simplicity sediment yield, Y, (gm m^{-2}) was regressed against [1] without conversion. The regression equation for the 6 events is:

$$Y = 15.2 + 0.382(EI_{30}) \qquad [3]$$

Equation [3] explains 70% of the variance in the data (63% when adjusted for degrees of freedom) and the b_1 coefficient (0.382) is significantly different from zero (P=0.03). Other researchers (eg. Foster et al., 1982 (78)) have also had reasonable success in relating EI_{30} to sediment yield.

The effect of pervious land characteristics on erosion rates becomes obvious when different test site responses are compared for the same or similar rainfall input. For example, a

thunderstorm on July 17 (EI_{30}=175) resulted in an average sediment yield rate of 95.34 gm m^{-2} from site 4. The average sediment yeld from site 5, a 3.84 m^2 grassed plot with a 24% slope was 0.018 gm m^{-2} for the same event. In four rainfall simulations at site 5 (RI_{30}=1800-7000) average sediment yeld ranged between 0.01-0.054 gm m^{-2}. At site 3, a 3.52 m^2 gravelled plot with 1-2% slope, the average yield for 2 rainfall simulations (RI_{30}=3000) was 20-25 gm m^{-2}. Rainfall on October 27 (EI_{30}=83) resulted in an average sediment yeild of 0.216 gm m^{-2} from site 1 (area=390 m^2, slope=3.5%) and 0.009 gm m^{-2} at site 2 (area=5688 m^2, slope-0.6%). The effect of lawn cover and paving are obvious in this last example. Clearly, the different response rates for the different land types would result in different b_1 coefficients in equation [3] and these different response rates should be considered when modelling pollutant input from pervious land.

Various researchers (eg. Ackermann et al., 1983 (79); Forstner and Wittman, 1981 (80)) have found that metals and organic compounds are preferentially adsorbed and transported by finer particles. The sediment grain size distribution also affects overland and in-sewer transport dynamics. Sediment in the samples from most events was therefore separated into sand (>62 um), silt (5-62 um) and clay (<5 um) fractions by wet filtering. Sediment in the samples to be analysed for metals was not dispersed prior to filtration.

The average observed eroded sediment grain size distributions calculated from all samples for selected events at the various sites were calculated . As expected, there are obvious differences in the eroded grain size distributions from the various sites. Some of the sites (eg. 4 and 5) also exhibited considerable variability in grain size distribution with time. This variability is in part related to clay enrichment at the end of an event as transport capacity decreases. Clay content is also less than average at the beginning of most events, probably due to its cohesive nature and resistance to erosion. Other factors affecting grain size distribution variability are being investigated.

Concentration of most metals analysed increase progressively from the sand to clay fraction although some exceptions do occur, as Mn concentrations, for example, can occasionally be greater in sand than silt. Average observed concentrations of V associated with the different size fractions from selected events at site 4 were also calculated Ackermann et al. (1983) (79) suggested that only the <20 um fraction of sediment need be analysed. However, given the high proportion of coarse material in our samples, a great deal of the metals load can be carried by this fraction even though the concentrations are lower. Our field observations show that 65% fo the total V load is transported by sand, 30% by silt and 5% by clay. We suggest that all size fractions by analysed to maximise information about pollutant movement.

Higher metal concentrations are associated with sediment eroded from sites that are industrial in nature or near major transportation routes. In particular, the average Mn (1778 ppm) and V (142 ppm) concentrations on clay for selected events at the gravelled industrial receiving area (site 1) are greater than from the nearby lawns and roadway (Site 2) and also greater than typical background (ie. natural) levels (Lisk, 1972 (81)). Vermette et al (82) also found progressively higher Mn concentrations towards the steel mills in Hamilton when grab samples from different pervious land types were analysed. Average Mn and V concentrations at Site 6 (samples not fractionated) were 909 and 96 ppm respectively, which are closer to natural levels.

Finally, Mn and V concentrations assocated with the clay fraction were plotted with total solids (TS) against time for one event from site 1 and one event from site 4. It is typically assumed that metals concentrations, being conservative, can be related to TS concentrations and TS data can be more easily and cheaply obtained (James, 1985 (1). The available data showed that in general the metals concentration time series obtained from the clay fraction most resembled the solids time series. The Mn and V concentrations appear to have some relationship with the TS concentration (and also with clay concentration which is not plotted but which exhibited a pattern similar to the TS concentration) although there may be a lag in response and some deviations from this general relationship due to variable source areas and to variable atmospheric input. It should also be noted that not all metals analysed (eg. Ca, Cl) had a similar close relationship with TS.

CONCLUSIONS

1. The rainfall simulator is simple, portable and easy to erect and may be used for simulation in both a laboratory setting and in-situ.
2. Test runs with the simulator indicate that the variation of simulated rainfall intensity within the test plot is small, typically 2-3 mm hr^{-1} under calm conditions.
3. Variation in simulated rainfall intensity during a single run, and between successive runs using the same water pressure is due to: a) effect of winds; b) fluctuations in water supply that are not totally controlled by the pressure regulator; c) unequal elevation of the spray nozzles; and d) catch errors in the rain gauges.
4. Generally, higher water pressures produce lower simulated intensity rainfalls and the different intensities are reproducible at adjacent sites.
5. The rainfall simulator can be used on plots with slopes of at least 24.5%.
6. In many areas of a city, erosion from pervious land may provide significant inputs of particulates, metals and organic compounds (see also Pitt, 1985 (5)). The current practice of assuming an exponential decay in pollutant load washoff through an event may be one reason that results are poor for urban stormwater quality models. The results show that metal and total solid concentrations from pervious land have multiple peaks throughout an event.
7. Although the USLE may not be suited for short time step simulations in an urban environment, it appears that a rainfall erosivity factor such as the EI_{30} of the USLE may be useful in determining the amount of sediment detatched from pervious land. We suggest that the erosivity factor be linked to some type of dynamic transport capacity model as is done in CREAMS. Work in this area is being carried out.
8. The importance of evaluating eroded grain size distribution is apparent when examining metal concentrations and potential pollutant transport. The highest metal concentrations were assocated with the clay fraction of the sediment. However, clay typically made up a small proportion of the eroded grain size distribution from most sites and a greater proportion of the total metal load could be transported by the coarse fraction. There appears to be a good relationship between concentrations for certain metals (eg. Mn, V) and the total solids and clay concentrations, and further investigation into these relationships will be helpful in pollutant transport modelling.

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UNCERTAINTY IN HYDROLOGIC MODELS:

A REVIEW OF THE LITERATURE

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ABSTRACT

Advances in hydrologic modeling techniques typically involves the incorporation of higher complexity into the hydrology model by use of improved hydraulic submodels or a more refined approximation of the several subprocesses integrated in the hydrologic cycle. With over 100 models reported in the open literature, it is appropriate to review the progress achieved by the complexification of hydrologic models. That is, it is time to evaluate whether the general level of success afforded by the many types of complex models provide a marked improvement over that achieved by the more commonly used and simpler models such as the unit hydrograph method. Such a review indicates that it is still not clear, in general, whether as modeling complexity increases, modeling accuracy increases. It appears that a major limitation to the successful development, calibration, and application, of any hydrologic model is the uncertainty of the effective rainfall distribution over the catchment.

INTRODUCTION

A review of the literature indicates that a substantial evolution in modeling complexity has occurred over the last two decades. The majority of changes have occurred in the incorporation of soil moisture accounting techniques and intricate link-node model discretization using approximations for hydraulics. However in spite of the advances made in the modeling complexity, the accuracy of models (in general) has not been significantly improved in the correlation of rain gage data to stream gage runoff data. Only a handful of papers and reports are available in the open literature which compare modeling performance, and each of these reports note that simpler models do as good as or better than complex models. Additionally, many of the papers indicate that the uncertainty in the effective rainfall distribution over the catchment may be a key factor in the lack of major gains in the development, calibration, and application, of hydrologic models. As a result of this lack in demonstrated success in the use of any particular advanced modeling technique or approach, there is continued reliance by the

engineering community to use the more simpler modeling approaches such as the rational method for peak flow rate estimates, or the classic unit hydrograph method when a runoff hydrograph is needed.

In this paper, the literature is reviewed to identify trends which support using simpler models such as the unit hydrograph method (more specifically, the design storm/unit hydrograph approach). From these trends, suggestions as to how the more complex modeling techniques may "prove themselves to practitioners and, therefore, find more use in the engineering community, may be formulated.

MODEL SELECTION

In the selection of the hydrologic model, the need for both runoff peak flow rates and runoff volumes (for the testing of detention basins) require the selection of a model that produces a runoff hydrograph. The U.S. Army Corps of Engineers (COE) Hydrologic Engineering Center (HEC) Training Document (TD) No. 11, (1980) (1) categorizes all hydrologic models into eight groupings of which three develop a runoff hydrograph; namely, single event (design storm), multiple discrete events, and continuous records (continuous simulation). These models can be further classified according to the submodels employed. For example, a unit hydrograph or a kinematic wave model may be used to represent the catchment hydraulics.

In a survey of hydrologic model usage by Federal and State governmentaal agencies and private engineering firms (U.S. Department of Transportation, Federal Highway Administration, Hydraulic Engineering Circular No. 19, October 1984 (2), it was found that "practically no use is made of watershed models for discrete event and continuous hydrograph simulation." In comparison, however, design storm methods were used from 24 to 34 times more frequently than the complex models by Federal agencies and the private sector, respectively. The frequent use of design storm methods appear to be due to several reasons: (1) design storm methods are considerably simpler to use than discrete event and continuous simulation models; (2) it has not been established in general that the more complex models provide an improvement in computational accuracy over design storm models; and (3) the level of complexity typically embodied in the continuous simulation class of models does not appear to be appropriate for the catchment rainfall-runoff data which is typically available. Consequently, the design storm approach is most often selected for flood control and drainage policies (considerations in the choice of modeling approach are contained in the latter sections).

The next decision is whether to use the standard unit hydrograph method or the more recently advanced kinematic wave method to model catchment hydraulics. Again, it has not been clearly established that the kinematic wave approach (e.g., the overload flow place concept) provides an improvement in modeling accuracy over the unit hydrograph approach that has been calibrated to local rainfall-runoff data.

For the choice of design storm to be used, the work of Beard and Chang (1979) (3) and HEC ("Hypothetical Floods", 1975) (4) provide a logical motivation for developing a design storm using rainfalls of identical return frequency, adjusted for watershed area effects.

Finally, specific components of the modeling approach must be selected and specified. Inherent in the choice of submodels is the ability to calibrate the model at two levels: (1) calibration of model parameters to represent local or regional catchment rainfall-runoff characteristics, and (2) calibration of the model parameters (or design

storm) to represent local rainfall intensity-duration-frequency characteristics. Beard and Chang (1979) (3) note that in a hydrologic model, the number of calibration parameters should be as small as possible in order to correlate model parameters with basin characteristics. They also write that a regional study should be prepared to establish the loss rate and unit hydrograph characteristics, "and to compute from balanced storms of selected frequencies (storms having the same rainfall frequency for all durations) the resulting floods."

LITERATURE REVIEW

CHOICE OF WATERSHED MODEL

In developing a flood control and drainage policy, the first, and possibly the most important question to answer is: what type of model should be used to form the basis for design calculations? To answer this question, the literature was reviewed extensively. Based on the research findings summarized in the following paragraphs, the design storm/unit hydrograph (UH) method appears to have continued support among practitioners. The question naturally arises as to why the simple UH method continues to be the dominant hydrologic tool when considerably more complex models (e.g., the continuous simulation class of models which has a mathematical approximation for each component of the hydrologic cycle, and typically utilizes physically based hydraulic flow routing approximations. The Stanford Watershed Model is an excellent example of this class of approach.) are available for public use. As explanation frequently cited in the literature appears to be that the uncertainty in the effective rainfall over the catchment overshadows the improved accuracy that may be possibly achieved by more complex models.

A criterion for complex and simple models is given by Beard and Chang (1979) (3) as as the "difficulty or reliability of model calibration...Perhaps the simplest type of model that produces a flood hydrograph is the unit hydrograph model"...and..." can be derived to some extent from physical drainage features but fairly easily and farily reliably calibrated through successive approximations by relating the time distribution of average basin rainfall excess to the time distribution of runoff." In comparison, the "most complicated type of model is one that represents each significant element of the hydrologic process by a mathematical algorithm. This is represented by the Stanford Watershed Model and requires extensive data and effort to calibrate."

The literature contains several reports of problems in using complex models, especially in parameter optimizations. Additionally, it has not been clearly established whether complex models, such as in the continuous simulation or discrete event classes of models, provide and increase in accuracy over a standard design storm unit hydrograph model.

There are only a few papers and reports in the literature that provide a comparison in hydrologic model performance. From these references, it appears that a simple unit hydrograph model provides as good as or better results than quasi-physically based (or QPB, see the work of Loague and Freeze (1985)) (5) or complex models.

In their paper, Beard and Chang (1979) (3) write that in the case of the unit hydrograph model, "the function of runoff versus rainfall excess is considered to be linear, whereas it usually is not in nature. Also, the variations in shapes of unit hydrographs are not derivable directly from physical factors. However, models of this general nature are usually as representative of physical conditions as can reasonably be validated by

available data, and there is little advantage in extending the degree of model sophistication beyond validation capability." It is suggested that "if 50 yr-100 yr of streamflow were available for a specified condition of watershed development, a frequency curve flows for that condition can be constructed from a properly selected set of flows."

Schilling and Fuchs (1986) (6) write "that the spatial resolution of rain data input is of paramount importance to the accuracy of the simulated hydrograph" due to "the high spatial variability of storms" and "the amplification of rainfall sampling errors by the nonlinear transformation" of rainfall into runoff. Their recommendations are the model should employ a simplified surface flow model if there are many subbasins; a simple runoff coefficient loss rate; and a diffusion (zero inertia) or storage channel routing technique. Hornberger, et al. (1985) (7) writes that "Even the most physically based models...cannot reflect the true complexity and heterogeneity of the processes occurring in the field. Catchment hydrology is still very much an empirical science."

In attempting to define the modeling processes by the available field data forms Hornberger, et.al. find that "Hydrological quantities measured in the field tend to be either integral variables (e.g. stream discharge, which reflects an integrated catchment response) or point estimates of variables that are likely to exhibit marked spatial and/or temporal variation (e.g., soil hydraulic conductivity)." Hence, the precise definition of the physics in modeling sense becomes a problem that is "poorly posed in the mathematical sense." Typically, the submodel parameters cannot be estimated precisely due to the large associated estimation error. "Such difficulties often indicate that the structural complexity of the model is greater than is warranted on the basis of the calibration data set."

Schilling and Fuchs (1986) (6) note that errors in simulation occur for several reasons including:

"1. The input data, consisting of rainfall and antecedent conditions, vary throughout the watershed and cannot be precisely measured.
2. The physical laws of fluid motion are simplified.
3. Model parameter estimates may be in error."

By reducing the rainfall data set resolution from a grid of 81 gages to a single catchment-centered gage in and 1,800 acre catchment, variations in runoff volumes and peak flows "is well above 100 percent over the entire range of storms implying that the spatial resolution of rainfall has a dominant influence on the reliability of computed runoff." It is also noted that "errors in the rainfall input are amplified by the rainfall-runoff transformation" so that "a rainfall depth error of 30 percent results in a volume error of 60 percent and peak flow error of 80 percent."

Schilling and Fuchs (1986) (6) also write that "it is inappropriate to use a sophisticated runoff model to achieve a desired level of modeling accuracy if the spatial resolution of rain input is low" (in their study, the rainage densities considered for the 1,800-acre catchment are 81-, 9-, and a single centered gage).

In a similar vein, Beard and Chang (1979) (3) write that in their study of 14 urban catchments, complex models such as continuous simulation typically have 20 to 40 parameters and functions that must be derived from recorded rainfall-runoff data. "Inasmuch as rainfall data are for scattered point locations and storm rainfall is highly variable in time and space, available data are generally inadequate in this region for

reliably calibrating the various interrelated functions of these complex models." Additionally, "changes in the model that would result from urbanization could not be reliably determined." They write that the application "of these complex models to evaluating changes in flood frequencies usually requires simulation of about 50 years of streamflow at each location under each alternative watershed condition."

Garen and Burges (1981) (8) noted the difficulties in rainfall measurement for use in the Stanford Watershed Model, because the K1 parameter (rainfall adjustment factor) and UZSN parameter (upper level storage) had the dominant impact on the model sensitivity. This is especially noteworthy because Dawdy and O'Donnell (1965) (9) concluded that insensitive model coefficients could not be caibrated accurately. Hence, they could not be reliably used to measure physical effects of watershed changes.

Using another complex model, Mein and Brown (1978) (10) write that on "the basis of several tests with the Boughton model it is concluded that for this model at least, relationships derived between any given parameter value and measureable watershed characteristics would be imprecise; i.e., they would have wide confidence limits. One could not be confident therefore in changing a particular parameter value of this model and then claiming that this alteration represented the effect of some proposed land use change. On the other hand, the model performed quite well in predicting flows with these insensitive parametrs, showing that individual parameter precision is not a prerequisite to satisfying output performance."

According to Gburek (1971),"...a model system is merely a researcher's idea of how a physical system interacts and behaves, and in the case of watershed research, watershed models are usually extremely simplified mathematical descriptions of a complex physical situation...until each internal submodel of the overall model can be independently verified, the model remains strictly a hypothesis with respect to its internal locations and transformations..." (also quoted in McPherson and Schneider, (1974) (12)).

The introduction of a paper by Sorooshian and Gupta (1983) provides a brief review of some of the problems reported by other researchers in attempting to find a "true optimum" parametr set for complex models, including the unsuccessful two man-year effort by Johnston and Pilgrim (1973) (14) to optimize parameters for a version of the Boughton model cited above.

In the extensive study by Loague and Freeze (1985) (5), three event-based rainfall-runoff models (a regression model, a unit hydrograph model, and a kinematic wave quasi-physically based model) were used on three data sets of 269 events from three small upland catchments. In that paper, the term "quasi-physically based" or QPB is used for the kinematic wave model. The three catchments were 25 acres, 2.8 mi^2, and 35 acres in size, and were extensively monitored with rain gage, stream gage, neutron probe, and soil site testing.

For example, the 25 acre site contained 35 neutron probe access sites, 26 soil parameter sites (all equally spaced), an on-site rain gage, and a stream gage. The QPB model utilized 22 overland flow planes and four channel segments. In comparative tests between the three modeling approaches to measured rainfall-runoff data it was concluded that all models performed poorly and the QPB performance was only slightly improved by calibration of its most sensitive parameter, hydraulic conductivity. They write that the "conclusion one is forced to draw...is that the QPB model does not represent reality very well; in other words, there is considerable model error present. We suspect this is the case with most, if not all conceptual models currently in use." Additionally, "the fact

that simpler, less data intensive models provided as good or better predictions than a QPB is food for thought."

Based on the above selected sample of literature, the main difficulty in the use, calibration, and development, of complex models appears to be the lack of precise rainfall data and the high model sensitivity to (and magnification of) rainfall measurement errors. Nash and Sutcliff (1970) (15) write that "As there is little point in applying exact laws to approximate boundary conditions, this, and the limited ranges of the variables encountered, suggest the use of simplified empirical relations."

It is noteworthy to consider HEC Research Note No. 6 (1979) (16) where the Hydrocomp HSP continuous simulation model was applied to the West Branch DuPage River in Illinois. Personnel from Hydrocomp, HEC, and COE participated in this study which started with a nearly complete hydrologic/meteorologic data base. "It took one person six months to assemble and analyze additional data, and to learn how to use the model. Another six months were spent in calibration and long-record simulation." This time allocation applies to only a 28.5 mi^2 basin. The quality of the final model is indicated by the average absolute monthly volume error of 32.1 and 28.1 percent for calibration and verification periods, respectively. Peak flow rate average absolute errors were 26 and 36 percent for calibration and verification periods, respectively. It was concluded that "Discharge frequency under changing urban conditions is a problem that could be handled by simpler, quicker, less costly approaches requiring much less data; e.g., design storms or several historial events used as input into a single-event model, or a continuous model with a less complex soil-moisture accounting algorithm."

The complex model parameter optimization problem has not been resolved. For example, Gupta and Sarooshian (1983) (17) write that "even when calibrated under ideal conditions (simulation studies), it is often impossible to obtain unique estimates for the parameters." Troutman (1982) (18) also discusses the often cited difficulties with the error in precipitation measurements "due to the spatial variability of precipitation". This source of error can result in "serious errors in runoff prediction and large biases in parameter estimates by calibration of the model."

Because it still has not been well established in the open literature whether there is a significant advantage in using a watershed model more complex or physically based than a design storm unit hydrograph approach, the design storm unit hydrograph method will probably have continued widespread use among practitioners for flood control design and planning studies.

NONLINEARITY: USE OF NONLINEAR KINEMATIC WAVE METHOD OR A LINEAR UNIT HYDROGRAPH METHOD

The dominant method used in runoff hydrograph development for presenting catchment runoff response is the unit hydrograph (UH). The next most frequently used method is the kinematic wave overland flowplane concept (KW). HEC TD#15 (1982) (19) provides a description and comparison of these two alternatives. The relative usage of KW by 1983 is indicated in Cermak and Feldman (1983) (20) who write that "actual applications by Corps field offices have been few to nonexistent. Even at HEC the KW approach has not been utilized in any special assistance projects." The relatively small usage of KW were then explained as being due to the slack in hydrologic studies and due to unfamiliarity with the technique.

Watt and Kidd (1975) (21) write that in the comparison of so-called 'physically-based' or 'black-box' modeling types (e.g., UH or n-linear reservoirs) the differences are not clear. For example, "except for certain 'ideal' laboratory catchments, the flow does not conform to the sheet-flow model but instead occurs in many small rivulets...The choice is then between a 'black-box' model and a 'physically-based' model which is based on a physical situation quite different than the actual field situation, i.e., a 'black box' model."

However, use of KW implies a non-linear response whereas the UH implies a linear response. Nash and Sutcliffe (1970) (15) write tht "the UH assumption of a linear time invariant relationship cannot be tested because neither the input (effective rainfall) nor output (storm runoff) are unequivocally defined." Although watershed response is often considered to be mathemtically nonlinear, the nonlinearity of the total watershed response has not been shown to be exactly described as a KW. Indeed, a diffusion hydrodynamic model, DHM (Hromadka and Yen, 1986) (22), provides another nonlinear watershead response that includes an additional term in the governing St. Vanant flow equations and that may differ significantly in response from a KW model (e.g. overland flow planes with KW channel routing). There are an infinity of nonlinear mathematical representations possible as a combination of surface runoff and channel routing analogs, therefore, merely claiming that the response of a watershed model can be classified as 'nonlinear' is not proof that the model represents the true response of the catchment.

Given that the KW analog is only used to obtain an approximation to catchment response, the KW approach does not appear to provide significantly better computational results (for floods of interest in flood control design and planning) than the commonly used UH method. Dickenson et al. (1967) (23) noted that "in the range of discharges normally considered as flood hydrographs, the time (of concentration) remained virtually constant. In other words, in the range of flood interest, the nonlinear effect approached linearity." An explanation was advanced that "at low discharges, the mean velocity may vary considerably with discharge. However, for higher discharges contained within banks, the mean velocity in the channel remains approximately constant."

In actual travel time measurements of flows in a 96-acre catchment using a radioactive tracing technique, Pilgrim (1976) (24) noted that although the flood runoff process "is grossly nonlinear at low flows, linearity is approximated at high flows." Pilgrim also writes that "simple nonlinear models fitted by data from events covering the whole range of flow may give gross errors when used to estimate large events." It is noted that overbank flow was one of the factors for linearity in this study.

Beven (1979) (25) proposed to place limits on the nonlinearity associated to KW by the specification of a constant flow velocity for catchment runoff for large floods. He proposes "a nonlinear channel system at low flows and a linear system at high flows into a single model." Hence for flood flows of interest in flood control planning and design, Beven's model would reduce to a linear representation of the catchment hydraulics.

A physical test of the KW concept was provided by Hjelmfelt and Burwell (1984) (26), who studied a set of 40 similar erosion plots and the net response to storm events. Due to the large variability in measured runoff quantities from the plots, however, it was concluded that a criterion for a valid rainfall-runoff model "is that it predicts the mean runoff for each event." However, it is noted that this test may be more of a test of effective rainfall variability over the catchment than a test of KW response.

In HEC Technical Paper No. 59 (1978) (27), six models, plus two variants of one of these models and a variant of another, were calibrated and tested on a 5.5 mi^2 urban

catchment in Castro Valley near Oakland, California. Both single event and continuous simulation models based on both UH and KW techniques were used in the test. The study concluded that for this watershed "the more complex models did not produce better results than the simple models..." An examination of the test results between the KW and HEC-1 UH models did not show a clear difference between the methods.

It is of interest with Singh (1977) (28) concluded that "if one is not very confident in estimates of watershed infiltration then in some circumstances linear models may have an advantage over nonlinear models in runoff peak predictions because they do not amplify the input errors." That is, the uncertainity in effective rainfall quantities may be magnified by a nonlinear model; consequently, there is an advantage in using a linear model when there are errors in loss rate and precipitation estimates.

Because it has not been well established whether the nonlinear KW method for modeling surface runoff provides an improvement in accuracy over the linear UH based hydrologic models, the UH model will probably continue to be the most often used runoff model among practitioners.

DESIGN STORMS

HEC (Beard, 1975) (29) provides an in-depth study of the use of design runoff hydrographs for flood control studies. "Hypothetical floods consists of hydrographs of artifical flood flows...that can be used as a basis for flood-control planning, design and operation decisions or evaluations. These floods represent classes of floods of a specified or impied range of severity." Such "floods are ordinarily derived from rainfall or snowmelt or both, with ground conditions that are appropriate to the objectives of the study, but they can be derived from runoff data alone, usually on the basis of runoff volume and peak-flow frequency studies and representative time sequences of runoff."

In complex watershed systems that include catchment subareas, and channel and basin routing components, Beard (1975) (29) writes that "it is usually necessary to simulate the effects of each reservoir on downstream flows for all relevant magnitudes of peaks and volumes of inflows. Here it is particularly important that each hypothetical flood has a peak flow and volumes for all pertinent durations that are commensurate in severity, so that each computed regulated flow will have a probability or frequency that is comparable to that of the corresponding unregulated flow...In the planning of a flood control project involving storage or in the development of reservoir operation rules, it is not ordinarily known what the critical duration will be, because this depends on the amounts of reservoir space and release in relation to flood magnitude. When alternate types of projects are considered, critical durations will be different, and a design flood should reflect a degree of protection that is comparable for the various types of projects."

Beard (1975) (29) notes that the balanced storm concept is an important argument for not using a historic storm pattern or sequence of storm patterns (e.g., continuous simulation or discrete event modeling) as "No one historical flood would ordinarily be representative of the same severity of peak flow and runoff volumes for all durations of interest." Indeed, should a continuous simulation study be proposed such that the "project is designed to regulate all floods of record, it is likely that one flood will dictate the type of project and its general features, because the largest flood for peak flows is also usually the largest-volume flood." Hence a continuous simulation model of say 40 years of data can be thought of as a 40 year duration design storm with its own probability of re-occurrence, which typically reduces for modeling purposes to simply a single or double day storm pattern.

Beard and Chang (1979) (30) write that for design storm construction, "it is generally considered that a satisfactory procedure is to construct an aproximately symmetrical pattern of rainfall with uniform areal distribution having intensities for all durations corresponding to the same recurrence interval and for that location and size of area" (i.e., depth-area effects).

The nested design storm concept is developed in detail in HEC TD#15 (1982) (19), including the use of depth-area adjustments. Again, because the current alternatives to the design storm approach (i.e., continuous simulation or discrete event modeling) have not been well established in the literature to provide more accurate estimates of flood frequency values, the design storm approach probably will have continued widespread usage among practitioners.

MODEL SELECTION

Of the over 100 models available, a design storm/unit hydrograph model (i.e., "model") is currently the most widely used modeling technique among practitioners. Some of the reasons are as follows: (1) the design storm approach—the multiple discrete event and continuous simulation categories of models have not been clearly established to provide better predictions of flood flow frequency estimates for evaluating the impact of urbanization and for design flood control systems than a calibrated design storm model; (2) the unit hydrograph method—it has not been shown that the kinematic wave modeling technique provides a significantly better representation of watershed hydrologic response than a model based on unit hydrographs (locally calibrated or regionally calibrated) that represent free-draining catchments; (3) model usage—the "model" has been used extensively nationwide and has proved generally acceptable and reliable; (4) parameter calibration—the "model" usually is based on a minimal number of parameters, generally giving higher accuracy in calibration of model parameters to rainfall-runoff data, and the design storm to local flood flow frequency tendencies; (5) calibration effort—the "model" does not require large data or time requirements for calibration; (6) application effort—the "model" does not require a large computational effort for application; (7) acceptability--the "model" uses algorithms (e.g., convolution, etc.) that have gained acceptance in engineering practice; (8) model flexibility for planning—data handling and computational submodels can be coupled to the "model" (e.g., channel and basin routing) resulting in a highly flexible modeling capability; (9) model certainty evaluation--the certainty of modeling results can be readily evaluated as a distribution of possible outcomes over the probabilistic distribution of parameter values.

FURTHER RESEARCH NEEDS

Even though there are many hydrologic and "physically based" models reported in the literature which contain algorithms to model each of the specific hydrologic cycle processes, the basic design storm/unit hydrograph method continues to be the most widely used approach among practitioners. It appears that for another class of model to become the engineering standard tool, the model must clearly demonstrate the benefits in its use for the corresponding increase in computational effort. Such demonstrations include exhaustive comparisons of modeling performance in accuracy and reliability; not only in the reproduction of known storm events, but in the estimate of flood frequency peak flow rate estimates. Finally, storm runoff hydrograph analyses should be conducted as verification runs, where the storms tested are not elements of the calibration data set.

CONCLUSIONS

Although modern hydrologic models have reached a high level of elegance in the mathematical approximation of the rainfall-runoff process, the simpler models such as the classic unit hydrograph approach continue to be the most widespread used study tool among practitioners. In this paper, a review of the literature indicates that the more complex models have not sufficiently proven themselves to be significantly better computational tools. Indeed, many key reports indicate that the simpler UH modeling approach provides computational results which are as good as or better than those achieved by the more complex models.

Based on the selected literature, the uncertainty in the effective rainfall distribution over the catchment appears to be a major limitation to the successful development, calibration, and usage, of any hydrologic model. And this uncertainty in effective rainfall appears to be more of a problem to complex model performance than for simpler model's performance.

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UNCERTAINTY IN FLOOD CONTROL DESIGN

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ABSTRACT

The classic single area unit hydrograph (UH) approach to modeling runoff response from a free draining catchment is shown to represent several important modeling considerations including, (i) subarea runoff response (in a discretized model), (ii) the subarea effective rainfall distribution including variations in magnitude, timing, and storm pattern shape, (iii) channel flow routing translation and storage effects, using the linear routing technique, (iv) subarea runoff hydrograph addition, among other factors. Because the UH method correlates the effective rainfall distribution to the runoff hydrograph distribution, the resulting catchment UH should be considered a correlation distribution in a probabilistic sense. Should the uncertainty in rainfall over the catchment be a major concern in modeling reliability, then the UH output in the predictive setting must be considered to be a random variable.

INTRODUCTION

The current trend in hydrologic surface runoff model development is to discretize the catchment (assumed to be free draining) into several small subareas, each linked by a channel hydraulic flow routing algorithm. The resulting model is then formulated as a link node model which responds hydraulically according to a specified effective rainfall in each subarea. While over 100 such models have been developed in the open literature (Hromadka, 1987), none have been shown to provide consistently "better" results than the classic single area unit hydrograph (UH) methods in the estimation of severe storm runoff of interest in flood control. It is shown in this paper that the classic UH technique provides, (i) a rational modeling structure which properly represents several hydrologic effects which a highly discretized model misrepresents; (ii) a correlation distribution (distribution frequency of UH's) which correlates the effective rainfall to be measured runoff hydrograph; and (iii) a probabilistic model which represents the model output as a random variable, whose variance represents the natural variance between effective rainfall and runoff.

CATCHMENT AND DATA DESCRIPTION

Let R be a free draining catchment with negligible detention effects. R is discretized into m subareas, R_j, each draining to a nodal point which is drained by a channel system. The m-subarea link node model resulting by combining the subarea runoffs for storm i, $Q_j^i(t)$, adding runoff hydrographs at nodal points, and routing through the channel system, is denoted as $Q_m^i(t)$. It is assumed that there is only a single rain gage and stream gage available for data analysis. The rain gage site is monitored for the 'true' effective rainfall distribution, $e_g^i(t)$. The stream gage data represents the entire catchment, R, and is denoted by $Q_g^i(t)$.

LINEAR EFFECTIVE RAINFALLS FOR SUBAREAS

The effective rainfall distribution (rainfall less losses) in R_j is given by $e_j^i(t)$ for storm i where $e_j^i(t)$ is assumed to be linear in $e_g^i(t)$ by

$$e_j^i(t) = \sum \lambda_{jk}^i \, e_g^i(t - \theta_{jk}^i) \tag{1}$$

where λ_{jk}^i and θ_{jk}^i are coefficients and timing offsets, respectively, for storm i and subarea R_j. In Eq. (1), the variations in the effective rainfall distribution over R due to magnitude and timing are accounted for by the λ_{jk}^i and θ_{jk}^i, respectively. The subareas, R_j, are chosen such that Eq. (1) is a good approximation for each subarea.

SUBAREA RUNOFF

The storm i subarea runoff from R_j is given by $Q_j^i(t)$ where

$$Q_j^i(t) = \int_{s=0}^{t} e_j^i(t - s) \, \phi_j^i(s) \, ds \tag{2}$$

where $\phi_j^i(s)$ is the subarea unit hydrograph (UH) for storm i such that Eq. (2) applies. Combining Eqs. (1) and (2) gives

$$Q_j^i(t) = \int_{s=0}^{t} \sum e_g^i(t - \theta_{jk}^i - s) \, \lambda_{jk}^i \, \phi_j^i(s) \, ds \tag{3}$$

Rearranging variables,

$$Q_j^i(t) = \int_{s=0}^{t} e_g^i(t - s) \sum \lambda_{jk}^i \, \phi_j^i \, (s - \theta_{jk}^i) \, ds \tag{4}$$

where throughout this paper, arbitrary function $F(s - Z)$ is notation that $F(s - Z) = 0$ for $s < Z$.

LINEAR ROUTING

Let $I_1(t)$ be the inflow hydrograph to a channel flow routing link (number 1), and $O_1(t)$ the outflow hydrograph. A linear routing model of the unsteady flow routing process is given by

$$O_1(t) = \sum_{k_1=1}^{n_1} a_{k_1} \; I_1(t - \alpha_{k_1}) \tag{5}$$

where the a_{k_1} are coefficients which seem to unity; and the α_{k_1} are timing offsets. Again, $I_1(t - \alpha_{k_1}) = 0$ for $t < \alpha_{k_1}$. Given stream gage data for $I_1(t)$ and $O_1(t)$, the best fit values for the a_{k_1} and α_{k_1} can be determined.

Should the above outflow hydrograph, $O_1(t)$, now be routed through another link (number 2), then $I_2(t) = O_1(t)$ and from the above

$$O_2(t) = \sum_{k_2=1}^{n_2} a_{k_2} \; I_2(t - \alpha_{k_2})$$

$$= \sum_{k_2=1}^{n_2} a_{k_2} \sum_{k_1=1}^{n_1} a_{k_1} \; I_1(t - \alpha_{k_1} - \alpha_{k_2}) \tag{6}$$

For L links, each with their own respective stream gage routing data, the above linear routing technique results in the outflow hydrograph for link number L, $O_L(t)$, being given by

$$O_L(t) = \sum_{k_L=1}^{n_L} a_{k_L} \sum_{k_{L-1}=1}^{n_{L-1}} a_{k_{L-1}} \cdots \sum_{k_2=1}^{n_2} a_{k_2} \sum_{k_1=1}^{n_1} a_{k_1} I_1(t - \alpha_{k_1} - \alpha_{k_2} - \cdots - \alpha_{k_{L-1}} - \alpha_{k_L}) \tag{7}$$

Using vector notation, the above $O_L(t)$ is written as

$$O_L(t) = \sum_{<k>} a_{<k>} I_1(t - \alpha_{<k>}) \tag{8}$$

For subarea R_j, the runoff hydrograph for storm i, $Q_j^i(t)$, flows through L_j links before arriving at the stream gage and contributing to the total measured runoff hydrograph, $Q_g^i(t)$. All of the constants $a^i_{<k>}$ and $\alpha^i_{<k>}$ are available on a storm by storm basis. Consequently from the linearity of the routing technique, the m-subarea link node model is given by the sum of the m, $Q_j^i(t)$ contributions,

$$Q_m^i(t) = \sum_{j=1}^{m} \sum_{<k>_j} a^i_{<k>_j} Q^i_j(t - \alpha^i_{<k>_j}) \tag{9}$$

where each vector $<k>_j$ is associated to a R_j, and all data is defined for storm i. It is noted that in all cases,

$$\sum_{<k>_j} a^i_{<k>_j} = 1 \tag{10}$$

LINK-NODE MODEL, $Q_m^i(t)$

For the above linear approximations for storm i, Eqs. (1), (4), and (9) can be combined to give the final form for $Q_m^i(t)$,

$$Q_m^i(t) = \sum_{j=1}^{m} \sum_{<k>_j} a^i_{<k>_j} \int_{s=0}^{t} e_g^i(t-s) \sum \lambda^i_{jk} \phi_j^i(s - \theta^i_{jk} - \alpha^i_{<k>_j})\, ds \tag{11}$$

Because the measured effective rainfall distribution, $e_g^i(t)$, is independent of the model, Eq. (1) is rewritten in the final form

$$Q_m^i(t) = \int_{s=0}^{t} e_g^i(t-s) \sum_{j=1}^{m} \sum_{<k>_j} a^i_{<k>_j} \sum \lambda^i_{jk} \phi_j^i(s - \theta^i_{jk} - \alpha^i_{<k>_j})\, ds \tag{12}$$

where all parameters are evaluated on a storm by storm basis, i.

Equation (12) describes a model which represents the total catchment runoff response based on variable subarea UH's, $\phi_j^{\,i}(s)$; variable effective rainfall distributions on a subarea-by-subarea basis with differences in magnitude (λ_{jk}^{i}), timing (θ_{jk}^{i}), and pattern shape (linearity assumption); and channel flow routing translation and storage effects (parameters $a^{i}_{<k>_j}$ and $\alpha^{i}_{<k>_j}$).

MODEL REDUCTION

The m-subarea model of Eq. (12) is directly reduced to the simple single area UH model (no discretization of R into subareas) given by $Q_1^{\,i}(t)$ where

$$Q_1^{\,i}(t) = \int_{s=0}^{t} e_g^{\,i}(t-s)\ \eta^{i}(s)\ ds \qquad (13)$$

where $\eta^{i}(s)$ is the correlation distribution between the data pair $\{Q_g^{\,i}(t),\ e_g^{\,i}(t)\}$.

From Eq. (13) it is seen that the classic single area UH model represents a highly complex link node modeling structure. For the case of having available a single rain gage and stream gage for data correlation purposes, the derived $\eta^{i}(s)$ represents the several effects used in the development leading to Eq. (12), integrated according to the sample from the several parameters' respective probability distributions. Because the simple $Q_1^{\,i}(t)$ model structure actually includes most of the effects which are important in flood control hydrologic response, it can be used to develop useful probabilistic distributions of modeling output.

STORM CLASSIFICATION SYSTEM

To proceed with the analysis, the full domain of effective rainfall distributions measured at the rain gage site are categorized into storm classes, $<\xi_q>$. That is, any two elements of a class $<\xi_q>$ would result in nearly identical effective rainfall distributions at the rain gage site, and hence one would "expect" nearly identical resulting runoff hydrographs from the stream gage. Typically, however, the resulting runoff hydrographs differ and, therefore, the randomness of the effective rainfall distribution over R results in variations in the modeling "best-fit" parameters in correlating the available rainfall-runoff data.

More precisely, any element of a specific storm class $<\xi_o>$ has the effective rainfall distribution, $e_g^o(t)$. In correlating $\{Q_g^i(t), e_g^o(t)\}$, a different $\eta^i(s)$ results due to the variations in the measured $Q_g^i(t)$ with respect to the single $e_g^o(t)$.

In the predictive mode, where one is given an assumed (or design) effective rainfall distribution, $e_g^D(t)$, to apply at the rain gage site, the storm class of which $e_g^D(t)$ is an element of is identified, $<\xi_D>$, and the predictive output for the input $e_g^D(t)$ must necessarily be the distribution

$$[Q_1(t)] = \int_{s=0}^{t} e_g^D(t-s)\ [\eta(s)]_D\ ds \tag{14}$$

where $[\eta(s)]_D$ is the distribution of $\eta^i(s)$ distributions associated to storm class $[\xi_D]$.

Generally, however, there is insufficient rainfall-runoff data to derive a sufficiently unique set of storm classes, $<\xi_q>$, and hence additional assumptions must be used. For example, one may lower the eligibility standards for each storm class, $<\xi_q>$, implicitly assuming that several distributions $[\eta(s)]_q$ are nearly identical; or one may transfer $[\eta(s)]_q$ distributions from another rainfall-runoff data set, implicitly assuming that the two- catchment data set correlation distributions are nearly identical. A common occurrence is the case of predicting the runoff response from a design storm effective rainfall distribution, $e_g^D(t)$, which is not an element of any observed storm class. In this case, another storm class distribution of $[\eta(s)]_q$ must be used which implicitly assumes that the two sets of correlation distributions are nearly identical. Consequently for a severe design storm condition, it would be preferable to develop correlation distributions using the severe historic storms which have rainfall-runoff data available for analysis.

EFFECTIVE RAINFALL UNCERTAINTY

The paper by Hromadka, (1987)(1), includes brief statements from several reports which conclude that the variability in the rainfall (and hence the effective rainfall) over the catchment is a dominant factor in the development, calibration, and application, of hydrologic models (e.g., Schilling and Fuchs, 1986; among others)(2). Including this premise in hydrologic studies would indicate that hydrologic model estimates must be functions of random variables, and hence the estimates are random variables themselves.

From Eq. (12), the correlation distribution for storm event i, $\eta^i(s)$, includes all the uncertainty in the effective rainfall distribution over R, as well as the uncertainty in the runoff and flow routing processes. That is, $\eta^i(s)$ must be an element of the random variable $[\eta(s)]$ where

$$\eta^i(s) = \sum_{j=1}^{m} \sum_{<k>_j} a^i_{<k>_j} \sum \lambda^i_{jk} \phi_j^{\,i}(s - \theta^i_{jk} - \alpha^i_{<k>_j}) \qquad (15)$$

and Eq. (15) applies to a specific storm. For severe storms of flood control interest, one would be dealing with only a subset of the set of all storm classes. In a particular storm class, $<\xi_o>$, should it be assumed that the subarea runoff parameters and channel flow routing uncertainties are minor in comparison to the uncertainties in the effective rainfall distribution over R (e.g., Schilling and Fuchs, 1986; among others), then Eq. (15) may be written as

$$[\eta(s)]_o = \sum_{j=1}^{m} \sum_{<k>_j} \bar{a}_{<k>_j} \sum [\lambda_{jk}]\, \bar{\phi}_j\, (s - [\theta_{jk}] - \bar{\alpha}_{<k>_j}) \qquad (16)$$

where the overbars are notation for mean values of the parameters for storm class $<\xi_o>$. Although use of Eq. (14) in deriving the $[\eta(s)]$ distributions results in both the uncertainties in both the effective rainfalls and also the submodel algorithms being integrated, Eq. (16) is useful in motivating the use of the distribution concept in design and planning studies for all hydrologic models, based on just the magnitude of the uncertainties in the effective rainfall distribution over R. That is, although one may argue that a particular model is "physically based" and represents the "true" hydraulic response distributed throughout the catchment, the uncertainty in rainfall still remains and is not reduced by increasing hydraulic routing modeling complexity.

DISCRETIZATION ERROR

The need for using the $Q_1(t)$ model in studies where detention effects are minor is made more apparent when examining the effects of discretizing the model into subareas without the benefit of subarea rainfall-runoff data.

In the above typical case, the engineer generally assigns the recorded precipitation from the single available rain gage, $P_g{}^i(t)$, to occur simultaneously over each R_j. That is from Eq. (1), the $\theta_{jk}^i = 0$ and the λ_{jk}^i are set to constants $\hat{\lambda}_j$ which reflect only the variations in loss rate nonhomogeneity. Hence, the 'true' $Q_m{}^i(t)$ model of Eq. (12), (and also Eq. (13)), becomes the estimator $\hat{Q}_m{}^i(t)$ where

$$\hat{Q}_m{}^i(t) = \int_{s=0}^{t} \hat{e}_g{}^i(t-s) \sum_{j=1}^{m} \sum_{<k>_j} \hat{a}^i{}_{<k>_j} \sum \hat{\lambda}_j \hat{\phi}_j{}^i (s - \hat{\alpha}^i{}_{<k>_j})\, ds \qquad (17)$$

where hats are notation for estimates. These incorrect assumptions result in 'discretization error'. Indeed, an obvious example of discretization error is the case where a subarea R_j actually receives no rainfall, and yet one assumes that $P_g{}^i(t)$ occurs over R_j in the discretized model. (It is easily shown that the Eq. (13) model accommodates this example case.)

DISCRETIZATION CALIBRATION ERROR

A current trend among practitioners is to develop an m-subarea link-node model estimator $\hat{Q}_m{}^i(t)$ such as Eq. (17), and then "calibrate" the model parameters using the available (single) rain gage and stream gage data pair. Because subarea rainfall-runoff data are unavailable, necessarily it is assumed that the random variables associated to the subarea effective rainfalls are given by

$$\left.\begin{aligned}[\theta_{jk}] &= 0 \\ [\lambda_{jk}] &= \hat{\lambda}_j\end{aligned}\right\} \begin{array}{l}\text{(estimator, } \hat{Q}_m{}^i(t), \\ \text{assumptions)}\end{array} \tag{18}$$

But these assumptions violate the previously stated premise that the uncertainty in the effective rainfall distribtion over R has a major effect in hydrologic modeling accuracy. The impact in using Eq. (18) becomes apparent when calibrating the model to only storms of a single storm class, $<\xi_o>$.

Again, for all storms in $<\xi_o>$, the effective rainfall distributions are all nearly identical and are essentially given by the single $e_g{}^o(t)$. But due to the variability in rainfall over R_j, the associated runoff hydrographs, $Q_g{}^i(t)$, differ even though $e_g{}^o(t)$ is the single model input.

It is recalled that in Eq. (17), the effective rainfall distribution is now the estimator, $\hat{e}_g{}^o(t)$, which is the true $e_g{}^i(t)$ modified to best correlate $\{Q_g{}^i(t), e_g{}^o(t)\}$. That is, due to the several assumptions leading to Eq. (18) for the discretized model estimator, $\hat{Q}_m{}^i(t)$, the variations due to $[\lambda_{jk}]$ and $[\theta_{jk}]$ are transferred from the $[\eta(s)]$ distribution to the $\hat{e}_g{}^i(t)$ function. For storm class $<\xi_o>$, the estimator $\hat{Q}_m{}^i(t)$ can be written approximately from Eqs. (16) and (17) as

$$\hat{Q}_m{}^i(t) = \int_{s=0}^{t} \hat{e}_g{}^i(t-s) \sum_{j=1}^{m} \sum_{<k>} \bar{a}_{<k>j} \sum \hat{\lambda}_j \, \bar{\phi}_j(s - \bar{\alpha}_{<k>j}) \, ds \tag{19}$$

where in Eq. (19), it is assumed that the variations in model output due to using mean values (overbar notation) are minor in comparison to the variations in model output due to $[\lambda_{jk}]$ and $[\theta_{jk}]$. But then Eq. (19) is another single area UH model,

$$\hat{Q}_m{}^i(t) = \int_{s=0}^{t} \hat{e}_g{}^i(t-s)\ \hat{\eta}(s)\ ds \tag{20}$$

where $\hat{\eta}(s)$ is an estimated distribution which is essentially 'fixed' for all storms in a specified storm class $<\xi_o>$. The $\hat{\eta}(s)$ is fixed due to nearly the same input being applied to each subarea for each storm in $<\xi_o>$. In calibrating $\hat{Q}_m{}^i(t)$, therefore, the work effort is focused towards finding the best fit effective rainfall distribution, $\hat{e}_g{}^i(t)$, which correlates the several pairs $\{Q_j{}^i(t), \hat{\eta}(s)\}$. That is, the 'true' single $e_g{}^o(t)$ is forced to be modified to be $\hat{e}_g{}^i(t)$ in order to correlate the $\{Q_g{}^i(t), \hat{\eta}(s)\}$, for each storm, i. This contrasts with finding the best fit $\eta^i(s)$ which correlates the pairs, $\{Q_g{}^i(t), e_g{}^o(t)\}$. It is recalled that from Eqs. (16), (17), and (20), $\hat{\eta}(s)$ is "fixed" due to the assumptions of Eq. (18), and due to using a single storm class, $<\xi_o>$.

Because the effective rainfall submodel used in $\hat{Q}_m{}^i(t)$ has a prescribed structure, it cannot match the best fit $\hat{e}_g{}^i(t)$ for all storms and, consequently, modeling error is introduced into the calibration parameters of the loss rate submodel in order to (1) modify the true $e_g{}^o(t)$ due to the effects of $[\lambda_{jk}]$ and $[\theta_{jk}]$; (2) the derivation of loss rate parameters which are not "physically based".

Another error which results due to use of Eq. (18) is that the estimator modeling distribution $[\hat{Q}_m(t)]$ for storm class $<\xi_o>$ will be imprecise due to the variation in derived loss rate parameters not achieving the true variation in $\hat{e}_g{}^i(t)$.

The above results indicate that for the given assumptions, the calibration of a highly discretized catchment model will generally lead to a model that is no more reliable in the predictive mode than the simple single area UH model. These results appear to be validated by the open literature (Hromadka, 1987).

EXPECTED VALUE ESTIMATES

In practice, the single area UH model is used to correlate several record data pairs $\{Q_g^{\,i}(t),\ e_g^{\,i}(t)\}$ of the same or similar storm class $<\xi_o>$ to derive the associate correlation distributions, $\{\eta^i(s)\}$. Although the $\{\eta^i(s)\}$ are often integrated and normalized, and the several normalizing parameters averaged together, the net effect of all this is finding the expected value of the distribution of correlation distributions, denoted by $E[\eta(s)]$. Then, the model used for predictive purposes (for storms of the same class, $<\xi_o>$ used to develop $[\eta(s)]$) is the expected distribution $E[Q_1(t)]$ given by

$$E[Q_1(t)] = \int_{s=0}^{t} e_g(t)\ E[\eta(s)]\ ds \tag{21}$$

where $e_g(t)$ is a design input in $<\xi_o>$. From Eqs. (12) and (13), $E[Q_1(t)] = E[Q_m(t)]$ which is the 'true' expected distribution for the given assumptions leading to Eq. (12).

In comparison, after calibrating the estimator, $\hat{Q}_m^{\,i}(t)$, to the available data, the averaging of parameters results in the model (for storms in $<\xi_o>$)

$$E[\hat{Q}_m(t)] = \int_{s=0}^{t} E[\hat{e}_g(t)]\ \hat{\eta}(s)\ ds \tag{22}$$

where $E[\hat{e}_g(t)]$ is the "best fit" to the expected value of the true effective rainfalls (needed to correlate the $\{Q_g^i(t), \hat{n}(s)\}$) using a specified rigid link-node model structure.

Comparing Eqs. (21) and (22), it is seen that for storm class $<\xi_o>$, Eq. (21) is the 'true' expected value.

VERIFICATION TESTS OF MODELS

From Eqs. (21) and (22), the standard use of verification tests on the models of $E[Q_1(t)]$ and $E[\hat{Q}_m(t)]$ simply test the distribution of $[Q_g(t)]$ about the mean estimates of $E[Q_1(t)]$ and $E[\hat{Q}_m(t)]$ for storm class $<\xi_o>$. The discrepancies reported in the literature for verification tests indicates that the natural variance between the $e_g^i(t)$ and $Q_g^i(t)$ is usually quite large.

CERTAINTY IN FLOOD CONTROL DESIGN

Recalling the premise that the variations in the effective rainfall distribution over the catchment, R, has a major impact on modeling accuracy, it may be questioned whether using the expected value of a model output is the proper use of a probabilistic distribution.

For example, suppose that a rain gage station with an extremely long record length shows that a severe storm condition occurs fairly frequently (say, about every 100 years), and each occurrence results in a nearly identical effective rainfall distribution at the rain gage site. Hence, a storm class of design interest is well defined, $<\xi_D>$, where each element has a nearly identical input, $e_g^D(t)$, for any catchment hydrologic model. Yet the catchment stream gage shows a variation in the runoff hydrographs, $Q_g^i(t)$, for each event of $e_g^D(t)$. From this information, a model distribution is derived from Eqs. (12) and (13) to give

$$[Q_D(t)] = \int_{s=0}^{t} e_g^{\,D}(t-s)[n^D(s)]\, ds \qquad (23)$$

Equation (23) is the distribution of hydrologic modeling estimates (see Figure 1), and is the best estimate available. Given another design storm event, with the same $e_g^{\,D}(t)$ resulting, the best a model can do in estimating the resulting runoff hydrograph is reflected in Eq. (23), and Figure 1.

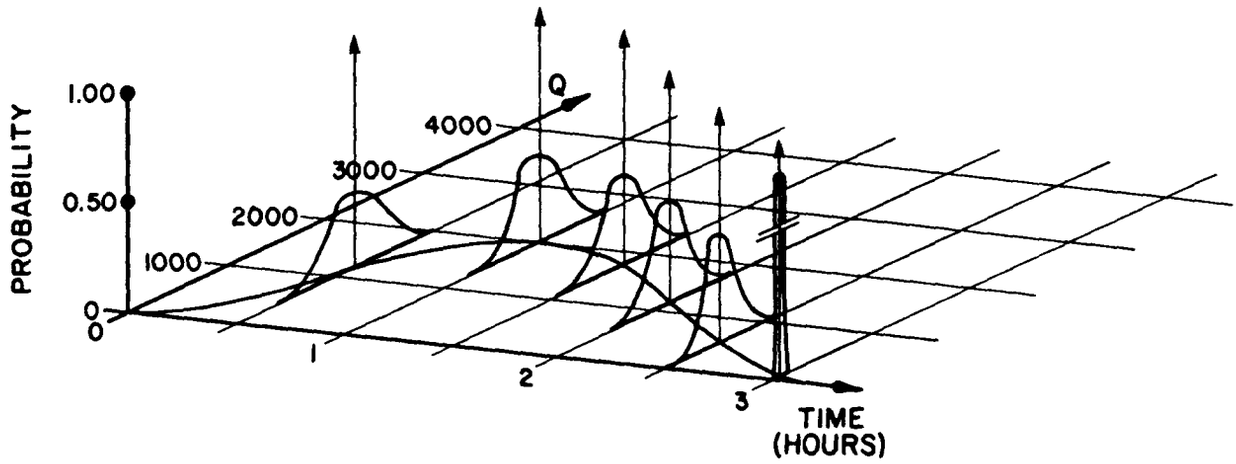

Figure 1. The Hydrologic Model Distribution (Eq. 23)) for a Predicted Response, $[Q_D(t)]$, from Input, $e_g^D(t)$. Heavy line is the Expected Distribution, $E[Q_D(t)]$

Should the expected model $E[Q_D(t)]$ be used for design study purposes, this expected runoff hydrograph typically would not be the most severe design condition for flood control facilities. Instead, the true distribution $[Q_D(t)]$ should be used to evaluate the flood control system performance, and a level of confidence selected as to the success in predictive design. That is, using the $E[Q_D(t)]$ model for design purposes often results in a design product that has only a 50-percent confidence level of protecting

for the specified design event, $e_g^D(t)$, given the available rainfall-runoff data. Perhaps a higher level of confidence, such as 85-percent or 95-percent, may be more appropriate in the interest of public safety, and to reduce the exposure to flood damage liability.

USING THE HYDROLOGIC MODEL DISTRIBUTION $[Q_1(t)]$

From the development leading to the model of Eq. (12), use of the standard single area UH model of Eq. (13) has a powerful representation of the catchment response including: random variations in the effective rainfall distribution pattern shape, magnitude, and timing, on an arbitrarily discretized subarea basis; variations in the subarea runoff response and channel flow routing effects on a storm by storm basis; storage effects in channel routing; among others. Calibration of the $Q_1(t)$ model to rainfall-runoff data on a storm class basis results in a distribution of correlation distributions, $[\eta(s)]$, which reflects the natural variance between the record data. The resulting model distribution, $[Q_1(t)]$, reflects the natural variance in predicting runoff quantities for storms of the same class used to derive $[\eta(s)]$.

The link node model estimator, $\hat{Q}_m^i(t)$, however, cannot achieve the true distribution of $[Q_1(t)]$. Only if rainfall-runoff data were available in each subarea (in order to determine the λ_{jk}^i and θ_{jk}^i on a storm by storm basis) would the model parameters (e.g., the loss rate model parameters be properly calibrated and the variance due to the rainfall effects (i.e., $[\lambda_{jk}]$ and $[\theta_{jk}]$ in Eq. (12)) be properly reflected. Consequently, $[Q_1(t)]$ should be used. The distribution $[\hat{Q}_m(t)]$, developed by varying the loss rate parameters (as the routing parameters are nearly invariant for storms of the same class), cannot

achieve the true variance between rainfall-runoff due to the loss rate algorithm structure. If $\hat{Q}_m{}^i(t)$ were supplied subarea rainfall-runoff data, and stream gage data to evaluate all routing parameters, then $\hat{Q}_m{}^i(t) = Q_m{}^i(t) = Q_1{}^i(t)$. That is, given enough runoff data to evaluate all model parameters on a subarea and link basis, the link node model will achieve the distribution variance between model output and the given rainfall data as achieved by the classic single area UH model.

APPLICATION: DETENTION BASIN VOLUME SIZING

The above developments are now applied to a simple application. A catchment of 1,800-acres is studied to size a detention basin. The design objective is to protect for a historic design storm. Based on the available stream gage and rain gage data, a class of severe storms, $<\xi_o>$, is developed and the $Q_1{}^i(t)$ model is calibrated for each element of $<\xi_o>$. The resulting $[\eta(s)]$ distribution is shown in mass curve form, $[M(s)]$, where

$$M^i(s) = \int_{x=0}^{t} \eta^i(x)\, dx \tag{24}$$

A frequency distribution for $[M(s)]$ is shown in Fig. 2.

Using $[M(s)]$, the $[\eta(s)]$ is found by differentiation and the model distribution, $[Q_D(t)]$, is given by Eq. (23) and shown in Fig. 1. Routing the $[Q_D(t)]$ through the detention basin resulted in the volume requirement distribution shown in Fig. 3. Shown in the figure is the expected volume requirement using $E[Q_D(t)]$, and also the 50-percent and 85-percent confidence estimates. Note that in this case, the "expected" volume requirement derived by using $E[Q(t)]$ (such as done in usual practice) is slightly less than the 50-percent confidence estimate.

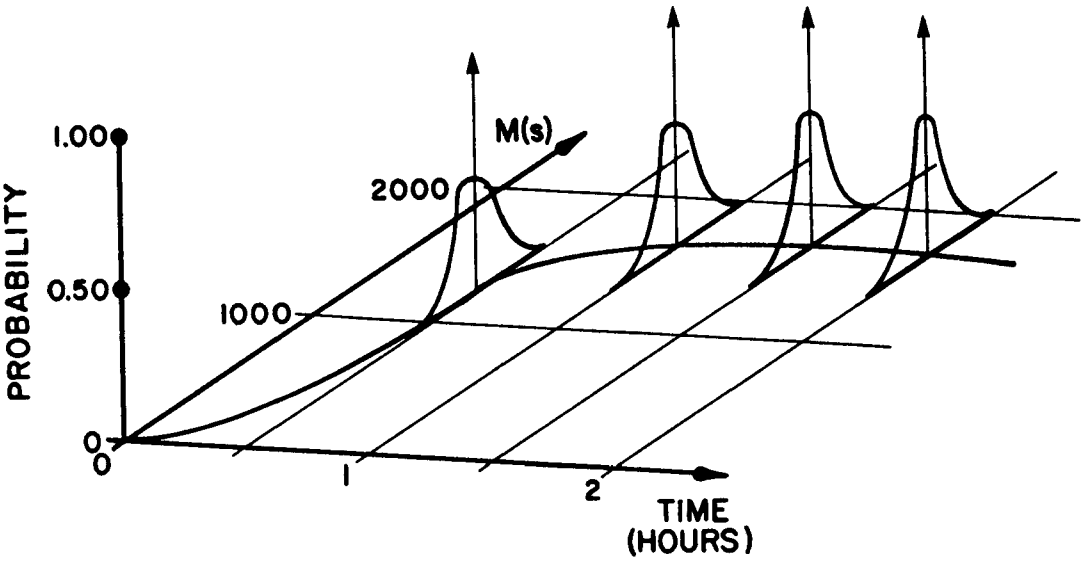

Figure 2. Frequency Distribution for [M(s)]. Heavy Line is the Expected Distribution, E[M(s)]

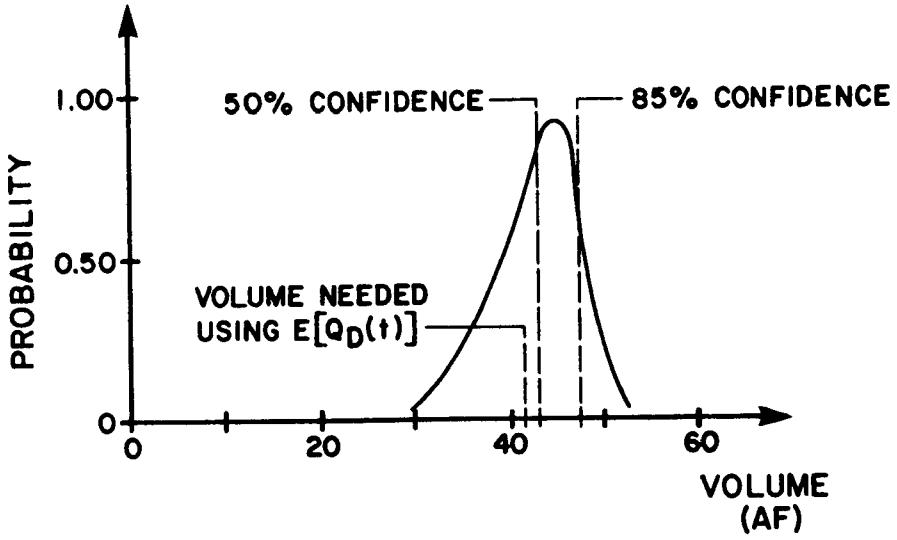

Figure 3. Detention Basin Volume Requirements

CONCLUSIONS

The classic single area unit hydrograph approach to modeling runoff response from a free draining catchment is shown to represent several important modeling considerations including, (i) subarea runoff response (in a discretized model), (ii) the subarea effective rainfall distribution including variations in magnitude, timing, and storm pattern shape, (iii) channel flow routing translation and storage effects, using the linear routing technique, (iv) subarea runoff hydrograph addition, among other factors. Because the UH method correlates the effective rainfall distribution to the runoff hydrograph distribution, the resulting catchment UH should be considered a correlation distribution in a probabilistic sense. Should the uncertainty in rainfall over the catchment be a major concern in modeling reliability, then the UH output in the predictive setting must be considered to be a random variable. In this paper, the UH method is shown to have a rational modeling structure for free-draining catchments. The correlations represented by the class of UH's derived from similarly categorized storms, properly reproduces the natural variance between the effective rainfall and runoff hydrograph. By using the full set of observed UH's (from the same storm category), a design product can be developed which accommodates modeling uncertainty due to the uncertainty in rainfall and other factors. The resulting UH model is then interpreted to be a probabilistic distribution, in which a flood control design needs to be tested by probabilistic simulation, varying the UH according to its frequency distribution. As a case study, a distribution of runoff hydrographs is used to estimate multi-outlet retarding basin design volume requirements.

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